ANSI/AISC 341-16
An American National Standard

Seismic Provisions for Structural Steel Buildings

July 12, 2016

Supersedes the *Seismic Provisions for Structural Steel Buildings*
dated June 22, 2010 and all previous versions

Approved by the Committee on Specifications

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PREFACE

(This Preface is not a part of ANSI/AISC 341-16, *Seismic Provisions for Structural Steel Buildings,* but is included for informational purposes only.)

AISC 360, *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) is intended to cover common design criteria. Accordingly, it is not feasible for it to also cover all of the special and unique problems encountered within the full range of structural design practice. This document, *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-16) (hereafter referred to as the Provisions), is a separate consensus standard that addresses one such topic: the design and construction of structural steel and composite structural steel/reinforced concrete building systems specifically detailed for seismic resistance.

The Symbols, Glossary, and Abbreviations are all considered part of this document. Accompanying the Provisions is a nonmandatory Commentary with background information and nonmandatory user notes interspersed throughout to provide guidance on the specific application of the document.

A number of significant technical modifications have also been made since the 2010 edition of the Provisions, including the following:

- Inclusion of ASTM A1085/A1085M material
- New provisions for diaphragms, chords and collectors, particularly horizontal truss diaphragms
- Inclusion of R_y in Table D1.1 for more accurate slenderness limits and to avoid use of lower F_y values for dual-certified material
- Requirement that simultaneous inelasticity be considered for columns participating in two or more seismic force resisting systems
- Clearer provisions on required strength of column splices and bases, including a reduced shear for column bases, returning the requirements to closer to those in the 2005 Provisions
- Allowance for non-full strength connections in special moment frames
- Option to use partial-joint-penetration groove welds in moment-frame column splices
- Revised and clarified continuity plate, doubler plate, and associated welding provisions
- Multi-tiered braced frame provisions for ordinary concentrically braced frames, special concentrically braced frames, and buckling-restrained braced frames
- Numerous revisions to special plate shear wall requirements
- New application of composite plate shear wall system using concrete-filled steel panel walls
- Power-actuated fasteners permitted in the protected zone up to a certain diameter
- New criteria to prequalify connections for composite moment frames

The AISC Committee on Specifications, Task Committee 9—Seismic Design is responsible for the ongoing development of these Provisions. The AISC Committee on Specifications gives final approval of the document through an ANSI-accredited balloting process, and has enhanced these Provisions through careful scrutiny, discussion and suggestions for improvement. The contributions of these two groups, comprising well more than 80 structural engineers with experience from throughout the structural steel industry, is gratefully acknowledged. AISC further acknowledges the significant contributions of the Building Seismic Safety Council (BSSC), the Federal Emergency Management Agency (FEMA), the National Science Foundation (NSF), and the Structural Engineers Association of California (SEAOC).

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The Committee gratefully acknowledges the following task committee (TC 9—Seismic Design) for their development of this document.

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SYMBOLS

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

Symbol	Definition	Reference
A_b	Cross-sectional area of a horizontal boundary element, in.^{2} (mm^2)	F5.5b
A_c	Cross-sectional area of a vertical boundary element, in.^{2} (mm^2)	F5.5b
A_{cw}	Area of concrete between web plates, in.^{2} (mm^2)	H7.5b
A_f	Gross area of flange, in.^{2} (mm^2)	E4.4b
A_g	Gross area, in.^{2} (mm^2)	E3.4a
A_{lw}	Web area of link (excluding flanges), in.^{2} (mm^2)	F3.5b
A_s	Cross-sectional area of the structural steel core, in.^{2} (mm^2)	D1.4b
A_{sc}	Cross-sectional area of the yielding segment of steel core, in.^{2} (mm^2)	F4.5b
A_{sh}	Minimum area of tie reinforcement, in.^{2} (mm^2)	D1.4b
A_{sp}	Horizontal area of stiffened steel plate in composite plate shear wall, in.^{2} (mm^2)	H6.3b
A_{sr}	Area of transverse reinforcement in coupling beam, in.^{2} (mm^2)	H4.5b
A_{sr}	Area of longitudinal wall reinforcement provided over the embedment length, L_e, in.^{2} (mm^2)	H5.5c
A_{st}	Horizontal cross-sectional area of the link stiffener, in.^{2} (mm^2)	F3.5b
A_{sw}	Area of steel web plates, in.^{2} (mm^2)	H7.5b
A_{tb}	Area of transfer reinforcement required in each of the first and second regions attached to each of the top and bottom flanges, in.^{2} (mm^2)	H5.5c
A_{tw}	Area of steel beam web, in.^{2} (mm^2)	H5.5c
A_w	Area of steel beam web, in.^{2} (mm^2)	H4.5b
C_a	Ratio of required strength to available axial yield strength	Table D1.1
C_d	Coefficient relating relative brace stiffness and curvature	D1.2a
D	Dead load due to the weight of the structural elements and permanent features on the building, kips (N)	D1.4b
D	Outside diameter of round HSS, in. (mm)	Table D1.1
D	Diameter of the holes, in. (mm)	F5.7a
E	Seismic load effect, kips (N)	F1.4a
E	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	Table D1.1
E_{cl}	Capacity-limited horizontal seismic load effect	B2
E_{mh}	Horizontal seismic load effect, including the overstrength factor, kips (N) or kip-in. (N-mm)	B2
F_{cr}	Critical stress, ksi (MPa)	F1.6a
F_{cre}	Critical stress calculated from *Specification* Chapter E using expected yield stress, ksi (MPa)	F1.6a

F_y Specified minimum yield stress, ksi (MPa). As used in the *Specification*, "yield stress" denotes either the minimum specified yield point (for those steels that have a yield point) or the specified yield strength (for those steels that do not have a yield point). A3.2

F_{yb} Specified minimum yield stress of beam, ksi (MPa) E3.4a

F_{yc} Specified minimum yield stress of column, ksi (MPa) E3.4a

F_{ysc} Specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, ksi (MPa) F4.5b

F_{ysr} Specified minimum yield stress of the ties, ksi (MPa) D1.4b

F_{ysr} Specified minimum yield stress of transverse reinforcement, ksi (MPa). H4.5b

F_{ysr} Specified minimum yield stress of transfer reinforcement, ksi (MPa) . . . H5.5c

F_{yw} Specified minimum yield stress of web skin plates, ksi (MPa) H7.5b

F_u Specified minimum tensile strength, ksi (MPa) A3.2

H Height of story, in. (mm) D2.5c

H_c Clear height of the column between beam connections, including a structural slab, if present, in. (mm) F2.6d

H_c Clear column (and web-plate) height between beam flanges, in. (mm) F5.7a.3

I Moment of inertia, in.^{4} (mm^4) E4.5c

I_b Moment of inertia of a horizontal boundary element taken perpendicular to the plane of the web, in.^{4} (mm^4) F5.4a

I_c Moment of inertia of a vertical boundary element taken perpendicular to the plane of the web, in.^{4} (mm^4) F5.4a

I_x Moment of inertia about an axis perpendicular to the plane of the EBF, in.^{4} (mm^4) F3.5b.1

I_y Moment of inertia about an axis in the plane of the EBF in.^{4} (mm^4) F3.5b

I_y Moment of inertia of the plate about the y-axis, in.^{4} (mm^4) F5.7b

K Effective length factor. F1.5b

L Live load due to occupancy and moveable equipment, kips (N) D1.4b

L Length of column, in. (mm) E3.4c

L Span length of the truss, in. (mm) E4.5c

L Length of brace, in. (mm) F1.5b

L Distance between vertical boundary element centerlines, in. (mm). F5.4a

L_b Length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm) D1.2a

L_c Effective length = KL, in. (mm) F1.5b

L_{cf} Clear length of beam, in. (mm). E1.6b

L_{cf} Clear distance between column flanges, in. (mm) F5.5b

L_e Embedment length of coupling beam, in. (mm). H4.5b

L_h Distance between beam plastic hinge locations, as defined within the test report or ANSI/AISC 358, in. (mm). E2.6d

L_s Length of the special segment, in. (mm) E4.5c

M_a Required flexural strength, using ASD load combinations, kip-in. (N-mm) D1.2c

M_f	Maximum probable moment at the column face, kip-in. (N-mm) E3.6f.1
M_{nc}	Nominal flexural strength of a chord member of the special segment, kip-in. (N-mm) E4.5c
$M_{n,PR}$	Nominal flexural strength of PR connection, kip-in. (N-mm) E1.6c
M_p	Plastic bending moment, kip-in. (N-mm) E1.6b
M_p	Plastic bending moment of a link, kip-in. (N-mm). F3.4a
M_p	Plastic bending moment of the steel, concrete-encased or composite beam, kip-in. (N-mm) G2.6b
M_p	Moment corresponding to plastic stress distribution over the composite cross section, kip-in. (N-mm). G4.6c
M_{pc}	Plastic bending moment of the column, kip-in. (N-mm) D2.5c
M_{pcc}	Plastic flexural strength of a composite column, kip-in. (N-mm) G2.6f
$M_{p,exp}$	Expected flexural strength, kip-in. (N-mm) D1.2c
M_{pr}	Maximum probable moment at the location of the plastic hinge, as determined in accordance with ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2, kip-in. (N-mm) E3.4a
M_r	Required flexural strength, kip-in. (N-mm) D1.2a
M_u	Required flexural strength, using LRFD load combinations, kip-in. (N-mm) D1.2c
M_{uv}	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline, kip-in. (N-mm) G3.4a
M_v	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD or ASD load combinations, kip-in. (N-mm) E3.4a
M_y	Yield moment corresponding to yielding of the steel plate in flexural tension and first yield in flexural compression. H7.5a
M^*_{pb}	Projection of the expected flexural strength of the beam as defined in Section E3.4a, kip-in. (N-mm) E3.4a
M^*_{pc}	Projection of the nominal flexural strength of the column as defined in Section E3.4a, kip-in. (N-mm) E3.4a
M^*_{pcc}	Projection of the nominal flexural strength of the composite or reinforced concrete column as defined in Section G3.4a, kip-in. (N-mm) . . . . . ….G3.4a
$M^*_{p,exp}$	Projection of the expected flexural strength of the steel or composite beam as defined in Section G3.4a, kip-in. (N-mm) G3.4a
N_r	Number of horizontal rows of perforations F5.7a
P_a	Required axial strength using ASD load combinations, kips (N). . . Table D1.1
P_{ac}	Required compressive strength using ASD load combinations, kips (N) E3.4a
P_b	Axial design strength of wall at balanced condition, kips (N) H5.4
P_c	Available axial strength, kips (N) E3.4a
P_n	Nominal axial compressive strength, kips (N) D1.4b

P_{nc} Nominal axial compressive strength of the chord member at the ends, kips (N) E4.4c

P_{nc} Nominal axial compressive strength of diagonal members of the special segment, kips (N) E4.5c

P_{nt} Nominal axial tensile strength of a diagonal member of the special segment, kips (N) E4.5c

P_r Required axial compressive strength, kips (N) E3.4a

P_{rc} Required axial strength, kips (N) E5.4a

P_u Required axial strength using LRFD load combinations, kips (N) Table D1.1

P_{uc} Required compressive strength using LRFD load combinations, kips (N) E3.4a

P_y Axial yield strength , kips (N) . . . . . Table D1.1

P_{ysc} Axial yield strength of steel core, kips (N) F4.2a

$P_{ysc\text{-}max}$ Maximum specified axial yield strength of steel core, ksi (MPa) F4.4d

$P_{ysc\text{-}min}$ Minimum specified axial yield strength of steel core, ksi (MPa) F4.4d

R Seismic response modification coefficient A1

R Radius of the cut-out, in. (mm) F5.7b

R_c Factor to account for expected strength of concrete = 1.5 . . . . . H5.5d

R_n Nominal strength, kips (N) A3.2

R_t Ratio of the expected tensile strength to the specified minimum tensile strength F_u A3.2

R_y Ratio of the expected yield stress to the specified minimum yield stress, F_y A3.2

R_{yr} Ratio of the expected yield stress of the transverse reinforcement material to the specified minimum yield stress H5.5d

S_{diag} Shortest center-to-center distance between holes, in. (mm) F5.7a

T_1 Tension force resulting from the locally buckled web plates developing plastic hinges on horizontal yield lines along the tie bars and at mid-vertical distance between tie bars H7.4e

T_2 Tension force that develops to prevent splitting of the concrete element on a plane parallel to the steel plate H7.4e

V_a Required shear strength using ASD load combinations, kips (N) E1.6b

V_{comp} Limiting expected shear strength of an encased composite coupling beam, kips (N) H4.5b

V_n Nominal shear strength of link, kips (N) F3.3

V_n Expected shear strength of a steel coupling beam, kips (N). H5.5c

$V_{n,comp}$ Expected shear strength of an encased composite coupling beam, kips (N). H4.5b

$V_{n,\ connection}$ Nominal shear strength of coupling beam connection to wall pier, kips (N) H4.5b

V_{ne} Expected vertical shear strength of the special segment, kips (N) E4.5c

V_p Plastic shear strength of a link, kips (N) F3.4a

V_r Required shear strength using LRFD or ASD load combinations, kips (N) F3.5b

V_u Required shear strength using LRFD load combinations, kips (N) E1.6b

V_y	Shear yield strength, kips (N)	F3.5b
Y_{con}	Distance from the top of the steel beam to the top of concrete slab or encasement, in. (mm)	G3.5a
Y_{PNA}	Maximum distance from the extreme concrete compression fiber to the plastic neutral axis, in. (mm)	G3.5a
Z	Plastic section modulus about the axis of bending, in.^{3} (mm^3)	D1.2a
Z_c	Plastic section modulus of the column about the axis of bending, in.^{3} (mm^3)	E3.4a
Z_x	Plastic section modulus about x-axis, in.^{3} (mm^3)	E2.6g
a	Distance between connectors, in. (mm)	F2.5b
b	Width of compression element as defined in *Specification* Section B4.1, in. (mm)	Table D1.1
b	Inside width of a box section, in. (mm)	F3.5b
b_{bf}	Width of beam flange, in. (mm)	E3.6f
b_f	Width of flange, in. (mm)	D2.5b
b_w	Thickness of wall pier, in. (mm)	H4.5b
b_w	Width of wall, in. (mm)	H5.5c
b_{wc}	Width of concrete encasement, in. (mm)	H4.5b
d	Overall depth of beam, in. (mm)	Table D1.1
d	Nominal bolt diameter, in. (mm)	D2.2
d	Overall depth of link, in. (mm)	F3.5b
d_c	Effective depth of concrete encasement, in. (mm)	H4.5b
d_z	d-$2t_f$ of the deeper beam at the connection, in. (mm)	E3.6e
d^*	Distance between centroids of beam flanges or beam flange connections to the face of the column, in. (mm)	E3.6f
e	Length of EBF link, defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face, in. (mm)	F3.5b
f'_c	Specified compressive strength of concrete, ksi (MPa)	D1.4b
g	Clear span of coupling beam, in. (mm)	H4.5b
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; and for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, in. (mm)	Table D1.1
h	Distance between horizontal boundary element centerlines, in. (mm)	F5.4a
h	Overall depth of the boundary member in the plane of the wall, in. (mm)	H5.5b
h_{cc}	Cross-sectional dimension of the confined core region in composite columns measured center-to-center of the tie reinforcement, in. (mm)	D1.4b
h_o	Distance between flange centroids, in. (mm)	D1.2c
r	Governing radius of gyration, in. (mm)	E3.4c
r_i	Minimum radius of gyration of individual component, in. (mm)	F2.5b

r_y	Radius of gyration about y-axis, in. (mm)	D1.2a
r_y	Radius of gyration of individual components about their weak axis, in. (mm)	E4.5e
s	Spacing of transverse reinforcement, in. (mm)	D1.4b
t	Thickness of element, in. (mm)	Table D1.1
t	Thickness of column web or individual doubler plate, in. (mm)	E3.6e
t	Thickness of the steel web plate, in. (mm)	H7.4a
t	Thickness of the part subjected to through-thickness strain, in. (mm)	J6.2c
t_{HSS}	Thickness of HSS, in. (mm)	H7.4c
t_{bf}	Thickness of beam flange, in. (mm)	E3.4c
t_{eff}	Effective web-plate thickness, in. (mm)	F5.7a
t_f	Thickness of flange, in. (mm)	D2.5b
t_{lim}	Limiting column flange thickness, in. (mm)	E3.6f
t_p	Thickness of the gusset plate, in. (mm)	F2.6c.4
t_s	Thickness of steel web plate, in. (mm)	H7.4e
t_w	Thickness of web, in. (mm)	F3.5b
t_w	Web-plate thickness, in. (mm)	F5.7a
t_w	Total thickness of wall, in. (mm)	H7.4e
w_{min}	Minimum of w_1 and w_2, in. (mm)	H7.4e
w_1	Maximum spacing of tie bars in vertical and horizontal directions, in. (mm)	H7.4a
w_1	Maximum spacing of tie bars or shear studs in vertical and horizontal directions, in. (mm)	H7.4b
w_1, w_2	Vertical and horizontal spacing of tie bars, respectively, in. (mm)	H7.4e
w_z	Width of panel zone between column flanges, in. (mm)	E3.6e
Δ	Design story drift, in. (mm)	F3.4a
Δ_b	Deformation quantity used to control loading of test specimen (total brace end rotation for the subassemblage test specimen; total brace axial deformation for the brace test specimen), in. (mm)	K3.4b
Δ_{bm}	Value of deformation quantity, Δ_b, at least equal to that corresponding to the design story drift, in. (mm)	K3.4c
Δ_{by}	Value of deformation quantity, Δ_b, at first yield of test specimen, in. (mm)	K3.4c
Ω	Safety factor	B3.2
Ω_c	Safety factor for compression	Table D1.1
Ω_o	System overstrength factor	B2
Ω_v	Safety factor for shear strength of panel zone of beam-to-column connections	E3.6e
α	Angle of diagonal members with the horizontal, degrees..	E4.5c
α	Angle of web yielding, as measured relative to the vertical, degrees	F5.5b
α	Angle of the shortest center-to-center lines in the opening array to vertical, degrees	F5.7a
α_s	LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD	D1.2a
β	Compression strength adjustment factor	F4.2a

β_1	Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, as defined in ACI 318	H4.5b
γ_{total}	Total link rotation angle, rad	K2.4c
θ	Story drift angle, rad	K2.4b
λ_{hd}, λ_{md}	Limiting slenderness parameter for highly and moderately ductile compression elements, respectively	D1.1b
ϕ	Resistance factor	B3.2
ϕ_c	Resistance factor for compression	Table D1.1
ϕ_v	Resistance factor for shear	E3.6e
$\overline{\rho}$	Strength adjusted reinforcement ratio	H7.5b
ω	Strain hardening adjustment factor	F4.2a

GLOSSARY

The terms listed below are to be used in addition to those in the AISC *Specification for Structural Steel Buildings*. Some commonly used terms are repeated here for convenience.

Notes:
(1) Terms designated with † are common AISI-AISC terms that are coordinated between the two standards developers.
(2) Terms designated with * are usually qualified by the type of load effect, for example, nominal tensile strength, available compressive strength, and design flexural strength.

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

Adjusted link shear strength. Link shear strength including the material overstrength and strain hardening.

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

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

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

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

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

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

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

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

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

Buckling-restrained brace. A pre-fabricated, or manufactured, brace element consisting of a steel core and a buckling-restraining system as described in Section F4 and qualified by testing as required in Section K3.

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

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

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

Capacity-limited seismic load. The capacity-limited horizontal seismic load effect, E_{cl}, determined in accordance with these Provisions, substituted for E_{mh}, and applied as prescribed by the load combinations in the applicable building code.

Collector. Also known as drag strut; member that serves to transfer loads between diaphragms and the members of the vertical force-resisting elements of the seismic force-resisting system.

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

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

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

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

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

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

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

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

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

Composite ordinary shear wall (C-OSW). Composite shear wall meeting the requirements of Section H4.

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

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

Composite plate shear wall—concrete filled (C-PSW/CF). Wall consisting of two planar steel web plates with concrete fill between the plates, with or without boundary elements, and meeting the requirements of Section H7.

Composite shear wall. Steel plate wall panel composite with reinforced concrete wall panel or reinforced concrete wall that has steel or concrete-encased structural steel sections as boundary members.

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

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

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

Composite special shear wall (C-SSW). Composite shear wall meeting the requirements of Section H5.

Concrete-encased shapes. Structural steel sections encased in concrete.

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

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

Demand critical weld. Weld so designated by these Provisions.

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

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

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

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

Ductile limit state. Ductile limit states include member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the seismic compactness limitations of Table D1.1. Rupture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.

Eccentrically braced frame (EBF). Diagonally braced frame meeting the requirements of Section F3 that has at least one end of each diagonal brace connected to a beam with a defined eccentricity from another beam-to-brace connection or a beam-to-column connection.

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

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

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

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

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

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

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

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

Filled composite column. HSS filled with structural concrete.

Fully composite beam. Composite beam that has a sufficient number of steel headed stud anchors to develop the nominal plastic flexural strength of the composite section.

Highly ductile member. A member that meets the requirements for highly ductile members in Section D1.

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

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

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

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

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

K-braced frame. A braced-frame configuration in which two or more braces connect to a column at a point other than a beam-to-column or strut-to-column connection.

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

Link intermediate web stiffeners. Vertical web stiffeners placed within the link in EBF.

Link rotation angle. Inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift.

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

Link design shear strength. Lesser of the available shear strength of the link based on the flexural or shear strength of the link member.

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

Lowest anticipated service temperature (LAST). Lowest daily minimum temperature, or other suitable temperature, as established by the engineer of record.

LRFD (load and resistance factor design)†. Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.

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

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

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

Moderately ductile member. A member that meets the requirements for moderately ductile members in Section D1.

Multi-tiered braced frame (MTBF). A braced-frame configuration with two or more levels of bracing between diaphragm levels or locations of out-of-plane bracing.

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

Ordinary cantilever column system (OCCS). A seismic force-resisting system in which the seismic forces are resisted by one or more columns that are cantilevered from the foundation or from the diaphragm level below and that meets the requirements of Section E5.

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

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

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

Overstrength seismic load. The horizontal seismic load effect including overstrength determined using the overstrength factor, Ω_o, and applied as prescribed by the load combinations in the applicable building code.

Partially composite beam. Steel beam with a composite slab with a nominal flexural strength controlled by the strength of the steel headed stud anchors.

Partially restrained composite connection. Partially restrained (PR) connections as defined in the *Specification* that connect partially or fully composite beams to steel columns with flexural resistance provided by a force couple achieved with steel reinforcement in the slab and a steel seat angle or comparable connection at the bottom flange.

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

Power-actuated fastener. Nail-like fastener driven by explosive powder, gas combustion, or compressed air or other gas to embed the fastener into structural steel.

Prequalified connection. Connection that complies with the requirements of Section K1 or ANSI/AISC 358.

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

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

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

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

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

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

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

Risk category. Classification assigned to a structure based on its use as specified by the applicable building code.

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

Seismic design category. A classification assigned to a structure based on its risk category and the severity of the design earthquake ground motion at the site.

Seismic force-resisting system (SFRS). That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed in the applicable building code.

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

Special cantilever column system (SCCS). A seismic force-resisting system in which the seismic forces are resisted by one or more columns that are cantilevered from the foundation or from the diaphragm level below and that meets the requirements of Section E6.

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

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

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

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

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

Steel core. Axial-force-resisting element of a buckling-restrained brace. The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it is permitted to also contain projections beyond the casing and transition segments between the projections and yielding segment.

Story drift angle. Interstory displacement divided by story height.

Strut. A horizontal member in a multi-tiered braced frame interconnecting brace connection points at columns.

Subassemblage test specimen. The combination of members, connections and testing apparatus that replicate as closely as practical the boundary conditions, loading and deformations in the prototype.

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

Test specimen. A member, connection or subassemblage test specimen.

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

V-braced frame. Concentrically braced frame (SCBF, OCBF, BRBF, C-OBF or C-SCBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an inverted-V-braced frame.

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

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

Yield length ratio. In a buckling-restrained brace, the ratio of the length over which the core area is equal to A_{sc}, to the length from intersection points of brace centerline and beam or column centerline at each end.

ABBREVIATIONS

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

ACI (American Concrete Institute)
AISC (American Institute of Steel Construction)
ANSI (American National Standards Institute)
ASCE (American Society of Civil Engineers)
ASD (allowable strength design)
AWS (American Welding Society)
BRBF (buckling-restrained braced frame)
C-EBF (composite eccentrically braced frame)
C-IMF (composite intermediate moment frame)
CJP (complete joint penetration)
C-OBF (composite ordinary braced frame)
C-OMF (composite ordinary moment frame)
C-OSW (composite ordinary shear wall)
C-PRMF (composite partially restrained moment frame)
CPRP (connection prequalification review panel)
C-PSW (composite plate shear wall)
C-SCBF (composite special concentrically braced frame)
C-SMF (composite special moment frame)
C-SSW (composite special shear wall)
CVN (Charpy V-notch)
EBF (eccentrically braced frame)
FCAW (flux cored arc welding)
FEMA (Federal Emergency Management Agency)
FR (fully restrained)
GMAW (gas metal arc welding)
HBE (horizontal boundary element)
HSS (hollow structural section)
IBE (intermediate boundary element)
IMF (intermediate moment frame)
LAST (lowest anticipated service temperature)
LRFD (load and resistance factor design)
MT (magnetic particle testing)
MT-OCBF (multi-tiered ordinary concentrically braced frame)
MT-SCBF (multi-tiered special concentrically braced frame)
MT-BRBF (multi-tiered buckling-restrained braced frame)
NDT (nondestructive testing)
OCBF (ordinary concentrically braced frame)
OCCS (ordinary cantilever column system)

OMF (ordinary moment frame)
OVS (oversized)
PJP (partial joint penetration)
PR (partially restrained)
QA (quality assurance)
QC (quality control)
RBS (reduced beam section)
RCSC (Research Council on Structural Connections)
SCBF (special concentrically braced frame)
SCCS (special cantilever column system)
SDC (seismic design category)
SEI (Structural Engineering Institute)
SFRS (seismic force-resisting system)
SMAW (shielded metal arc welding)
SMF (special moment frame)
SPSPW (special perforated steel plate wall)
SPSW (special plate shear wall)
SRC (steel-reinforced concrete)
STMF (special truss moment frame)
UT (ultrasonic testing)
VBE (vertical boundary element)
WPQR (welder performance qualification records)
WPS (welding procedure specification)

CHAPTER A
GENERAL REQUIREMENTS

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

The chapter is organized as follows:

A1. Scope
A2. Referenced Specifications, Codes and Standards
A3. Materials
A4. Structural Design Drawings and Specifications

A1. SCOPE

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

Wherever these Provisions refer to the applicable building code and there is none, the loads, load combinations, system limitations, and general design requirements shall be those in ASCE/SEI 7.

User Note: ASCE/SEI 7 (Table 12.2-1, Item H) specifically exempts structural steel systems in seismic design categories B and C from the requirements in these Provisions if they are designed in accordance with the AISC *Specification for Structural Steel Buildings* and the seismic loads are computed using a seismic response modification coefficient, R, of 3; composite systems are not covered by this exemption. These Provisions do not apply in seismic design category A.

User Note: ASCE/SEI 7 (Table 15.4-1) permits certain nonbuilding structures to be designed in accordance with the AISC *Specification for Structural Steel Buildings* in lieu of the Provisions with an appropriately reduced R factor.

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

These Provisions shall be applied in conjunction with the AISC *Specification for Structural Steel Buildings*, hereafter referred to as the *Specification*. All requirements of the *Specification* are applicable unless otherwise stated in these Provisions. Members and connections of the SFRS shall satisfy the requirements of the applicable building code, the *Specification*, and these Provisions. The phrases "is permitted" and "are permitted" in these Provisions identify provisions that comply with the *Specification*, but are not mandatory.

In these Provisions, *Building Code Requirements for Structural Concrete* (ACI 318) and the *Metric Building Code Requirements for Structural Concrete and Commentary* (ACI 318M) are referred to collectively as ACI 318. ACI 318, as modified in these Provisions, shall be used for the design and construction of reinforced concrete components in composite construction. For the SFRS in composite construction incorporating reinforced concrete components designed in accordance with ACI 318, the requirements of *Specification* Section B3.1, Design for Strength Using Load and Resistance Factor Design, shall be used.

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

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

(a) American Institute of Steel Construction (AISC)
ANSI/AISC 360-16 *Specification for Structural Steel Buildings*
ANSI/AISC 358-16 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*

(b) American Welding Society (AWS)
AWS D1.8/D1.8M:2016 *Structural Welding Code—Seismic Supplement*
AWS B4.0:2007 *Standard Methods for Mechanical Testing of Welds* (U.S. Customary Units)
AWS B4.0M:2000 *Standard Methods for Mechanical Testing of Welds* (Metric Customary Units)
AWS D1.4/D1.4M:2011 *Structural Welding Code—Reinforcing Steel*

(c) ASTM International (ASTM)
ASTM C31/C31M-15 *Standard Practice for Making and Curing Concrete Test Specimens in the Field*
ASTM C39/C39M-16 *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*
ASTM E8/E8M-15 *Standard Test Methods for Tension Testing of Metallic Materials*

A3. MATERIALS

1. Material Specifications

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

Exception: Specified minimum yield stress of structural steel shall not exceed 70 ksi (485 MPa) for columns in systems defined in Sections E3, E4, G3, H1, H2 and H3 and for columns in all systems in Chapter F.

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

(a) Hot-rolled structural shapes
ASTM A36/A36M
ASTM A529/A529M
ASTM A572/A572M [Grade 42 (290), 50 (345) or 55 (380)]
ASTM A588/A588M
ASTM A913/A913M [Grade 50 (345), 60 (415), 65 (450) or 70 (485)]
ASTM A992/A992M

(b) Hollow structural sections (HSS)
ASTM A500/A500M (Grade B or C)
ASTM A501/A501M
ASTM A1085/A1085M
ASTM A53/A53M

(c) Plates
ASTM A36/A36M
ASTM A529/A529M
ASTM A572/A572M [Grade 42 (290), 50 (345) or 55 (380)]
ASTM A588/A588M
ASTM A1011/A1011M HSLAS Grade 55 (380)
ASTM A1043/A1043M

(d) Bars
ASTM A36/A36M
ASTM A529/A529M
ASTM A572/A572M [Grade 42 (290), 50 (345) or 55 (380)]
ASTM A588/A588M

(e) Sheets
ASTM A1011/A1011M HSLAS Gr. 55 (380)

The structural steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283/A283M Grade D. Other steels and nonsteel materials in buckling-restrained braced frames are permitted to be used subject to the requirements of Sections F4 and K3.

User Note: This section only covers material properties for structural steel used in the SFRS and included in the definition of structural steel given in Section 2.1 of the AISC *Code of Standard Practice*. Other steel, such as cables for permanent bracing, is not covered. Steel reinforcement used in components in composite SFRS is covered in Section A3.5.

2. Expected Material Strength

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

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

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

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

User Note: The expected compressive strength of concrete may be estimated using values from *Seismic Rehabilitation of Existing Buildings* (ASCE/SEI 41-13).

TABLE A3.1
R_y and R_t Values for Steel and Steel Reinforcement Materials

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M	1.5	1.2
• ASTM A1043/A1043M Gr. 36 (250)	1.3	1.1
• ASTM A992/A992M	1.1	1.1
• ASTM A572/A572M Gr. 50 (345) or 55 (380)	1.1	1.1
• ASTM A913/A913M Gr. 50 (345), 60 (415), 65 (450), or 70 (485)	1.1	1.1
• ASTM A588/A588M	1.1	1.1
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
• ASTM A529 Gr. 50 (345)	1.2	1.2
• ASTM A529 Gr. 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
• ASTM A500/A500M Gr. B	1.4	1.3
• ASTM A500/A500M Gr. C	1.3	1.2
• ASTM A501/A501M	1.4	1.3
• ASTM A53/A53M	1.6	1.2
• ASTM A1085/A1085M	1.25	1.15
Plates, Strips and Sheets:		
• ASTM A36/A36M	1.3	1.2
• ASTM A1043/A1043M Gr. 36 (250)	1.3	1.1
• ASTM A1011/A1011M HSLAS Gr. 55 (380)	1.1	1.1
• ASTM A572/A572M Gr. 42 (290)	1.3	1.0
• ASTM A572/A572M Gr. 50 (345), Gr. 55 (380)	1.1	1.2
• ASTM A588/A588M	1.1	1.2
• ASTM A1043/A1043M Gr. 50 (345)	1.2	1.1
Steel Reinforcement:		
• ASTM A615/A615M Gr. 60 (420)	1.2	1.2
• ASTM A615/A615M Gr. 75 (520) and Gr. 80 (550)	1.1	1.2
• ASTM A706/A706M Gr. 60 (420) and Gr. 80 (550)	1.2	1.2

3. Heavy Sections

For structural steel in the SFRS, in addition to the requirements of *Specification* Section A3.1c, hot rolled shapes with flange thickness equal to or greater than 1½ in. (38 mm) shall have a minimum Charpy V-notch (CVN) toughness of 20 ft-lb (27 J) at 70°F (21°C), tested in the alternate core location as described in ASTM A6 Supplementary Requirement S30. Plates with thickness equal to or greater than 2 in. (50 mm) shall have a minimum Charpy V-notch toughness of 20 ft-lb (27 J) at 70°F

(21°C), measured at any location permitted by ASTM A673, Frequency P, where the plate is used for the following:

(a) Members built up from plate
(b) Connection plates where inelastic strain under seismic loading is expected
(c) The steel core of buckling-restrained braces

4. Consumables for Welding

4a. Seismic Force-Resisting System Welds

All welds used in members and connections in the SFRS shall be made with filler metals meeting the requirements specified in clauses 6.1, 6.2 and 6.3 of *Structural Welding Code—Seismic Supplement* (AWS D1.8/D1.8M), hereafter referred to as AWS D1.8/D1.8M.

User Note: AWS D1.8/D1.8M clauses 6.2.1, 6.2.2, 6.2.3, and 6.3.1 apply only to demand critical welds.

4b. Demand Critical Welds

Welds designated as demand critical shall be made with filler metals meeting the requirements specified in AWS D1.8/D1.8M clauses 6.1, 6.2 and 6.3.

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

Filler Metal Classification Properties for Seismic Force-Resisting System Welds

Property	Classification		
	70 ksi (480 MPa)	80 ksi (550 MPa)	90 ksi (620 MPa)
Yield Strength, ksi (MPa)	58 (400) min.	68 (470) min.	78 (540) min.
Tensile Strength, ksi (MPa)	70 (480) min.	80 (550) min.	90 (620) min.
Elongation, %	22 min.	19 min.	17 min.
CVN Toughness, ft-lb (J)[a]	20 (27) min. @ 0°F (–18°C)[a]		25 (34) min. @ –20°F (–30°C)

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

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

Mechanical Properties for Demand Critical Welds			
	Classification		
Property	70 ksi (480 MPa)	80 ksi (550 MPa)	90 ksi (620 MPa)
Yield Strength, ksi (MPa)	58 (400) min.	68 (470) min.	78 (540) min.
Tensile Strength, ksi (MPa)	70 (480) min.	80 (550) min.	90 (620) min.
Elongation (%)	22 min.	19 min.	17 min.
CVN Toughness, ft-lb (J)[b, c]	40 (54) min. @ 70°F (20°C)		40 (54) min. @ 50°F (10°C)

[b] For LAST of +50°F (+10°C). For LAST less than +50°F (+10°C), see AWS D1.8/D1.8M clause 6.2.2.

[c] Tests conducted in accordance with AWS D1.8/D1.8M Annex A meeting 40 ft-lb (54 J) min. at a temperature lower than +70°F (+20°C) also meet this requirement.

5. Concrete and Steel Reinforcement

Concrete and steel reinforcement used in composite components in composite intermediate or special SFRS of Sections G2, G3, G4, H2, H3, H5, H6 and H7 shall satisfy the requirements of ACI 318 Chapter 18. Concrete and steel reinforcement used in composite components in composite ordinary SFRS of Sections G1, H1 and H4 shall satisfy the requirements of ACI 318 Section 18.2.1.4.

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

1. General

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

(a) Designation of the SFRS
(b) Identification of the members and connections that are part of the SFRS
(c) Locations and dimensions of protected zones
(d) Connection details between concrete floor diaphragms and the structural steel elements of the SFRS
(e) Shop drawing and erection drawing requirements not addressed in Section I1

User Note: The *Code of Standard Practice* uses the term "design documents" in place of "design drawings" to generalize the term and to reflect both paper drawings and electronic models. Similarly, "fabrication documents" is used in place of "shop drawings," and "erection documents" is used in place of "erection drawings". The use of "drawings" in this standard is not intended to create a conflict.

2. Steel Construction

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

(a) Configuration of the connections
(b) Connection material specifications and sizes
(c) Locations of demand critical welds
(d) Locations where gusset plates are to be detailed to accommodate inelastic rotation
(e) Locations of connection plates requiring Charpy V-notch toughness in accordance with Section A3.3(b)
(f) Lowest anticipated service temperature of the steel structure, if the structure is not enclosed and maintained at a temperature of 50°F (10°C) or higher
(g) Locations where weld backing is required to be removed
(h) Locations where fillet welds are required when weld backing is permitted to remain
(i) Locations where fillet welds are required to reinforce groove welds or to improve connection geometry
(j) Locations where weld tabs are required to be removed
(k) Splice locations where tapered transitions are required
(l) The shape of weld access holes, if a shape other than those provided for in the *Specification* is required
(m) Joints or groups of joints in which a specific assembly order, welding sequence, welding technique, or other special precautions are required, where such items are designated to be submitted to the engineer of record

3. Composite Construction

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

(a) Bar placement, cutoffs, lap and mechanical splices, hooks and mechanical anchorage, placement of ties, and other transverse reinforcement
(b) Requirements for dimensional changes resulting from temperature changes, creep and shrinkage
(c) Location, magnitude and sequencing of any prestressing or post-tensioning present
(d) Location of steel headed stud anchors and welded reinforcing bar anchors

CHAPTER B
GENERAL DESIGN REQUIREMENTS

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

This chapter is organized as follows:

B1. General Seismic Design Requirements
B2. Loads and Load Combinations
B3. Design Basis
B4. System Type
B5. Diaphragms, Chords and Collectors

B1. GENERAL SEISMIC DESIGN REQUIREMENTS

The required strength and other seismic design requirements for seismic design categories, risk categories, and the limitations on height and irregularity shall be as specified in the applicable building code.

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

B2. LOADS AND LOAD COMBINATIONS

Where the required strength defined in these Provisions refers to the capacity-limited seismic load, the capacity-limited horizontal seismic load effect, E_{cl}, shall be determined in accordance with these Provisions, substituted for E_{mh}, and applied as prescribed by the load combinations in the applicable building code.

Where the required strength defined in these Provisions refers to the overstrength seismic load, the horizontal seismic load effect including overstrength, E_{mh}, shall be determined using the overstrength factor, Ω_o, and applied as prescribed by the load combinations in the applicable building code. Where the required strength refers to the overstrength seismic load, it is permitted to use the capacity-limited seismic load instead.

User Note: The seismic load effect including overstrength is defined in ASCE/SEI 7 Section 12.4.3. In ASCE/SEI 7 Section 12.4.3.1, the horizontal seismic load effect, E_{mh}, is determined using Equation 12.4-7: $E_{mh} = \Omega_o Q_E$. There is a cap on the value of E_{mh}: it need not be taken larger than E_{cl}. Thus, in effect, where these Provisions refer to overstrength seismic load, E_{mh} is permitted to be based upon the overstrength factor, Ω_o, or E_{cl}. However, where capacity-limited seismic load is required, it is intended that E_{cl} replace E_{mh} as specified in ASCE/SEI 7 Section 12.4.3.2 and use of ASCE/SEI 7 Equation 12.4-7 is not permitted.

In composite construction, incorporating reinforced concrete components designed in accordance with the requirements of ACI 318, the requirements of *Specification* Section B3.1, Design for Strength Using Load and Resistance Factor Design, shall be used for the seismic force-resisting system (SFRS).

B3. DESIGN BASIS

1. Required Strength

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

(a) The required strength as determined by structural analysis for the applicable load combinations, as stipulated in the applicable building code, and in Chapter C
(b) The required strength given in Chapters D, E, F, G and H

2. Available Strength

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

B4. SYSTEM TYPE

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

B5. DIAPHRAGMS, CHORDS AND COLLECTORS

1. General

Diaphragms and chords shall be designed for the loads and load combinations in the applicable building code. Collectors shall be designed for the load combinations in the applicable building code, including overstrength.

2. Truss Diaphragms

When a truss is used as a diaphragm, all members of the truss and their connections shall be designed for forces calculated using the load combinations of the applicable building code, including overstrength.

Exceptions:

(a) The forces specified in this section need not be applied to the diagonal members of the truss diaphragms and their connections, where these members and connections conform to the requirements of Sections F2.4a, F2.5a, F2.5b and F2.6c. Braces in K- or V- configurations and braces supporting gravity loads other than self-weight are not permitted under this exception.

User Note: Chords in truss diaphragms serve a function analogous to columns in vertical special concentrically braced frames, and should meet the requirements for highly ductile members as required for columns in Section F2.5a.

(b) The forces specified in this section need not be applied to truss diaphragms designed as a part of a three-dimensional system in which the seismic force-resisting system types consist of ordinary moment frames, ordinary concentrically braced frames, or combinations thereof, and truss diagonal members conform to Sections F1.4b and F1.5 and connections conform to Section F1.6.

CHAPTER C
ANALYSIS

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

C1. General Requirements
C2. Additional Requirements
C3. Nonlinear Analysis

C1. GENERAL REQUIREMENTS

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

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

C2. ADDITIONAL REQUIREMENTS

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

C3. NONLINEAR ANALYSIS

When nonlinear analysis is used to satisfy the requirements of these Provisions, it shall be performed in accordance with the applicable building code.

User Note: ASCE/SEI 7 permits nonlinear analysis by a response history procedure. Material and geometric nonlinearities are to be included in the analytical model. The main purpose is to determine expected member inelastic deformations and story drifts under representative ground motions. The analysis results also provide values of maximum expected internal forces at locations such as column splices, which can be used as upper limits on required strength for design.

CHAPTER D

GENERAL MEMBER AND CONNECTION DESIGN REQUIREMENTS

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

The chapter is organized as follows:

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

D1. MEMBER REQUIREMENTS

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

1. Classification of Sections for Ductility

When required for the systems defined in Chapters E, F, G, H and Section D4, members designated as moderately ductile members or highly ductile members shall comply with this section.

1a. Section Requirements for Ductile Members

Structural steel sections for both moderately ductile members and highly ductile members shall have flanges continuously connected to the web or webs.

Encased composite columns shall comply with the requirements of Section D1.4b.1 for moderately ductile members and Section D1.4b.2 for highly ductile members.

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

Concrete sections shall comply with the requirements of ACI 318 Section 18.4 for moderately ductile members and ACI 318 Section 18.6 and 18.7 for highly ductile members.

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

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

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

TABLE D1.1
Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members

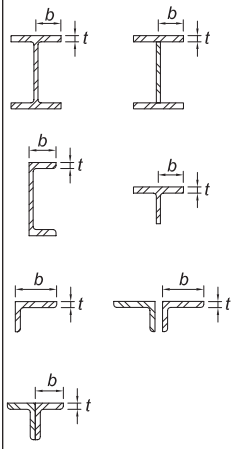
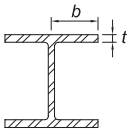
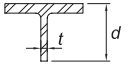
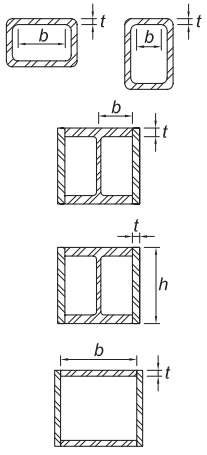
	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Example
			λ_{hd} Highly Ductile Members	λ_{md} Moderately Ductile Members	
Unstiffened Elements	Flanges of rolled or built-up I-shaped sections, channels and tees; legs of single angles or double-angle members with separators; outstanding legs of pairs of angles in continuous contact	b/t	$0.32\sqrt{\frac{E}{R_yF_y}}$	$0.40\sqrt{\frac{E}{R_yF_y}}$	
	Flanges of H-pile sections per Section D4	b/t	not applicable	$0.48\sqrt{\frac{E}{R_yF_y}}$	
	Stems of tees	d/t	$0.32\sqrt{\frac{E}{R_yF_y}}$ [a]	$0.40\sqrt{\frac{E}{R_yF_y}}$	
Stiffened Elements	Walls of rectangular HSS used as diagonal braces	b/t	$0.65\sqrt{\frac{E}{R_yF_y}}$	$0.76\sqrt{\frac{E}{R_yF_y}}$	
	Flanges of boxed I-shaped sections	b/t			
	Side plates of boxed I-shaped sections and walls of built-up box shapes used as diagonal braces	h/t			
	Flanges of built-up box shapes used as link beams	b/t			

TABLE D1.1 (continued)
Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members

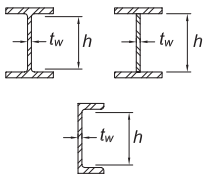
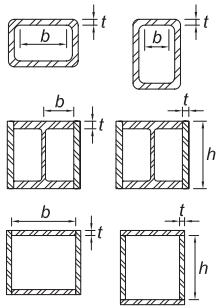
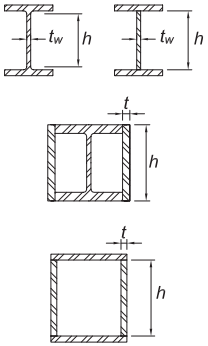
	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio: λ_{md} Moderately Ductile Members	Example
Stiffened Elements	Webs of rolled or built-up I shaped sections and channels used as diagonal braces	h/t_w	$1.57\sqrt{\frac{E}{R_yF_y}}$	$1.57\sqrt{\frac{E}{R_yF_y}}$	
	Where used in beams or columns as flanges in uniform compression due to axial, flexure, or combined axial and flexure: 1) Walls of rectangular HSS 2) Flanges and side plates of boxed I-shaped sections, webs and flanges of built-up box shapes	b/t h/t	$0.65\sqrt{\frac{E}{R_yF_y}}$	$1.18\sqrt{\frac{E}{R_yF_y}}$	
	Where used in beams, columns, or links, as webs in flexure, or combined axial and flexure: 1) Webs of rolled or built-up I-shaped sections or channels [b] 2) Side plates of boxed I-shaped sections 3) Webs of built-up box sections	h/t_w h/t h/t	For $C_a \le 0.114$ $2.57\sqrt{\frac{E}{R_yF_y}}(1-1.04C_a)$ For $C_a > 0.114$ $0.88\sqrt{\frac{E}{R_yF_y}}(2.68-C_a)$ $\ge 1.57\sqrt{\frac{E}{R_yF_y}}$ where $C_a = \frac{P_u}{\phi_c P_y}$ (LRFD) $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD) $P_y = R_yF_yA_g$	For $C_a \le 0.114$ $3.96\sqrt{\frac{E}{R_yF_y}}(1-3.04C_a)$ For $C_a > 0.114$ $1.29\sqrt{\frac{E}{R_yF_y}}(2.12-C_a)$ $\ge 1.57\sqrt{\frac{E}{R_yF_y}}$ where $C_a = \frac{P_u}{\phi_c P_y}$ (LRFD) $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD) $P_y = R_yF_yA_g$	

TABLE D1.1 (continued)
Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members

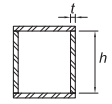
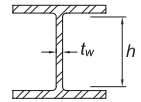
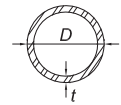
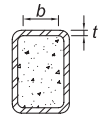
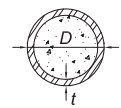
	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Example
			λ_{hd} Highly Ductile Members	λ_{md} Moderately Ductile Members	
Stiffened Elements	Webs of built-up box sections used as EBF links	h/t	$0.67\sqrt{\frac{E}{R_yF_y}}$	$1.75\sqrt{\frac{E}{R_yF_y}}$	
	Webs of H-Pile sections	h/t_w	not applicable	$1.57\sqrt{\frac{E}{R_yF_y}}$	
	Walls of round HSS	D/t	$0.053\frac{E}{R_yF_y}$	$0.062\frac{E}{R_yF_y}$ [c]	
Composite	Walls of rectangular filled composite members	b/t	$1.48\sqrt{\frac{E}{R_yF_y}}$	$2.37\sqrt{\frac{E}{R_yF_y}}$	
	Walls of round filled composite members	D/t	$0.085\frac{E}{R_yF_y}$	$0.17\frac{E}{R_yF_y}$	

TABLE D1.1 (continued)
Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members

[a] For tee-shaped compression members, the limiting width-to-thickness ratio for highly ductile members for the stem of the tee shall be $0.40\sqrt{\frac{E}{R_yF_y}}$ where either of the following conditions are satisfied:

(1) Buckling of the compression member occurs about the plane of the stem.

(2) The axial compression load is transferred at end connections to only the outside face of the flange of the tee resulting in an eccentric connection that reduces the compression stresses at the tip of the stem.

[b] For I-shaped beams in SMF systems, where C_a is less than or equal to 0.114, the limiting ratio h/t_w shall not exceed $2.57\sqrt{\frac{E}{R_yF_y}}$. For I-shaped beams in intermediate moment frame (IMF) systems, where C_a is less than or equal to 0.114, the limiting width-to-thickness ratio shall not exceed $3.96\sqrt{\frac{E}{R_yF_y}}$.

[c] The limiting diameter-to-thickness ratio of round HSS members used as beams or columns shall not exceed $0.077\frac{E}{R_yF_y}$,

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_y = specified minimum yield stress, ksi (MPa)
P_a = required axial strength using ASD load combinations, kips (N)
P_u = required axial strength using LRFD load combinations, kips (N)
R_y = ratio of the expected yield stress to the specified minimum yield stress
ϕ_c = resistance factor for compression
Ω_c = safety factor for compression

2. Stability Bracing of Beams

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

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

2a. Moderately Ductile Members

1. Steel Beams

The bracing of moderately ductile steel beams shall satisfy the following requirements:

(a) Both flanges of beams shall be laterally braced or the beam cross section shall be braced with point torsional bracing.

(b) Beam bracing shall meet the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams, where C_d is 1.0 and the required flexural strength of the member shall be:

$$M_r = R_y F_y Z/\alpha_s \quad \text{(D1-1)}$$

where

R_y = ratio of the expected yield stress to the specified minimum yield stress
Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)
α_s = LRFD-ASD force level adjustment factor
= 1.0 for LRFD and 1.5 for ASD

(c) Beam bracing shall have a maximum spacing of

$$L_b = 0.19 r_y E/(R_y F_y) \quad \text{(D1-2)}$$

where

r_y = radius of gyration about y-axis, in. (mm)

2. Concrete-Encased Composite Beams

The bracing of moderately ductile concrete-encased composite beams shall satisfy the following requirements:

(a) Both flanges of members shall be laterally braced or the beam cross section shall be braced with point torsional bracing.

(b) Lateral bracing shall meet the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams, where $M_r = M_{p,exp}$ of the beam as specified in Section G2.6d, and $C_d = 1.0$.

(c) Member bracing shall have a maximum spacing of

$$L_b = 0.19 r_y E/(R_y F_y) \quad \text{(D1-3)}$$

using the material properties of the steel section and r_y in the plane of buckling calculated based on the elastic transformed section.

2b. Highly Ductile Members

In addition to the requirements of Sections D1.2a.1(a) and (b), and D1.2a.2(a) and (b), the bracing of highly ductile beam members shall have a maximum spacing of $L_b = 0.095 r_y E/(R_y F_y)$. For concrete-encased composite beams, the material properties of the steel section shall be used and the calculation for r_y in the plane of buckling shall be based on the elastic transformed section.

2c. Special Bracing at Plastic Hinge Locations

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

1. Steel Beams

For structural steel beams, such bracing shall satisfy the following requirements:

(a) Both flanges of beams shall be laterally braced or the member cross section shall be braced with point torsional bracing.

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

$$P_r = 0.06 R_y F_y Z / (\alpha_s h_o) \quad \text{(D1-4)}$$

where

h_o = distance between flange centroids, in. (mm)

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

$$M_r = 0.06 R_y F_y Z / \alpha_s \quad \text{(D1-5)}$$

(c) The required bracing stiffness shall satisfy the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams with $C_d = 1.0$ and where the required flexural strength of the beam shall be taken as:

$$M_r = R_y F_y Z / \alpha_s \quad \text{(D1-6)}$$

2. Concrete-Encased Composite Beams

For concrete-encased composite beams, such bracing shall satisfy the following requirements:

(a) Both flanges of beams shall be laterally braced or the beam cross section shall be braced with point torsional bracing.

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

$$P_u = 0.06 M_{p,exp} / h_o \quad \text{(D1-7)}$$

of the beam, where

$M_{p,exp}$ = expected flexural strength of the steel, concrete-encased or composite beam, kip-in. (N-mm), determined in accordance with Section G2.6d.

The required strength for torsional bracing provided adjacent to plastic hinges shall be $M_u = 0.06 M_{p,exp}$ of the beam.

(c) The required bracing stiffness shall satisfy the requirements of Appendix 6 of the *Specification* for lateral or torsional bracing of beams, where $M_r = M_u = M_{p,exp}$ of the beam is determined in accordance with Section G2.6d, and $C_d = 1.0$.

3. Protected Zones

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

Exception: Welded steel headed stud anchors and other connections are permitted in protected zones when designated in ANSI/AISC 358, or as otherwise determined with a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Sections K2 and K3.

4. Columns

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

4a. Required Strength

The required strength of columns in the SFRS shall be determined from the greater effect of the following:

(a) The load effect resulting from the analysis requirements for the applicable system per Chapters E, F, G and H.
(b) The compressive axial strength and tensile strength as determined using the overstrength seismic load. It is permitted to neglect applied moments in this determination unless the moment results from a load applied to the column between points of lateral support.

For columns that are common to intersecting frames, determination of the required axial strength, including the overstrength seismic load or the capacity-limited seismic load, as applicable, shall consider the potential for simultaneous inelasticity from all such frames. The direction of application of the load in each such frame shall be selected to produce the most severe load effect on the column.

Exceptions:

(a) It is permitted to limit the required axial strength for such columns based on a three-dimensional nonlinear analysis in which ground motion is simultaneously applied in two orthogonal directions, in accordance with Section C3.
(b) Columns common to intersecting frames that are part of Sections E1, F1, G1, H1, H4 or combinations thereof need not be designed for these loads.

4b. Encased Composite Columns

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

1. Moderately Ductile Members

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

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

(1) One-half the least dimension of the section

(2) 8 longitudinal bar diameters

(3) 24 tie bar diameters

(4) 12 in. (300 mm)

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

(1) One-sixth the vertical clear height of the column

(2) Maximum cross-sectional dimension

(3) 18 in. (450 mm)

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

(d) Splices and end bearing details for encased composite columns in composite ordinary SFRS of Sections G1, H1 and H4 shall satisfy the requirements of the *Specification* and ACI 318 Section 10.7.5.3. The design shall comply with ACI 318 Sections 18.2.7 and 18.2.8. The design shall consider any adverse behavioral effects due to abrupt changes in either the member stiffness or the nominal tensile strength. Transitions to reinforced concrete sections without embedded structural steel members, transitions to bare structural steel sections, and column bases shall be considered abrupt changes.

(e) Welded wire fabric shall be prohibited as transverse reinforcement.

2. Highly Ductile Members

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

(a) Longitudinal load-carrying reinforcement shall satisfy the requirements of ACI 318 Section 18.7.4.

(b) Transverse reinforcement shall be hoop reinforcement as defined in ACI 318 Chapter 18 and shall satisfy the following requirements:

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

$$A_{sh} = 0.09 h_{cc} s \left(1 - \frac{F_y A_s}{P_n} \right) \left(\frac{f_c'}{F_{ysr}} \right) \qquad \text{(D1-8)}$$

where

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

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

D = dead load due to the weight of the structural elements and permanent features on the building, kips (N)
L = live load due to occupancy and moveable equipment, kips (N)

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

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

(c) Encased composite columns in braced frames with required compressive strengths greater than $0.2P_n$, not including the overstrength seismic load, shall have transverse reinforcement as specified in Section D1.4b.2(b)(3) over the total element length. This requirement need not be satisfied if the nominal strength of the concrete-encased steel section alone is greater than the load effect from a load combination of $1.0D + 0.5L$.

(d) Composite columns supporting reactions from discontinued stiff members, such as walls or braced frames, shall have transverse reinforcement as specified in Section D1.4b.2(b)(3) over the full length beneath the level at which the discontinuity occurs if the required compressive strength exceeds $0.1P_n$, not including the overstrength seismic load. Transverse reinforcement shall extend into the discontinued member for at least the length required to develop full yielding in the concrete-encased steel section and longitudinal reinforcement. This requirement need not be satisfied if the nominal

strength of the concrete-encased steel section alone is greater than the load effect from a load combination of $1.0D + 0.5L$.

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

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

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

(3) The required shear strength of the column shall satisfy the requirements of ACI 318 Section 18.7.6.1.1.

(f) When the column terminates on a footing or mat foundation, the transverse reinforcement as specified in this section shall extend into the footing or mat at least 12 in. (300 mm). When the column terminates on a wall, the transverse reinforcement shall extend into the wall for at least the length required to develop full yielding in the concrete-encased shape and longitudinal reinforcement.

4c. Filled Composite Columns

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

5. Composite Slab Diaphragms

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

5a. Load Transfer

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

5b. Nominal Shear Strength

The nominal in-plane shear strength of composite diaphragms and concrete slab on steel deck diaphragms shall be taken as the nominal shear strength of the reinforced concrete above the top of the steel deck ribs in accordance with ACI 318 excluding Chapter 14. Alternatively, the composite diaphragm nominal shear strength shall be determined by in-plane shear tests of concrete-filled diaphragms.

6. Built-Up Structural Steel Members

This section addresses connections between components of built-up members where specific requirements are not provided in the system chapters of these Provisions or in ANSI/AISC 358.

Connections between components of built-up members subject to inelastic behavior shall be designed for the expected forces arising from that inelastic behavior.

Connections between components of built-up members where inelastic behavior is not expected shall be designed for the load effect including the overstrength seismic load.

Where connections between elements of a built-up member are required in a protected zone, the connections shall have an available tensile strength equal to $R_y F_y t_p/\alpha_s$ of the weaker element for the length of the protected zone.

Built-up members may be used in connections requiring testing in accordance with the Provisions provided they are accepted by ANSI/AISC 358 for use in a prequalified joint or have been verified in a qualification test.

D2. CONNECTIONS

1. General

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

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

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

2. Bolted Joints

Bolted joints shall satisfy the following requirements:

(a) The available shear strength of bolted joints using standard holes or short-slotted holes perpendicular to the applied load shall be calculated as that for bearing-type joints in accordance with *Specification* Sections J3.6 and J3.10. The nominal bolt bearing and tearout equations per Section J3.10 of the *Specification* where deformation at the bolt hole at service load is a design consideration shall be used.

Exception: Where the required strength of a connection is based upon the expected strength of a member or element, it is permitted to use the bolt bearing and tearout equations in accordance with *Specification* Section J3.10 where deformation is not a design consideration.

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

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

(c) Bolt holes shall be standard holes or short-slotted holes perpendicular to the applied load in bolted joints where the seismic load effects are transferred by shear in the bolts. Oversized holes or short-slotted holes are permitted in connections where the seismic load effects are transferred by tension in the bolts but not by shear in the bolts.

Exception:

(1) For diagonal braces, oversized holes are permitted in one connection ply only when the connection is designed as a slip-critical joint.

(2) Alternative hole types are permitted if designated in ANSI/AISC 358, or if otherwise determined in a connection prequalification in accordance with Section K1, or if determined in a program of qualification testing in accordance with Section K2 or Section K3.

User Note: Diagonal brace connections with oversized holes must also satisfy other limit states including bolt bearing and bolt shear for the required strength of the connection as defined in Sections F1, F2, F3 and F4.

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

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

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

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

3. Welded Joints

Welded joints shall be designed in accordance with *Specification* Chapter J.

4. Continuity Plates and Stiffeners

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

5. Column Splices

5a. Location of Splices

For all building columns, including those not designated as part of the SFRS, column splices shall be located 4 ft (1.2 m) or more away from the beam-to-column flange connections.

Exceptions:

(a) When the column clear height between beam-to-column flange connections is less than 8 ft (2.4 m), splices shall be at half the clear height

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

(c) Splices in composite columns

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

5b. Required Strength

(1) The required strength of column splices in the SFRS shall be the greater of:

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

(b) The required strength determined using the overstrength seismic load.

(2) In addition, welded column splices in which any portion of the column is subject to a calculated net tensile load effect determined using the overstrength seismic load shall satisfy all of the following requirements:

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

Exception: Partial-joint-penetration (PJP) groove welds are excluded from this requirement if the Exceptions in Sections E2.6g, E3.6g or E4.6c are invoked.

(b) The available strength for each flange splice shall be at least equal to $0.5R_yF_yb_ft_f/\alpha_s$,
where
F_y = specified minimum yield stress, ksi (MPa)
R_y = ratio of expected yield stress to the specified minimum yield stress, F_y
b_f = width of flange, in. (mm) of the smaller column connected
t_f = thickness of flange, in. (mm) of the smaller column connected

(c) Where butt joints in column splices are made with complete-joint-penetration groove welds and when tension stress at any location in the smaller flange exceeds $0.30F_y/\alpha_s$, tapered transitions are required between flanges of unequal thickness or width. Such transitions shall be in accordance with AWS D1.8/D1.8M clause 4.2.

5c. Required Shear Strength

For all building columns, including those not designated as part of the SFRS, the required shear strength of column splices with respect to both orthogonal axes of the column shall be $M_{pc}/(\alpha_s H)$, where M_{pc} is the lesser plastic flexural strength of the column sections for the direction in question, and H is the height of the story, which is permitted to be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below.

The required shear strength of splices of columns in the SFRS shall be the greater of the foregoing requirement or the required shear strength determined per Section D2.5b(1).

5d. Structural Steel Splice Configurations

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

Splice plates or channels used for making web splices in SFRS columns shall be placed on both sides of the column web.

For welded butt-joint splices made with groove welds, weld tabs shall be removed in accordance with AWS D1.8/D1.8M clause 6.16. Steel backing of groove welds need not be removed.

5e. Splices in Encased Composite Columns

For encased composite columns, column splices shall conform to Section D1.4b and ACI 318 Section 18.7.4.2.

6. Column Bases

The required strength of column bases, including those that are not designated as part of the SFRS, shall be determined in accordance with this section.

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

Where columns are welded to base plates with groove welds, weld tabs and weld backing shall be removed, except that weld backing located on the inside of flanges and weld backing on the web of I-shaped sections need not be removed if backing is attached to the column base plate with a continuous 5⁄16-in. (8 mm) fillet weld. Fillet

welds of backing to the inside of column flanges are prohibited. Weld backing located on the inside of HSS and box-section columns need not be removed.

The available strength of concrete elements and reinforcing steel at the column base shall be in accordance with ACI 318. When the design of anchor rods assumes that the ductility demand is provided for by deformations in the anchor rods and anchorage into reinforced concrete, the design shall meet the requirements of ACI 318 Chapter 17. Alternatively, when the ductility demand is provided for elsewhere, the anchor rods and anchorage into reinforced concrete are permitted to be designed for the maximum loads resulting from the deformations occurring elsewhere, including the effects of material overstrength and strain hardening.

User Note: When using concrete reinforcing steel as part of the anchorage embedment design, it is important to consider the anchor failure modes and provide reinforcement that is developed on both sides of the expected failure surface. See ACI 318 Chapter 17, including Commentary.

6a. Required Axial Strength

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

(a) The column axial load calculated using the overstrength seismic load

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

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

6b. Required Shear Strength

The required shear strength of column bases, including those not designated as part of the SFRS, and their attachments to the foundations, shall be the summation of the horizontal component of the required connection strengths of the steel elements that are connected to the column base as follows:

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

(b) For columns, the horizontal component shall be equal to the lesser of the following:

(1) $2R_yF_yZ/(\alpha_sH)$ of the column

(2) The shear calculated using the overstrength seismic load.

(c) The summation of the required strengths of the horizontal components shall not be less than $07F_yZ/(\alpha_sH)$ of the column.

Exceptions:

(a) Single story columns with simple connections at both ends need not comply with Sections D2.6b(b) or D2.6b(c).

(b) Columns that are part of the systems defined in Sections E1, F1, G1, H1, H4 or combinations thereof need not comply with Section D2.6b(c).

(c) The minimum required shear strength per Section D2.6b(c) need not exceed the maximum load effect that can be transferred from the column to the foundation as determined by either a nonlinear analysis per Section C3, or an analysis that includes the effects of inelastic behavior resulting in $0.025H$ story drift at either the first or second story, but not both concurrently.

User Note: The horizontal components can include the shear load from columns and the horizontal component of the axial load from diagonal members framing into the column base. Horizontal forces for columns that are not part of the SFRS determined in accordance with this section typically will not govern over those determined according to Section D2.6b(c).

6c. Required Flexural Strength

Where column bases are designed as moment connections to the foundation, the required flexural strength of column bases that are designated as part of the SFRS, including their attachment to the foundation, shall be the summation of the required connection strengths of the steel elements that are connected to the column base as follows:

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

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

(1) $1.1R_yF_yZ/\alpha_s$ of the column; or

(2) The moment calculated using the overstrength seismic load, provided that a ductile limit state in either the column base or the foundation controls the design.

User Note: Moments at column to column base connections designed as simple connections may be ignored.

7. Composite Connections

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

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

(1) direct bearing from internal bearing mechanisms;

(2) shear connection;

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

(4) a combination of these means.

The contribution of different mechanisms is permitted to be combined only if the stiffness and deformation capacity of the mechanisms are compatible. Any potential bond strength between structural steel and reinforced concrete shall be ignored for the purpose of the connection force transfer mechanism.

(b) The nominal bearing and shear-friction strengths shall meet the requirements of ACI 318. Unless a higher strength is substantiated by cyclic testing, the nominal bearing and shear-friction strengths shall be reduced by 25% for the composite seismic systems described in Sections G3, H2, H3, H5 and H6.

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

(d) The nominal shear strength of concrete-encased steel panel zones in beam-to-column connections shall be calculated as the sum of the nominal strengths of the structural steel and confined reinforced concrete shear elements as determined in Section E3.6e and ACI 318 Section 18.8, respectively.

(e) Reinforcement shall be provided to resist all tensile forces in reinforced concrete components of the connections. Additionally, the concrete shall be confined with transverse reinforcement. All reinforcement shall be fully developed in tension or compression, as applicable, beyond the point at which it is no longer required to resist the forces. Development lengths shall be determined in accordance with ACI 318 Chapter 25. Additionally, development lengths for the systems described in Sections G3, H2, H3, H5 and H6 shall satisfy the requirements of ACI 318 Section 18.8.5.

(f) Composite connections shall satisfy 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, diagonal braces and walls.

(2) For connections between structural steel or composite beams and reinforced concrete or encased composite columns, transverse hoop reinforcement shall be provided in the connection region of the column to satisfy the requirements of ACI 318 Section 18.8, except for the following modifications:

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

(ii) Lap splices are permitted for perimeter ties when confinement of the splice is provided by face bearing plates or other means that prevents spalling of the concrete cover in the systems described in Sections G1, G2, H1 and H4.

(iii) 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.

User Note: The commentary provides guidance for determining panel-zone shear strength.

8. Steel Anchors

Where steel headed stud anchors or welded reinforcing bar anchors are part of the intermediate or special SFRS of Sections G2, G3, G4, H2, H3, H5 and H6, their shear and tensile strength shall be reduced by 25% from the specified strengths given in *Specification* Chapter I. The diameter of steel headed stud anchors shall be limited to ¾ in. (19 mm).

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

D3. DEFORMATION COMPATIBILITY OF NON-SFRS MEMBERS AND CONNECTIONS

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

User Note: ASCE/SEI 7 stipulates the preceding requirement for both structural steel and composite members and connections. Flexible shear connections that allow member end rotations in accordance with *Specification* Section J1.2 should be considered to satisfy these requirements. Inelastic deformations are permitted in connections or members provided they are self-limiting and do not create instability in the member. See the Commentary for further discussion.

D4. H-PILES

1. Design Requirements

Design of H-piles shall comply with the requirements of the *Specification* regarding design of members subjected to combined loads. H-piles located in site classes E or F as defined by ASCE/SEI 7 shall satisfy the requirements for moderately ductile members of Section D1.1.

2. Battered H-Piles

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

3. Tension

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

4. Protected Zone

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

CHAPTER E
MOMENT-FRAME SYSTEMS

This chapter provides the basis of design, the requirements for analysis, and the requirements for the system, members and connections for steel moment-frame systems.

The chapter is organized as follows:

E1. Ordinary Moment Frames (OMF)
E2. Intermediate Moment Frames (IMF)
E3. Special Moment Frames (SMF)
E4. Special Truss Moment Frames (STMF)
E5. Ordinary Cantilever Column Systems (OCCS)
E6. Special Cantilever Column Systems (SCCS)

User Note: The requirements of this chapter are in addition to those required by the *Specification* and the applicable building code.

E1. ORDINARY MOMENT FRAMES (OMF)

1. Scope

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

2. Basis of Design

OMF designed in accordance with these provisions are expected to provide minimal inelastic deformation capacity in their members and connections.

3. Analysis

There are no requirements specific to this system.

4. System Requirements

There are no requirements specific to this system.

5. Members

5a. Basic Requirements

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

5b. Protected Zones

There are no designated protected zones for OMF members.

6. Connections

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

6a. Demand Critical Welds

Complete-joint-penetration (CJP) groove welds of beam flanges to columns are demand critical welds, and shall satisfy the requirements of Sections A3.4b and I2.3.

6b. FR Moment Connections

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

(a) FR moment connections shall be designed for a required flexural strength that is equal to the expected beam flexural strength, $R_y M_p$, multiplied by 1.1 and divided by α_s, where α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD.

The required shear strength of the connection, V_u or V_a, as applicable, shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be determined as follows:

$$E_{cl} = 2(1.1R_y M_p)/L_{cf} \quad \text{(E1-1)}$$

where

L_{cf} = clear length of beam, in. (mm)
M_p = plastic bending moment, kip-in. (N-mm)
R_y = ratio of expected yield stress to the specified minimum yield stress, F_y

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

User Note: Factors that may limit the maximum moment and corresponding shear that can be transferred to the connection include column yielding, panel zone yielding, the development of the flexural strength of the beam at some distance away from the connection when web tapered members are used, and others. Further discussion is provided in the commentary.

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

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

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

(3) The shape of weld access holes shall be in accordance with clause 6.11.1.2 of AWS D1.8/D1.8M. Weld access hole quality requirements shall be in accordance with clause 6.11.2 of AWS D1.8/D1.8M.

(4) Continuity plates shall satisfy the requirements of Section E3.6f.

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

(5) The beam web shall be connected to the column flange using either a CJP groove weld extending between weld access holes, or using a bolted single plate shear connection designed for the required shear strength given in Section E1.6b(a).

For options (a) and (b) in Section E1.6b, continuity plates shall be provided as required by *Specification* Sections J10.1, J10.2 and J10.3. The bending moment used to check for continuity plates shall be the same bending moment used to design the beam-to-column connection; in other words, $1.1R_yM_n/\alpha_s$ or the maximum moment that can be transferred to the connection by the system.

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

6c. PR Moment Connections

PR moment connections shall satisfy the following requirements:

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

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

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

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

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

E2. INTERMEDIATE MOMENT FRAMES (IMF)

1. Scope

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

2. Basis of Design

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

3. Analysis

There are no requirements specific to this system.

4. System Requirements

4a. Stability Bracing of Beams

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

In addition, unless otherwise indicated by testing, beam braces shall be placed near concentrated forces, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the IMF. The placement of stability bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.

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

5. Members

5a. Basic Requirements

Beam and column members shall satisfy the requirements of Section D1 for moderately ductile members, unless otherwise qualified by tests.

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

5b. Beam Flanges

Changes in beam flange area in the protected zones, as defined in Section E2.5c, shall be gradual. The drilling of flange holes or trimming of beam flange width is not permitted unless testing or qualification demonstrates that the resulting configuration is able to develop stable plastic hinges to accommodate the required story drift angle. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.

5c. Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a protected zone and shall satisfy the requirements of Section D1.3. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Section K2.

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

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds and shall satisfy the requirements of Sections A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied.

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Complete-joint-penetration groove welds of beam flanges and beam webs to columns, unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Section K2.

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

6b. Beam-to-Column Connection Requirements

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

(a) The connection shall be capable of accommodating a story drift angle of at least 0.02 rad.

(b) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at a story drift angle of 0.02 rad.

6c. Conformance Demonstration

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

(a) Use of IMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for IMF in accordance with Section K1.

(c) Provision of qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(1) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2.

(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Section K2.

6d. Required Shear Strength

The required shear strength of the connection shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be determined as:

$$E_{cl} = 2(1.1R_yM_p)/L_h \quad \text{(E2-1)}$$

where

L_h = distance between beam plastic hinge locations, as defined within the test report or ANSI/AISC 358, in. (mm)

M_p = plastic bending moment, kip-in. (N-mm)

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

Exception: In lieu of Equation E2-1, the required shear strength of the connection shall be as specified in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.

6e. Panel Zone

There are no additional panel zone requirements.

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

6f. Continuity Plates

Continuity plates shall be provided in accordance with the provisions of Section E3.6f.

6g. Column Splices

Column splices shall comply with the requirements of Section E3.6g.

E3. SPECIAL MOMENT FRAMES (SMF)

1. Scope

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

2. Basis of Design

SMF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through flexural yielding of the SMF beams and limited yielding of column panel zones, or, where equivalent performance of the moment-frame system is demonstrated by substantiating analysis and testing, through yielding of the connections of beams to columns. Except where otherwise permitted in this section, columns shall be designed to be stronger than the fully yielded and strain-hardened beams or girders. Flexural yielding of columns at the base is permitted. Design of connections of beams to columns, including panel zones and continuity plates, shall be based on connection tests that provide the performance

required by Section E3.6b, and demonstrate this conformance as required by Section E3.6c.

3. Analysis

For special moment-frame systems that consist of isolated planar frames, there are no additional analysis requirements.

For moment-frame systems that include columns that form part of two intersecting special moment frames in orthogonal or multi-axial directions, the column analysis of Section E3.4a shall consider the potential for beam yielding in both orthogonal directions simultaneously.

User Note: For these columns, the required axial loads are defined in Section D1.4a(b).

4. System Requirements

4a. Moment Ratio

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

$$\frac{\sum M_{pc}^{*}}{\sum M_{pb}^{*}} > 1.0 \qquad \text{(E3-1)}$$

where

$\sum M_{pc}^{*}$ = sum of the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column, kip-in. (N-mm). It is permitted to determine $\sum M_{pc}^{*}$ as follows:

$$\sum M_{pc}^{*} = \sum Z_c \left(F_{yc} - \alpha_s P_r / A_g \right) \qquad \text{(E3-2)}$$

When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

$\sum M_{pb}^{*}$ = sum of the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline, kip-in. (N-mm). It is permitted to determine $\sum M_{pb}^{*}$ as follows:

$$\sum M_{pb}^{*} = \sum \left(M_{pr} + \alpha_s M_v \right) \qquad \text{(E3-3)}$$

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

F_{yb} = specified minimum yield stress of beam, ksi (MPa)

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

M_{pr} = maximum probable moment at the location of the plastic hinge, as determined in accordance with ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2, kip-in. (N-mm)

M_v = additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD or ASD load combinations, kip-in. (N-mm)
P_r = required axial compressive strength according to Section D1.4a, kips (N)
Z_c = plastic section modulus of the column about the axis of bending, in.^{3} (mm^3)

Exception: The requirement of Equation E3-1 shall not apply if the following conditions in (a) or (b) are satisfied.

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

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

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

User Note: For purposes of this exception, the available shear strengths of the columns should be calculated as the limit strengths considering the flexural strength at each end as limited by the flexural strength of the attached beams, or the flexural strength of the columns themselves, divided by H, where H is the story height.

The nominal compressive strength, P_c, shall be determined as follows:

$$P_c = F_{yc}A_g/\alpha_s \tag{E3-5}$$

and the required axial strength is $P_{rc} = P_{uc}$ (LRFD) or $P_{rc} = P_{ac}$ (ASD), as applicable.

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

4b. Stability Bracing of Beams

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

In addition, unless otherwise indicated by testing, beam braces shall be placed near concentrated forces, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the SMF. The placement of lateral bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.

The required strength and stiffness of stability bracing provided adjacent to plastic hinges shall be as required by Section D1.2c.

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

1. Braced Connections

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

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

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

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

(b) Each column-flange member brace shall be designed for a required strength that is equal to 2% of the available beam flange strength, $F_y b_f t_{bf}$, divided by α_s,
where
b_f = width of flange, in. (mm)
t_{bf} = thickness of beam flange, in. (mm)

2. Unbraced Connections

A column containing a beam-to-column connection with no member bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent member braces as the column height for buckling

transverse to the seismic frame and shall conform to *Specification* Chapter H, except that:

(a) The required column strength shall be determined from the load combinations in the applicable building code that include the overstrength seismic load.

The overstrength seismic load, E_{mh}, need not exceed 125% of the frame available strength based upon either the beam available flexural strength or panel-zone available shear strength.

(b) The slenderness L/r for the column shall not exceed 60,
where
L = length of column, in. (mm)
r = governing radius of gyration, in. (mm)

(c) The column required flexural strength transverse to the seismic frame shall include that moment caused by the application of the beam flange force specified in Section E3.4c(1)(b), in addition to the second-order moment due to the resulting column flange lateral displacement.

5. Members

5a. Basic Requirements

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

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

5b. Beam Flanges

Abrupt changes in beam flange area are prohibited in plastic hinge regions. The drilling of flange holes or trimming of beam flange width are not permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges to accommodate the required story drift angle. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.

5c. Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall satisfy the requirements of Section D1.3. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Section K2.

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

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied.

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Complete-joint-penetration groove welds of beam flanges and beam webs to columns, unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Section K1, or as determined in a program of qualification testing in accordance with Section K2.

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

6b. Beam-to-Column Connections

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

(a) The connection shall be capable of accommodating a story drift angle of at least 0.04 rad.

(b) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at a story drift angle of 0.04 rad, unless equivalent performance of the moment frame system is demonstrated through substantiating analysis conforming to ASCE/SEI 7 Sections 12.2.1.1 or 12.2.1.2,
where
M_p = plastic bending moment, kip-in. (N-mm)

6c. Conformance Demonstration

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

(a) Use of SMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for SMF in accordance with Section K1.

(c) Provision of qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided and shall be based on one of the following:

(1) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2

(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Section K2

6d. Required Shear Strength

The required shear strength of the connection shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be taken as:

$$E_{cl} = 2M_{pr}/L_h \tag{E3-6}$$

where
L_h = distance between plastic hinge locations as defined within the test report or ANSI/AISC 358, in. (mm)
M_{pr} = maximum probable moment at the plastic hinge location, as defined in Section E3.4a, kip-in. (N-mm)

When E_{cl} as defined in Equation E3-6 is used in ASD load combinations that are additive with other transient loads and that are based on ASCE/SEI 7, the 0.75 combination factor for transient loads shall not be applied to E_{cl}.

Where the exceptions to Equation E3-1 in Section E3.4a apply, the shear, E_{cl}, is permitted to be calculated based on the beam end moments corresponding to the expected flexural strength of the column multiplied by 1.1.

6e. Panel Zone

1. Required Shear Strength

The required shear strength of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The design shear strength shall be $\phi_v R_n$ and the allowable shear strength shall be R_n/Ω_v,

where

ϕ_v = 1.00 (LRFD)
Ω_v = 1.50 (ASD)

and the nominal shear strength, R_n, in accordance with the limit state of shear yielding, is determined as specified in *Specification* Section J10.6.

Alternatively, the required thickness of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested or prequalified connection.

Where the exceptions to Equation E3-1 in Section E3.4a apply, the beam moments used in calculating the required shear strength of the panel zone need not exceed those corresponding to the expected flexural strength of the column multiplied by 1.1.

2. Panel-Zone Thickness

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

$$t \geq (d_z + w_z)/90 \tag{E3-7}$$

where

$d_z = d - 2t_f$ of the deeper beam at the connection, in. (mm)
t = thickness of column web or individual doubler plate, in. (mm)
w_z = width of panel zone between column flanges, in. (mm)

When plug welds are used to join the doubler to the column web, it is permitted to use the total panel-zone thickness to satisfy Equation E3-7. Additionally, the individual thicknesses of the column web and doubler plate shall satisfy Equation E3-7, where d_z and w_z are modified to be the distance between plug welds. When plug welds are required, a minimum of four plug welds shall be provided and spaced in accordance with Equation E3-7.

3. Panel-Zone Doubler Plates

The thickness of doubler plates, if used, shall not be less than ¼ in. (6 mm).

When used, doubler plates shall meet the following requirements.

Where the required strength of the panel zone exceeds the design strength, or where the panel zone does not comply with Equation E3-7, doubler plates shall

be provided. Doubler plates shall be placed in contact with the web, or shall be spaced away from the web. Doubler plates with a gap of up to 1/16 in. (2 mm) between the doubler plate and the column web are permitted to be designed as being in contact with the web. When doubler plates are spaced away from the web, they shall be placed symmetrically in pairs on opposite sides of the column web.

Doubler plates in contact with the web shall be welded to the column flanges either using partial-joint-penetration (PJP) groove welds in accordance with AWS D1.8/D1.8M clause 4.3 that extend from the surface of the doubler plate to the column flange, or by using fillet welds. Spaced doubler plates shall be welded to the column flanges using complete-joint-penetration (CJP) groove welds, PJP groove welds, or fillet welds. The required strength of partial-joint-penetration groove welds or fillet welds shall equal the available shear yielding strength of the doubler-plate thickness.

(a) Doubler plates used without continuity plates

Doubler plates and the welds connecting the doubler plates to the column flanges shall extend at least 6 in. (150 mm) above and below the top and bottom of the deeper moment frame beam. For doubler plates in contact with the web, if the doubler-plate thickness alone and the column-web thickness alone both satisfy Equation E3-7, then no weld is required along the top and bottom edges of the doubler plate. If either the doubler-plate thickness alone or the column-web thickness alone does not satisfy Equation E3-7, then a minimum size fillet weld, as stipulated in *Specification* Table J2.4, shall be provided along the top and bottom edges of the doubler plate. These welds shall terminate 1.5 in. (38 mm) from the toe of the column fillet.

(b) Doubler plates used with continuity plates

Doubler plates are permitted to be either extended above and below the continuity plates or placed between the continuity plates.

(1) Extended doubler plates

Extended doubler plates shall be in contact with the web. Extended doubler plates and the welds connecting the doubler plates to the column flanges shall extend at least 6 in. (150 mm) above and below the top and bottom of the deeper moment frame beam. Continuity plates shall be welded to the extended doubler plates in accordance with the requirements in Section E3.6f.2(c). No welds are required at the top and bottom edges of the doubler plate.

(2) Doubler plates placed between continuity plates

Doubler plates placed between continuity plates are permitted to be in contact with the web or away from the web. Welds between the doubler plate and the column flanges shall extend between continuity plates, but

are permitted to stop no more than 1 in. (25 mm) from the continuity plate. The top and bottom of the doubler plate shall be welded to the continuity plates over the full length of the continuity plates in contact with the column web. The required strength of the doubler plate-to-continuity plate weld shall equal 75% of the available shear yield strength of the full doubler plate thickness over the contact length with the continuity plate.

User Note: When a beam perpendicular to the column web connects to a doubler plate, the doubler plate should be sized based on the shear from the beam end reaction in addition to the panel zone shear. When welding continuity plates to extended doubler plates, force transfer between the continuity plate and doubler plate must be considered. See commentary for further discussion.

6f. Continuity Plates

Continuity plates shall be provided as required by this section.

Exception: This section shall not apply in the following cases.

(a) Where continuity plates are otherwise determined in a connection prequalification in accordance with Section K1.

(b) Where a connection is qualified in accordance with Section K2 for conditions in which the test assembly omits continuity plates and matches the prototype beam and column sizes and beam span.

1. Conditions Requiring Continuity Plates

Continuity plates shall be provided in the following cases:

(a) Where the required strength at the column face exceeds the available column strength determined using the applicable local limit states stipulated in *Specification* Section J10, where applicable. Where so required, continuity plates shall satisfy the requirements of *Specification* Section J10.8 and the requirements of Section E3.6f.2.

For connections in which the beam flange is welded to the column flange, the column shall have an available strength sufficient to resist an applied force consistent with the maximum probable moment at face of column, M_f.

User Note: The beam flange force, P_f, corresponding to the maximum probable moment at the column face, M_f, may be determined as follows:

For connections with beam webs with a bolted connection to the column, P_f may be determined assuming only the beam flanges participate in transferring the moment M_f:

$$P_f = \frac{M_f}{\alpha_s d^*}$$

For connections with beam webs welded to the column, P_f may be determined assuming that the beam flanges and web both participate in transferring the moment, M_f, as follows:

$$P_f = \frac{0.85 M_f}{\alpha_s d^*}$$

where

M_f = maximum probable moment at face of column as defined in ANSI/AISC 358 for a prequalified moment connection or as determined from qualification testing, kip-in. (N-mm)

P_f = required strength at the column face for local limit states in the column, kip (N)

d^* = distance between centroids of beam flanges or beam flange connections to the face of the column, in. (mm)

(b) Where the column flange thickness is less than the limiting thickness, t_{lim}, determined in accordance with this provision.

(1) Where the beam flange is welded to the flange of a W-shape or built-up I-shaped column, the limiting column-flange thickness is:

$$t_{lim} = \frac{b_{bf}}{6} \quad \text{(E3-8)}$$

(2) Where the beam flange is welded to the flange of the I-shape in a boxed wide-flange column, the limiting column-flange thickness is:

$$t_{lim} = \frac{b_{bf}}{12} \quad \text{(E3-9)}$$

User Note: These continuity-plate requirements apply only to wide-flange column sections. Detailed formulas for determining continuity plate requirements for box-section columns have not been developed. It is noted that the performance of moment connections is dependent on the column flange stiffness in distributing the strain across the beam-to-column flange weld. Designers should consider the relative stiffness of the box-section column flange compared to those of tested assemblies in resisting the beam flange force to determine the need for continuity plates.

2. Continuity-Plate Requirements

Where continuity plates are required, they shall meet the requirements of this section.

(a) Continuity-Plate Width

The width of the continuity plate shall be determined as follows:

(1) For W-shape columns, continuity plates shall, at a minimum, extend from the column web to a point opposite the tips of the wider beam flanges.

(2) For boxed wide-flange columns, continuity plates shall extend the full width from column web to side plate of the column.

(b) Continuity-Plate Thickness

The minimum thickness of the plates shall be determined as follows:

(1) For one-sided connections, the continuity plate thickness shall be at least 50% of the thickness of the beam flange.

(2) For two-sided connections, the continuity plate thickness shall be at least equal to 75% of the thickness of the thicker beam flange on either side of the column.

(c) Continuity-Plate Welding

Continuity plates shall be welded to column flanges using CJP groove welds.

Continuity plates shall be welded to column webs or extended doubler plates using groove welds or fillet welds. The required strength of the welded joints of continuity plates to the column web or extended doubler plate shall be the lesser of the following:

(1) The sum of the available tensile strengths of the contact areas of the continuity plates to the column flanges that have attached beam flanges

(2) The available shear strength of the contact area of the plate with the column web or extended doubler plate

(3) The available shear strength of the column web, when the continuity plate is welded to the column web, or the available shear strength of the doubler plate, when the continuity plate is welded to an extended doubler plate

6g. Column Splices

Column splices shall comply with the requirements of Section D2.5.

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

1. Welded Column Flange Splices Using CJP Groove Welds

Where welds are used to make the flange splices, they shall be CJP groove welds, unless otherwise permitted in Section E3.6g.2.

2. Welded Column Flange Splices Using PJP Groove Welds

Where the specified minimum yield stress of the column shafts does not exceed 60 ksi (415 MPa) and the thicker flange is at least 5% thicker than the thinner flange, PJP groove welds are permitted to make the flange splices, and shall comply with the following requirements:

(a) The PJP flange weld or welds shall provide a minimum total effective throat of 85% of the thickness of the thinner column flange.

(b) A smooth transition in the thickness of the weld is provided from the outside of the thinner flange to the outside of the thicker flange. The transition shall be at a slope not greater than 1 in 2.5, and may be accomplished by sloping the weld surface, by chamfering the thicker flange to a thickness no less than 5% greater than the thickness of the thinner flange, or by a combination of these two methods.

(c) Tapered transitions between column flanges of different width shall be provided in accordance with Section D2.5b(2)(c).

(d) Where the flange weld is a double-bevel groove weld (i.e., on both sides of the flange):

(1) The unfused root face shall be centered within the middle half of the thinner flange, and

(2) Weld access holes that comply with the *Specification* shall be provided in the column section containing the groove weld preparation.

(e) Where the flange thickness of the thinner flange is not greater than 2½ in. (63 mm), and the weld is a single-bevel groove weld, weld access holes shall not be required.

3. Welded Column Web Splices Using CJP Groove Welds

The web weld or welds shall be made in a groove or grooves in the column web that extend to the access holes. The weld end(s) may be stepped back from the ends of the bevel(s) using a block sequence for approximately one weld size.

4. Welded Column Web Splices Using PJP Groove Welds

When PJP groove welds in column flanges that comply with Section E3.6g.2 are used, and the thicker web is at least 5% thicker than the thinner web, it is permitted to use PJP groove welds in column webs that comply with the following requirements:

(a) The PJP groove web weld or welds provide a minimum total effective throat of 85% of the thickness of the thinner column web.

(b) A smooth transition in the thickness of the weld is provided from the outside of the thinner web to the outside of the thicker web.

(c) Where the weld is a single-bevel groove, the thickness of the thinner web is not greater than 2½ in. (63 mm).

(d) Where no access hole is provided, the web weld or welds are made in a groove or grooves prepared in the column web extending the full length of the web between the *k*-areas. The weld end(s) are permitted to be stepped back from the ends of the bevel(s) using a block sequence for approximately one weld size.

(e) Where an access hole is provided, the web weld or welds are made in a groove or grooves in the column web that extend to the access holes. The weld end(s) are permitted to be stepped back from the ends of the bevel(s) using a block sequence for approximately one weld size.

5. Bolted Column Splices

Bolted column splices shall have a required flexural strength that is at least equal to $R_y F_y Z_x/\alpha_s$ of the smaller column, where Z_x is the plastic section modulus about the *x*-axis. The required shear strength of column web splices shall be at least equal to $\sum M_{pc}/(\alpha_s H_c)$, where $\sum M_{pc}$ is the sum of the plastic flexural strengths at the top and bottom ends of the column.

E4. SPECIAL TRUSS MOMENT FRAMES (STMF)

1. Scope

Special truss moment frames (STMF) of structural steel shall satisfy the requirements in this section.

2. Basis of Design

STMF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity within a special segment of the truss. STMF shall be limited to span lengths between columns not to exceed 65 ft (20 m) and overall depth not to exceed 6 ft (1.8 m). The columns and truss segments outside of the special segments shall be designed to remain essentially elastic under the forces that are generated by the fully yielded and strain-hardened special segment.

3. Analysis

Analysis of STMF shall satisfy the following requirements.

3a. Special Segment

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

3b. Nonspecial Segment

The required strength of nonspecial segment members and connections, including column members, shall be determined using the capacity-limited horizontal seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be taken as

the lateral forces necessary to develop the expected vertical shear strength of the special segment acting at mid-length and defined in Section E4.5c. Second-order effects at maximum design drift shall be included.

4. System Requirements

4a. Special Segment

Each horizontal truss that is part of the SFRS shall have a special segment that is located between the quarter points of the span of the truss. The length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67.

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

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

The required axial strength of the diagonal web members in the special segment due to dead and live loads within the special segment shall not exceed $0.03F_yA_g/\alpha_s$.

4b. Stability Bracing of Trusses

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

$$P_r = 0.06R_yF_yA_f/\alpha_s \quad \text{(E4-1)}$$

where

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

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

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

$$P_r = 0.02R_yP_{nc}/\alpha_s \quad \text{(E4-2)}$$

where

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

4d. Stiffness of Stability Bracing

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

$$P_r = R_y P_{nc}/\alpha_s \tag{E4-3}$$

5. Members

5a. Basic Requirements

Columns shall satisfy the requirements of Section D1.1 for highly ductile members.

5b. Special Segment Members

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

The available strength, ϕP_n (LRFD) and P_n/Ω (ASD), determined in accordance with the limit state of tensile yielding, shall be equal to or greater than 2.2 times the required strength, where

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

$$P_n = F_y A_g \tag{E4-4}$$

5c. Expected Vertical Shear Strength of Special Segment

The expected vertical shear strength of the special segment, V_{ne}, at mid-length, shall be determined as follows:

$$V_{ne} = \frac{3.60 R_y M_{nc}}{L_s} + 0.036 EI \frac{L}{L_s^3} + R_y \left(P_{nt} + 0.3 P_{nc}\right) \sin \alpha \tag{E4-5}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
I = moment of inertia of a chord member of the special segment, in.^{4} (mm^4)
L = span length of the truss, in. (mm)
L_s = length of the special segment, in. (mm)
M_{nc} = nominal flexural strength of a chord member of the special segment, kip-in. (N-mm)
P_{nc} = nominal axial compressive strength of a diagonal member of the special segment, kips (N)
P_{nt} = nominal axial tensile strength of a diagonal member of the special segment, kips (N)
α = angle of diagonal members with the horizontal, degrees

5d. Width-to-Thickness Limitations

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

5e. Built-Up Chord Members

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

5f. Protected Zones

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

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds and shall satisfy the requirements of Sections A3.4b and I2.3:

(a) Groove welds at column splices
(b) Welds at column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied.

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

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

The end connection of diagonal web members in the special segment shall have a required strength that is at least equal to the expected yield strength of the web member, determined as $R_yF_yA_g/\alpha_s$.

6c. Column Splices

Column splices shall comply with the requirements of Section E3.6g.

E5. ORDINARY CANTILEVER COLUMN SYSTEMS (OCCS)

1. Scope

Ordinary cantilever column systems (OCCS) of structural steel shall be designed in conformance with this section.

2. Basis of Design

OCCS designed in accordance with these provisions are expected to provide minimal inelastic drift capacity through flexural yielding of the columns.

3. Analysis

There are no requirements specific to this system.

4. System Requirements

4a. Columns

Columns shall be designed using the load combinations including the overstrength seismic load. The required axial strength, P_{rc}, shall not exceed 15% of the available axial strength, P_c, for these load combinations only.

4b. Stability Bracing of Columns

There are no additional requirements.

5. Members

5a. Basic Requirements

There are no additional requirements.

5b. Column Flanges

There are no additional requirements.

5c. Protected Zones

There are no designated protected zones.

6. Connections

6a. Demand Critical Welds

No demand critical welds are required for this system.

6b. Column Bases

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

E6. SPECIAL CANTILEVER COLUMN SYSTEMS (SCCS)

1. Scope

Special cantilever column systems (SCCS) of structural steel shall be designed in conformance with this section.

2. Basis of Design

SCCS designed in accordance with these provisions are expected to provide limited inelastic drift capacity through flexural yielding of the columns.

3. Analysis

There are no requirements specific to this system.

4. System Requirements

4a. Columns

Columns shall be designed using the load combinations including the overstrength seismic load. The required strength, P_{rc}, shall not exceed 15% of the available axial strength, P_c, for these load combinations only.

4b. Stability Bracing of Columns

Columns shall be braced to satisfy the requirements applicable to beams classified as moderately ductile members in Section D1.2a.

5. Members

5a. Basic Requirements

Column members shall satisfy the requirements of Section D1.1 for highly ductile members.

5b. Column Flanges

Abrupt changes in column flange area are prohibited in the protected zone as designated in Section E6.5c.

5c. Protected Zones

The region at the base of the column subject to inelastic straining shall be designated as a protected zone, and shall satisfy the requirements of Section D1.3. The length of the protected zone shall be two times the column depth.

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at column-to-base plate connections

6b. Column Bases

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

CHAPTER F

BRACED FRAME AND SHEAR WALL SYSTEMS

This chapter provides the basis of design, the requirements for analysis, and the requirements for the system, members and connections for steel braced-frame and shear-wall systems.

The chapter is organized as follows:

F1. Ordinary Concentrically Braced Frames (OCBF)
F2. Special Concentrically Braced Frames (SCBF)
F3. Eccentrically Braced Frames (EBF)
F4. Buckling-Restrained Braced Frames (BRBF)
F5. Special Plate Shear Walls (SPSW)

User Note: The requirements of this chapter are in addition to those required by the *Specification* and the applicable building code.

F1. ORDINARY CONCENTRICALLY BRACED FRAMES (OCBF)

1. Scope

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

2. Basis of Design

This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if they are accounted for in the member design by determination of eccentric moments using the over-strength seismic load.

OCBF designed in accordance with these provisions are expected to provide limited inelastic deformation capacity in their members and connections.

3. Analysis

There are no additional analysis requirements.

4. System Requirements

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

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

(a) The required strength of the beam shall be determined assuming that the braces provide no support of dead and live loads. For load combinations that include

earthquake effects, the seismic load effect, E, on the beam shall be determined as follows:

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

(i) The load effect based upon the overstrength seismic load

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

(2) The forces in braces in compression shall be assumed to be equal to $0.3P_n$ where

P_n = nominal axial compressive strength, kips (N)

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

4b. K-Braced Frames

K-type braced frames shall not be used for OCBF.

4c. Multi-Tiered Braced Frames

An ordinary concentrically braced frame is permitted to be configured as a multi-tiered braced frame (MT-OCBF) when the following requirements are met.

(a) Braces shall be used in opposing pairs at every tier level.

(b) Braced frames shall be configured with in-plane struts at each tier level.

(c) Columns shall be torsionally braced at every strut-to-column connection location.

User Note: The requirements for torsional bracing are typically satisfied by connecting the strut to the column to restrain torsional movement of the column. The strut must have adequate flexural strength and stiffness and an appropriate connection to the column to perform this function.

(d) The required strength of brace connections shall be determined from the load combinations of the applicable building code, including the overstrength seismic load, with the horizontal seismic load effect, E, multiplied by a factor of 1.5.

(e) The required axial strength of the struts shall be determined from the load combinations of the applicable building code, including the overstrength seismic load, with the horizontal seismic load effect, E, multiplied by a factor of 1.5. In tension-compression X-bracing, these forces shall be determined in the absence of compression braces.

(f) The required axial strengths of the columns shall be determined from the load combinations of the applicable building code, including the overstrength seismic load, with the horizontal seismic load effect, E, multiplied by a factor of 1.5.

(g) For all load combinations, columns subjected to axial compression shall be designed to resist bending moments due to second-order and geometric imperfection effects. As a minimum, imperfection effects are permitted to be represented by an out-of-plane horizontal notional load applied at every tier level and equal to 0.006 times the vertical load contributed by the compression brace connecting the column at the tier level.

(h) When tension-only bracing is used, requirements (d), (e) and (f) need not be satisfied if:

(1) All braces have a controlling slenderness ratio of 200 or more.

(2) The braced frame columns are designed to resist additional in-plane bending moments due to the unbalanced lateral forces determined at every tier level using the capacity-limited seismic load based on expected brace strengths. The expected brace strength in tension is $R_y F_y A_g$,
where
F_y = specified minimum yield stress, ksi (MPa)
R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y

The unbalanced lateral force at any tier level shall not be less than 5% of the larger horizontal brace component resisted by the braces below and above the tier level.

5. Members

5a. Basic Requirements

Braces shall satisfy the requirements of Section D1.1 for moderately ductile members.

Exception: Braces in tension-only frames with slenderness ratios greater than 200 need not comply with this requirement.

5b. Slenderness

Braces in V or inverted-V configurations shall have

$$\frac{L_c}{r} \le 4\sqrt{E/F_y} \qquad \text{(F1-1)}$$

where
E = modulus of elasticity of steel, ksi (MPa)
L_c = effective length of brace = KL, in. (mm)
K = effective length factor
r = governing radius of gyration, in. (mm)

5c. Beams

The required strength of beams and their connections shall be determined using the overstrength seismic load.

6. Connections

6a. Brace Connections

The required strength of diagonal brace connections shall be determined using the overstrength seismic load.

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

(a) In tension, the expected yield strength divided by α_s, which shall be determined as $R_y F_y A_g/\alpha_s$, where α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD.

(b) In compression, the expected brace strength in compression divided by α_s, which is permitted to be taken as the lesser of $R_y F_y A_g/\alpha_s$ and $1.1F_{cre}A_g/\alpha_s$, where F_{cre} is determined from *Specification* Chapter E using the equations for F_{cr}, except that the expected yield stress, $R_y F_y$, is used in lieu of F_y. The brace length used for the determination of F_{cre} shall not exceed the distance from brace end to brace end.

(c) When oversized holes are used, the required strength for the limit state of bolt slip need not exceed the seismic load effect based upon the load combinations without overstrength as stipulated by the applicable building code.

7. Ordinary Concentrically Braced Frames above Seismic Isolation Systems

OCBF above the isolation system shall satisfy the requirements of this section and of Section F1, excluding Section F1.4a.

7a. System Requirements

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

7b. Members

Braces shall have a slenderness ratio, $L_c/r \le 4\sqrt{E/F_y}$.

F2. SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)

1. Scope

Special concentrically braced frames (SCBF) of structural steel shall be designed in conformance with this section. Collector beams that connect SCBF braces shall be considered to be part of the SCBF.

2. Basis of Design

This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.

SCBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension.

3. Analysis

The required strength of columns, beams, struts and connections in SCBF shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be taken as the larger force determined from the following analyses:

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

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

(c) For multi-tiered braced frames, analyses representing progressive yielding and buckling of the braces from weakest tier to strongest. Analyses shall consider both directions of frame loading.

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

The expected brace strength in tension is $R_yF_yA_g$, where A_g is the gross area, in.^{2} (mm^2).

The expected brace strength in compression is permitted to be taken as the lesser of $R_yF_yA_g$ and $(1/0.877)F_{cre}A_g$, where F_{cre} is determined from *Specification* Chapter E using the equations for F_{cr}, except that the expected yield stress, R_yF_y, is used in lieu of F_y. The brace length used for the determination of F_{cre} shall not exceed the distance from brace end to brace end.

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

User Note: Braces with a slenderness ratio of 200 (the maximum permitted by Section F2.5b) buckle elastically for permissible materials; the value of $0.3F_{cr}$ for such braces is 2.1 ksi (14 MPa). This value may be used in Section F2.3(b) for braces of any slenderness and a liberal estimate of the required strength of framing members will be obtained.

Exceptions:

(a) It is permitted to neglect flexural forces resulting from seismic drift in this determination.

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

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

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

(c) The required strength of bracing connections shall be as specified in Section F2.6c.

4. System Requirements

4a. Lateral Force Distribution

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

Where opposing diagonal braces along a frame line do not occur in the same bay, the required strengths of the diaphragm, collectors, and elements of the horizontal framing system shall be determined such that the forces resulting from the post-buckling behavior using the analysis requirements of Section F2.3 can be transferred between the braced bays. The required strength of the collector need not exceed the required strength determined by the load combinations of the applicable building code, including the overstrength seismic load, applied to a building model in which all compression braces have been removed. The required strengths of the collectors shall not be based on a load less than that stipulated by the applicable building code.

4b. V- and Inverted V-Braced Frames

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

(a) Beams shall be continuous between columns.

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

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

User Note: One method of demonstrating sufficient out-of-plane strength and stiffness of the beam is to apply the bracing force defined in Equation A-6-7 of Appendix 6 of the *Specification* to each flange so as to form a torsional couple; this loading should be in conjunction with the flexural forces determined from the analysis required by Section F2.3. The stiffness of the beam (and its restraints) with respect to this torsional loading should be sufficient to satisfy Equation A-6-8 of the *Specification*.

4c. K-Braced Frames

K-type braced frames shall not be used for SCBF.

4d. Tension-Only Frames

Tension-only frames shall not be used in SCBF.

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

4e. Multi-Tiered Braced Frames

A special concentrically braced frame is permitted to be configured as a multi-tiered braced frame (MT-SCBF) when the following requirements are satisfied.

(a) Braces shall be used in opposing pairs at every tier level.

(b) Struts shall satisfy the following requirements:

(1) Horizontal struts shall be provided at every tier level.

(2) Struts that are intersected by braces away from strut-to-column connections shall also meet the requirements of Section F2.4b. When brace buckling occurs out-of-plane, torsional moments arising from brace buckling shall be considered when verifying lateral bracing or minimum out-of-plane strength and stiffness requirements. The torsional moments shall correspond to $1.1R_yM_p/\alpha_s$ of the brace about the critical buckling axis, but need not exceed forces corresponding to the flexural resistance of the brace connection, where M_p is the plastic bending moment, kip-in. (N-mm), and α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD.

(c) Columns shall satisfy the following requirements:

(1) Columns shall be torsionally braced at every strut-to-column connection location.

User Note: The requirements for torsional bracing are typically satisfied by connecting the strut to the column to restrain torsional movement of the column. The strut must have adequate flexural strength and stiffness and an appropriate connection to the column to perform this function.

(2) Columns shall have sufficient strength to resist forces arising from brace buckling. These forces shall correspond to $1.1R_yM_p/\alpha_s$ of the brace about the critical buckling axis, but need not exceed forces corresponding to the flexural resistance of the brace connections.

(3) For all load combinations, columns subjected to axial compression shall be designed to resist bending moments due to second-order and geometric imperfection effects. As a minimum, imperfection effects are permitted to be represented by an out-of-plane horizontal notional load applied at every tier level and equal to 0.006 times the vertical load contributed by the compression brace intersecting the column at the tier level. In all cases, the multiplier B_1, as defined in *Specification* Appendix 8, need not exceed 2.0.

(d) Each tier in a multi-tiered braced frame shall be subject to the drift limitations of the applicable building code, but the drift shall not exceed 2% of the tier height.

5. Members

5a. Basic Requirements

Columns, beams and braces shall satisfy the requirements of Section D1.1 for highly ductile members. Struts in MT-SCBF shall satisfy the requirements of Section D1.1 for moderately ductile members.

5b. Diagonal Braces

Braces shall comply with the following requirements:

(a) Slenderness: Braces shall have a slenderness ratio of $L_c/r \le 200$,
where
L_c = effective length of brace = KL, in. (mm)
r = governing radius of gyration, in. (mm)

(b) Built-up braces: The spacing of connectors shall be such that the slenderness ratio, a/r_i, of individual elements between the connectors does not exceed 0.4 times the governing slenderness ratio of the built-up member,
where
a = distance between connectors, in. (mm)
r_i = minimum radius of gyration of individual component, in. (mm)

The sum of the available shear strengths of the connectors shall equal or exceed the available tensile strength of each element. The spacing of connectors shall be uniform. Not less than two connectors shall be used in a built-up member. Connectors shall not be located within the middle one-fourth of the clear brace length.

Exception: Where the buckling of braces about their critical bucking axis does not cause shear in the connectors, the design of connectors need not comply with this provision.

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

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

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

5c. Protected Zones

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

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

(b) Elements that connect braces to beams and columns

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied.

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Welds at beam-to-column connections conforming to Section F2.6b(c)

6b. Beam-to-Column Connections

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

(a) The connection assembly shall be a simple connection meeting the requirements of *Specification* Section B3.4a, where the required rotation is taken to be 0.025 rad; or

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

(1) A moment corresponding to the expected beam flexural strength, R_yM_p, multiplied by 1.1 and divided by α_s

(2) A moment corresponding to the sum of the expected column flexural strengths, $\Sigma\left(R_yF_yZ\right)$, multiplied by 1.1 and divided by α_s

This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength seismic load.

(c) The beam-to-column connection shall meet the requirements of Section E1.6b(c).

6c. Brace Connections

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

1. Required Tensile Strength

The required tensile strength shall be the lesser of the following:

(a) The expected yield strength in tension of the brace, determined as $R_yF_yA_g$, divided by α_s.

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

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

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

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

When oversized holes are used, the required strength for the limit state of bolt slip need not exceed the seismic load effect determined using the overstrength seismic loads.

User Note: For other limit states, the loadings of (a) and (b) apply.

2. Required Compressive Strength

Brace connections shall be designed for a required compressive strength, based on buckling limit states, that is equal to the expected brace strength in compression divided by α_s, where the expected brace strength in compression is as defined in Section F2.3.

3. Accommodation of Brace Buckling

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

(a) Required Flexural Strength: Brace connections designed to withstand the flexural forces imposed by brace buckling shall have a required flexural strength equal to the expected brace flexural strength multiplied by 1.1 and divided by α_s. The expected brace flexural strength shall be determined as $R_y M_p$ of the brace about the critical buckling axis.

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

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

4. Gusset Plates

For out-of-plane brace buckling, welds that attach a gusset plate directly to a beam flange or column flange shall have available shear strength equal to $0.6R_yF_yt_p/\alpha_s$ times the joint length,

where

F_y = specified minimum yield stress of the gusset plate, ksi (MPa)
R_y = ratio of the expected yield stress to the specified minimum yield stress of the gusset plate, F_y
t_p = thickness of the gusset plate, in (mm)

Exception: Alternatively, these welds may be designed to have available strength to resist gusset-plate edge forces corresponding to the brace force specified in Section F2.6c.2 combined with the gusset plate weak-axis flexural strength determined in the presence of those forces.

User Note: The expected shear strength of the gusset plate may be developed using double-sided fillet welds with leg size equal to $0.74t_p$ for ASTM A572 Grade 50 plate and $0.62t_p$ for ASTM A36 plate and E70 electrodes. Smaller welds may be justified using the exception.

6d. Column Splices

Column splices shall comply with the requirements of Section D2.5. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds. Column splices shall be designed to develop at least 50% of the lesser plastic flexural strength, M_p, of the connected members, divided by α_s.

The required shear strength shall be $\left(\sum M_p/\alpha_s\right)/H_c$,

where

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

$\sum M_p$ = sum of the plastic flexural strengths, F_yZ, of the top and bottom ends of the column, kip-in. (N-mm)

F3. ECCENTRICALLY BRACED FRAMES (EBF)

1. Scope

Eccentrically braced frames (EBF) of structural steel shall be designed in conformance with this section.

2. Basis of Design

This section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace or column, forming a link that is subject to shear and flexure. Eccentricities less than the beam depth are permitted in the brace connection away from the link if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.

EBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through shear or flexural yielding in the links.

Where links connect directly to columns, design of their connections to columns shall provide the performance required by Section F3.6e.1 and demonstrate this conformance as required by Section F3.6e.2.

3. Analysis

The required strength of diagonal braces and their connections, beams outside links, and columns shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be taken as the forces developed in the member assuming the forces at the ends of the links correspond to the

adjusted link shear strength. The adjusted link shear strength shall be taken as R_y times the nominal shear strength of the link, V_n, given in Section F3.5b.2, multiplied by 1.25 for I-shaped links and 1.4 for box links.

Exceptions:

(a) The effect of capacity-limited horizontal forces, E_{cl}, is permitted to be taken as 0.88 times the forces determined in this section for the design of the portions of beams outside links.

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

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

(1) Forces corresponding to the resistance of the foundation to overturning uplift

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

The inelastic link rotation angle shall be determined from the inelastic portion of the design story drift. Alternatively, the inelastic link rotation angle is permitted to be determined from nonlinear analysis as defined in Section C3.

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

4. System Requirements

4a. Link Rotation Angle

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

(a) For links of length $1.6M_p/V_p$ or less: 0.08 rad

(b) For links of length $2.6M_p/V_p$ or greater: 0.02 rad

where

M_p = plastic bending moment of a link, kip-in. (N-mm)
V_p = plastic shear strength of a link, kips (N)

Linear interpolation between the above values shall be used for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

4b. Bracing of Link

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

5. Members

5a. Basic Requirements

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

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

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

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

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

5b. Links

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

1. Limitations

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

Links shall satisfy the requirements of Section D1.1 for highly ductile members.

Exceptions: Flanges of links with I-shaped sections with link lengths, $e \le 1.6\, M_p/V_p$, are permitted to satisfy the requirements for moderately ductile members. Webs of links with box sections with link lengths, $e \le 1.6 M_p/V_p$, are permitted to satisfy the requirements for moderately ductile members.

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

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

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

2. Shear Strength

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

$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

(a) For shear yielding

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

where

$$V_p = 0.6F_yA_{lw} \text{ for } \alpha_sP_r/P_y \le 0.15 \tag{F3-2}$$

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

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

$$= 2(d - 2t_f)t_w \text{ for box link sections} \tag{F3-5}$$

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

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

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

$$P_y = \text{axial yield strength} = F_yA_g \tag{F3-6}$$

d = overall depth of link, in. (mm)

t_f = thickness of flange, in. (mm)

t_w = thickness of web, in. (mm)

(b) For flexural yielding

$$V_n = 2M_p/e \tag{F3-7}$$

where

$$M_p = F_yZ \text{ for } \alpha_sP_r/P_y \le 0.15 \tag{F3-8}$$

$$M_p = F_yZ\left(\frac{1-\alpha_sP_r/P_y}{0.85}\right) \text{ for } \alpha_sP_r/P_y > 0.15 \tag{F3-9}$$

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

e = length of link, defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face, in. (mm)

3. Link Length

If $\alpha_sP_r/P_y > 0.15$, the length of the link shall be limited as follows:

When $\rho' \le 0.5$

$$e \le \frac{1.6M_p}{V_p} \tag{F3-10}$$

When $\rho' > 0.5$

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

where

$$\rho' = \frac{P_r/P_y}{V_r/V_y} \tag{F3-12}$$

V_r = V_u (LRFD) or V_a (ASD), as applicable, kips (N)
V_u = required shear strength using LRFD load combinations, kips (N)
V_a = required shear strength using ASD load combinations, kips (N)
V_y = shear yield strength, kips (N)

$$= 0.6F_yA_{lw} \tag{F3-13}$$

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

4. Link Stiffeners for I-Shaped Cross Sections

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than the larger of $0.75t_w$ or ⅜ in. (10 mm), where b_f and t_w are the link flange width and link web thickness, respectively.

Links shall be provided with intermediate web stiffeners as follows:

(a) Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle of 0.08 rad or $(52t_w - d/5)$ for link rotation angles of 0.02 rad or less. Linear interpolation shall be used for values between 0.08 and 0.02 rad.

(b) Links of length greater than or equal to $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the link.

(c) Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements of (a) and (b) in the preceding.

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

Intermediate web stiffeners shall be full depth. For links that are less than 25 in. (630 mm) in depth, stiffeners shall be provided on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or ⅜ in. (10 mm), whichever is larger, and the width shall not be less than $(b_f/2) - t_w$. For links that are 25 in. (630 mm) in depth or greater, intermediate stiffeners with these dimensions shall be provided on both sides of the web.

The required strength of fillet welds connecting a link stiffener to the link web shall be F_yA_{st}/α_s, where A_{st} is the horizontal cross-sectional area of the link stiffener, F_y is the specified minimum yield stress of the stiffener, and α_s is the LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD. The required strength of fillet welds connecting the stiffener to the link flanges is $F_yA_{st}/(4\alpha_s)$.

5. Link Stiffeners for Box Sections

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

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

(a) For links of length $1.6M_p/V_p$ or less, and with web depth-to-thickness ratio, h/t_w, greater than or equal to $0.67\sqrt{\frac{E}{R_yF_y}}$, full-depth web stiffeners shall be provided on one side of each link web, spaced at intervals not exceeding $20t_w - (d - 2t_f)/8$.

(b) For links of length $1.6M_p/V_p$ or less and with web depth-to-thickness ratio, h/t_w, less than $0.67\sqrt{\frac{E}{R_yF_y}}$, no intermediate web stiffeners are required.

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

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

The required strength of fillet welds connecting a link stiffener to the link web shall be F_yA_{st}/α_s, where A_{st} is the horizontal cross-sectional area of the link stiffener.

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

5c. Protected Zones

Links in EBF are protected zones, and shall meet the requirements of Section D1.3.

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds and shall meet the requirements of Sections A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied:

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Welds at beam-to-column connections conforming to Section F3.6b(c)

(d) Where links connect to columns, welds attaching the link flanges and the link web to the column

(e) In built-up beams, welds within the link connecting the webs to the flanges

6b. Beam-to-Column Connections

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

(a) The connection assembly is a simple connection meeting the requirements of *Specification* Section B3.4a where the required rotation is taken to be 0.025 rad; or

(b) The connection assembly is designed to resist a moment equal to the lesser of the following:

(1) A moment corresponding to the expected beam flexural strength, R_yM_p, multiplied by 1.1 and divided by α_s,
where
M_p = plastic bending moment, kip-in. (N-mm)

(2) A moment corresponding to the sum of the expected column flexural strengths, $\sum(R_yF_yZ)$, multiplied by 1.1 and divided by α_s,
where
F_y = specified minimum yield stress, ksi (MPa)
Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength seismic load.

(c) The beam-to-column connection satisfies the requirements of Section E1.6b(c).

6c. Brace Connections

When oversized holes are used, the required strength for the limit state of bolt slip need not exceed the seismic load effect determined using the overstrength seismic load.

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

6d. Column Splices

Column splices shall comply with the requirements of Section D2.5. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds. Column splices shall be designed to develop at least 50% of the lesser plastic bending moment, M_p, of the connected members, divided by α_s.

The required shear strength shall be $\sum M_p/(\alpha_s H_c)$,
where

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

$\sum M_p$ = sum of the plastic flexural strengths, $F_y Z$, at the top and bottom ends of the column, kip-in. (N-mm)

6e. Link-to-Column Connections

1. Requirements

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

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

(b) The shear resistance of the connection, measured at the required link rotation angle, shall be at least equal to the expected shear strength of the link, $R_y V_n$, where V_n is determined in accordance with Section F3.5b.2.

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

2. Conformance Demonstration

Link-to-column connections shall meet the preceding requirements by one of the following:

(a) Use a connection prequalified for EBF in accordance with Section K1.

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

(b) Provide qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(1) Tests reported in research literature or documented tests performed for other projects that are representative of project conditions, within the limits specified in Section K2.

(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection material properties, within the limits specified in Section K2.

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

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

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

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

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

F4. BUCKLING-RESTRAINED BRACED FRAMES (BRBF)

1. Scope

Buckling-restrained braced frames (BRBF) of structural steel shall be designed in conformance with this section.

2. Basis of Design

This section is applicable to frames with specially fabricated braces concentrically connected to beams and columns. Eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.

BRBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through brace yielding in tension and compression. Design of braces shall provide the performance required by Sections F4.5b.1 and F4.5b.2, and demonstrate this conformance as required by Section F4.5b.3. Braces shall be designed, tested and detailed to accommodate expected

deformations. Expected deformations are those corresponding to a story drift of at least 2% of the story height or two times the design story drift, whichever is larger, in addition to brace deformations resulting from deformation of the frame due to gravity loading.

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

2a. Brace Strength

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

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

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

P_{ysc} = axial yield strength of steel core, ksi (MPa)
β = compression strength adjustment factor
ω = strain hardening adjustment factor

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

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

2b. Adjustment Factors

Adjustment factors shall be determined as follows:

The compression strength adjustment factor, β, shall be calculated as the ratio of the maximum compression force to the maximum tension force of the test specimen measured from the qualification tests specified in Section K3.4c at strains corresponding to the expected deformations. The larger value of β from the two required brace qualification tests shall be used. In no case shall β be taken as less than 1.0.

The strain hardening adjustment factor, ω, shall be calculated as the ratio of the maximum tension force measured from the qualification tests specified in Section K3.4c at strains corresponding to the expected deformations to the measured yield force, P_{ysc}, of the test specimen. The larger value of ω from the two required qualification tests shall be used. Where the tested steel core material of the subassemblage test specimen required in Section K3.2 does not match that of the prototype, ω shall be based on coupon testing of the prototype material.

2c. Brace Deformations

The expected brace deformation shall be determined from the story drift specified in Section F4.2. Alternatively, the brace expected deformation is permitted to be determined from nonlinear analysis as defined in Section C3.

3. Analysis

The required strength of columns, beams, struts and connections in BRBF shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be taken as the forces developed in the member assuming the forces in all braces correspond to their adjusted strength in compression or in tension.

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

The adjusted brace strength in tension shall be as given in Section F4.2a.

Exceptions:

(a) It is permitted to neglect flexural forces resulting from seismic drift in this determination. Moment resulting from a load applied to the column between points of lateral support, including Section F4.4d loads, must be considered.

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

(1) The forces corresponding to the resistance of the foundation to overturning uplift. Section F4.4d in-plane column load requirements shall apply.

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

4. System Requirements

4a. V- and Inverted V-Braced Frames

V-type and inverted-V-type braced frames shall satisfy the following requirements:

(a) The required strength of beams and struts intersected by braces, their connections and supporting members shall be determined based on the load combinations of the applicable building code assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the vertical and horizontal earthquake effect, E, on the beam shall be determined from the adjusted brace strengths in tension and compression.

(b) Beams and struts shall be continuous between columns. Beams and struts shall be braced to meet the requirements for moderately ductile members in Section D1.2a.1.

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

User Note: The beam has sufficient out-of-plane strength and stiffness if the beam bent in the horizontal plane meets the required brace strength and required brace stiffness for column nodal bracing as prescribed in the *Specification*. P_r may be taken as the required compressive strength of the brace.

4b. K-Braced Frames

K-type braced frames shall not be used for BRBF.

4c. Lateral Force Distribution

Where the compression strength adjustment factor, β, as determined in Section F4.2b exceeds 1.3, the lateral force distribution shall comply with the following:

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

4d. Multi-Tiered Braced Frames

A buckling-restrained braced frame is permitted to be configured as a multi-tiered braced frame (MT-BRBF) when the following requirements are satisfied.

(a) The effects of out-of-plane forces due to the mass of the structure and supported items as required by the applicable building code shall be combined with the forces obtained from the analyses required by Section F4.3.

(b) Struts shall be provided at every brace-to-column connection location.

(c) Columns shall meet the following requirements:

(1) Columns of multi-tiered braced frames shall be designed as simply supported for the height of the frame between points of out-of-plane support and shall satisfy the greater of the following in-plane load requirements at each tier:

(i) Loads induced by the summation of frame shears from adjusted brace strengths between adjacent tiers from Section F4.3 analysis. Analysis shall consider variation in permitted core strength.

> **User Note:** Specifying the BRB using the desired brace capacity, P_{ysc}, rather than a desired core area is recommended for the multi-tiered buckling-restrained braced (BRB) frame to reduce the effect of material variability and allow for the design of equal or nearly equal tier capacities.

(ii) A minimum notional load equal to 0.5% times the adjusted braced strength frame shear of the higher strength adjacent tier. The notional load shall be applied to create the greatest load effect on the column.

(2) Columns shall be torsionally braced at every strut-to-column connection location.

User Note: The requirements for torsional bracing are typically satisfied by connecting the strut to the column to restrain torsional movement of the column. The strut must have adequate flexural strength and stiffness and have an appropriate connection to the column to perform this function.

(d) Each tier in a multi-tiered braced frame shall be subject to the drift limitations of the applicable building code, but the drift shall not exceed 2% of the tier height.

5. Members

5a. Basic Requirements

Beams and columns shall satisfy the requirements of Section D1.1 for moderately ductile members.

5b. Diagonal Braces

1. Assembly

Braces shall be composed of a structural steel core and a system that restrains the steel core from buckling.

(a) Steel Core

Plates used in the steel core that are 2 in. (50 mm) thick or greater shall satisfy the minimum notch toughness requirements of Section A3.3.

Splices in the steel core are not permitted.

(b) Buckling-Restraining System

The buckling-restraining system shall consist of the casing for the steel core. In stability calculations, beams, columns and gussets connecting the core shall be considered parts of this system.

The buckling-restraining system shall limit local and overall buckling of the steel core for the expected deformations.

User Note: Conformance to this provision is demonstrated by means of testing as described in Section F4.5b.3.

2. Available Strength

The steel core shall be designed to resist the entire axial force in the brace.

The brace design axial strength, ϕP_{ysc} (LRFD), and the brace allowable axial strength, P_{ysc}/Ω (ASD), in tension and compression, in accordance with the limit state of yielding, shall be determined as follows:

$$P_{ysc} = F_{ysc} A_{sc} \tag{F4-1}$$

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

where

A_{sc} = cross-sectional area of the yielding segment of the steel core, in.^{2} (mm^2)

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

User Note: Load effects calculated based on adjusted brace strengths should not be based upon the overstrength seismic load.

3. Conformance Demonstration

The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Section K3. Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace subassemblage that includes brace connection rotational demands complying with Section K3.2 and the other shall be either a uniaxial or a subassemblage test complying with Section K3.3. Both test types shall be based upon one of the following:

(a) Tests reported in research or documented tests performed for other projects

(b) Tests that are conducted specifically for the project

Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that addresses the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes. Tests are permitted to qualify a design when the provisions of Section K3 are met.

5c. Protected Zones

The protected zone shall include the steel core of braces and elements that connect the steel core to beams and columns, and shall satisfy the requirements of Section D1.3.

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds and shall satisfy the requirements of Section A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at the column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied:

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Welds at beam-to-column connections conforming to Section F4.6b(c)

6b. Beam-to-Column Connections

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

(a) The connection assembly shall be a simple connection meeting the requirements of *Specification* Section B3.4a where the required rotation is taken to be 0.025 rad; or

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

(1) A moment corresponding to the expected beam flexural strength, R_yM_p, multiplied by 1.1 and divided by α_s,
where
M_p = plastic bending moment, kip-in. (N-mm)

(2) A moment corresponding to the sum of the expected column flexural strengths, $\sum(R_yF_yZ)$, multiplied by 1.1 and divided by α_s,
where
Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)
α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD

This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength seismic load.

(c) The beam-to-column connection shall meet the requirements of Section E1.6b(c).

6c. Diagonal Brace Connections

1. Required Strength

The required strength of brace connections in tension and compression (including beam-to-column connections if part of the braced-frame system) shall be the adjusted brace strength divided by α_s, where the adjusted brace strength is as defined in Section F4.2a.

When oversized holes are used, the required strength for the limit state of bolt slip need not exceed P_{ysc}/α_s.

2. Gusset Plate Requirements

Lateral bracing consistent with that used in the tests upon which the design is based shall be provided.

User Note: This provision may be met by designing the gusset plate for a transverse force consistent with transverse bracing forces determined from testing, by adding a stiffener to it to resist this force, or by providing a brace to the gusset plate. Where the supporting tests did not include transverse bracing, no such bracing is required. Any attachment of bracing to the steel core must be included in the qualification testing.

6d. Column Splices

Column splices shall comply with the requirements of Section D2.5. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds. Column splices shall be designed to develop at least 50% of the lesser plastic bending moment, M_p, of the connected members, divided by α_s.

The required shear strength, V_r, shall be determined as follows:

$$V_r = \frac{\sum M_p}{\alpha_s H_c} \tag{F4-2}$$

where

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

$\sum M_p$ = sum of the plastic bending moments, $F_y Z$, at the top and bottom ends of the column, kip-in. (N-mm)

F5. SPECIAL PLATE SHEAR WALLS (SPSW)

1. Scope

Special plate shear walls (SPSW) of structural steel shall be designed in conformance with this section. This section is applicable to frames with steel web plates connected to beams and columns.

2. Basis of Design

SPSW designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through web plate yielding and as plastic-hinge formation in the ends of horizontal boundary elements (HBE). Vertical boundary elements (VBE) are not expected to yield in shear; VBE are not expected to yield in flexure except at the column base.

3. Analysis

The webs of SPSW shall not be considered as resisting gravity forces.

(a) An analysis in conformance with the applicable building code shall be performed. The required strength of web plates shall be 100% of the required shear strength of the frame from this analysis. The required strength of the frame consisting of VBE and HBE alone shall be not less than 25% of the frame shear force from this analysis.

(b) The required strength of HBE, VBE, and connections in SPSW shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be determined from an analysis in which all webs are assumed to resist forces corresponding to their expected strength in tension at an angle, α, as determined in Section F5.5b and HBE are resisting flexural forces at each end equal to $1.1R_yM_p/\alpha_s$,

where

F_y = specified minimum yield stress, ksi (MPa)
M_p = plastic bending moment, kip-in. (N-mm)
R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y
α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD

Webs shall be determined to be in tension neglecting the effects of gravity loads.

The expected web yield stress shall be taken as R_yF_y. When perforated walls are used, the effective expected tension stress is as defined in Section F5.7a.4.

Exception: The required strength of VBE need not exceed the forces determined from nonlinear analysis as defined in Section C3.

User Note: Shear forces per Equation E1-1 must be included in this analysis. Designers should be aware that in some cases forces from the analysis in the applicable building code will govern the design of HBE.

User Note: Shear forces in beams and columns are likely to be high and shear yielding must be evaluated.

4. System Requirements

4a. Stiffness of Boundary Elements

The stiffness of vertical boundary elements (VBE) and horizontal boundary elements (HBE) shall be such that the entire web plate is yielded at the design story drift. VBE and HBE conforming to the following requirements shall be deemed to comply with this requirement. The VBE shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_c, not less than $0.0031t_wh^4/L$. The HBE have moments of inertia about an axis taken perpendicular to the plane of the web, I_b, not less than $0.0031L^4/h$ times the difference in web plate thicknesses above and below,

where

L = distance between VBE centerlines, in. (mm)
h = distance between HBE centerlines, in. (mm)
t_w = thickness of the web, in. (mm)

4b. HBE-to-VBE Connection Moment Ratio

The moment ratio provisions in Section E3.4a shall be met for all HBE/VBE intersections without including the effects of the webs.

4c. Bracing

HBE shall be braced to satisfy the requirements for moderately ductile members in Section D1.2a.

4d. Openings in Webs

Openings in webs shall be bounded on all sides by intermediate boundary elements extending the full width and height of the panel respectively, unless otherwise justified by testing and analysis or permitted by Section F5.7.

5. Members

5a. Basic Requirements

HBE, VBE and intermediate boundary elements shall satisfy the requirements of Section D1.1 for highly ductile members.

5b. Webs

The panel design shear strength, ϕV_n (LRFD), and the allowable shear strength, V_n/Ω (ASD), in accordance with the limit state of shear yielding, shall be determined as follows:

$$V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha \qquad \text{(F5-1)}$$

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

where

L_{cf} = clear distance between column flanges, in. (mm)
t_w = thickness of the web, in. (mm)
α = angle of web yielding in degrees, as measured relative to the vertical. The angle of inclination, α, is permitted to be taken as 40°, or is permitted to be calculated as follows:

$$\tan^4 \alpha = \frac{1 + \dfrac{t_w L}{2A_c}}{1 + t_w h \left(\dfrac{1}{A_b} + \dfrac{h^3}{360 I_c L} \right)} \qquad \text{(F5-2)}$$

where

A_b = cross-sectional area of an HBE, in.^{2} (mm^2)

A_c = cross-sectional area of a VBE, in.^{2} (mm^2)

5c. HBE

HBE shall be designed to preclude flexural yielding at regions other than near the beam-to-column connection. This requirement shall be met by one of the following:

(a) HBE with available strength to resist twice the simple-span beam moment based on gravity loading and web-plate yielding.

(b) HBE with available strength to resist the simple-span beam moment based on gravity loading and web-plate yielding and with reduced flanges meeting the requirements of ANSI/AISC 358 Section 5.8 Step 1 with $c = 0.25b_f$.

5d. Protected Zone

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

(a) The webs of SPSW

(b) Elements that connect webs to HBE and VBE

(c) The plastic hinging zones at each end of the HBE, over a region ranging from the face of the column to one beam depth beyond the face of the column, or as otherwise specified in Section E3.5c

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds and shall satisfy the requirements of Section A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied.

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Welds at HBE-to-VBE connections

6b. HBE-to-VBE Connections

HBE-to-VBE connections shall satisfy the requirements of Section E1.6b.

1. Required Strength

The required shear strength of an HBE-to-VBE connection shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be taken as the shear calculated from Equation E1-1 together with the shear resulting from the expected yield strength in tension of the webs yielding at an angle α.

2. Panel Zones

The VBE panel zone next to the top and base HBE of the SPSW shall comply with the requirements in Section E3.6e.

6c. Connections of Webs to Boundary Elements

The required strength of web connections to the surrounding HBE and VBE shall equal the expected yield strength, in tension, of the web calculated at an angle α.

6d. Column Splices

Column splices shall comply with the requirements of Section D2.5. Where welds are used to make the splice, they shall be complete-joint-penetration groove welds. Column splices shall be designed to develop at least 50% of the lesser plastic bending moment, M_p, of the connected members, divided by α_s. The required shear strength, V_r, shall be determined by Equation F4-2.

7. Perforated Webs

7a. Regular Layout of Circular Perforations

A perforated plate conforming to this section is permitted to be used as the web of an SPSW. Perforated webs shall have a regular pattern of holes of uniform diameter spaced evenly over the entire web-plate area in an array pattern so that holes align diagonally at a uniform angle to the vertical. A minimum of four horizontal and four vertical lines of holes shall be used. Edges of openings shall have a surface roughness of 500 μ-in. (13 microns) or less.

1. Strength

The panel design shear strength, ϕV_n (LRFD), and the allowable shear strength, V_n/Ω (ASD), in accordance with the limit state of shear yielding, shall be determined as follows for perforated webs with holes that align diagonally at 45° from the horizontal:

$$V_n = 0.42 F_y t_w L_{cf} \left(1 - \frac{0.7D}{S_{diag}}\right) \qquad \text{(F5-3)}$$

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

where

D = diameter of the holes, in. (mm)

S_{diag} = shortest center-to-center distance between the holes measured on the 45° diagonal, in. (mm)

2. Spacing

The spacing, S_{diag}, shall be at least $1.67D$.

The distance between the first holes and web connections to the HBE and VBE shall be at least D, but shall not exceed $D + 0.7S_{diag}$.

3. Stiffness

The stiffness of such regularly perforated infill plates shall be calculated using an effective web-plate thickness, t_{eff}, given by:

$$t_{eff} = \frac{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)}{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)\left(1 - \frac{N_r D \sin\alpha}{H_c}\right)} t_w \qquad \text{(F5-4)}$$

where

H_c = clear column (and web-plate) height between beam flanges, in. (mm)

N_r = number of horizontal rows of perforations

t_w = web-plate thickness, in. (mm)

α = angle of the shortest center-to-center lines in the opening array to vertical, degrees

User Note: Perforating webs in accordance with Section F5.7a forces the development of web yielding in a direction parallel to that of the holes alignment. As such, for the case addressed by Section F5.7a, α is equal to 45°.

4. Effective Expected Tension Stress

The effective expected tension for analysis is $R_y F_y(1 - 0.7D/S_{diag})$.

7b. Reinforced Corner Cut-Out

Quarter-circular cut-outs are permitted at the corners of the webs provided that the webs are connected to a reinforcement arching plate following the edge of the cut-outs. The plates shall be designed to allow development of the full strength of the solid web and maintain its resistance when subjected to deformations corresponding to the design story drift.

1. Design for Tension

The arching plate shall have the available strength to resist the axial tension force, P_r, resulting from web-plate tension in the absence of other forces:

$$P_r = \frac{R_y F_y t_w R^2/\alpha_s}{4e} \tag{F5-5}$$

where

F_y = specified minimum yield stress of the web plate, in.^{2} (mm^2)
R = radius of the cut-out, in. (mm)
R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y
$e = R\left(1-\sqrt{2}/2\right)$, in. (mm) (F5-6)

HBE and VBE shall be designed to resist the axial tension forces acting at the end of the arching reinforcement.

2. Design for Combined Axial and Flexural Forces

The arching plate shall have the available strength to resist the combined effects of axial force, P_r, and moment, M_r, in the plane of the web resulting from connection deformation in the absence of other forces:

$$P_r = \frac{15EI_y}{\alpha_s\left(16e^2\right)}\left(\frac{\Delta}{H}\right) \tag{F5-7}$$

$$M_r = P_r e \tag{F5-8}$$

where

E = modulus of elasticity, ksi (MPa)
H = height of story, in. (mm)
I_y = moment of inertia of the plate about the y-axis, in.^{4} (mm^4)
Δ = design story drift, in. (mm)

HBE and VBE shall be designed to resist the combined axial and flexural required strengths acting at the end of the arching reinforcement.

CHAPTER G
COMPOSITE MOMENT-FRAME SYSTEMS

This chapter provides the basis of design, the requirements for analysis, and the requirements for the system, members and connections for composite moment-frame systems.

The chapter is organized as follows:

G1. Composite Ordinary Moment Frames (C-OMF)
G2. Composite Intermediate Moment Frames (C-IMF)
G3. Composite Special Moment Frames (C-SMF)
G4. Composite Partially Restrained Moment Frames (C-PRMF)

User Note: The requirements of this chapter are in addition to those required by the *Specification* and the applicable building code.

G1. COMPOSITE ORDINARY MOMENT FRAMES (C-OMF)

1. Scope

Composite ordinary moment frames (C-OMF) shall be designed in conformance with this section. This section is applicable to moment frames with fully restrained (FR) connections that consist of either composite or reinforced concrete columns and structural steel, concrete-encased composite, or composite beams.

2. Basis of Design

C-OMF designed in accordance with these provisions are expected to provide minimal inelastic deformation capacity in their members and connections.

The requirements of Sections A1, A2, A3.5, A4, B1, B2, B3, B4, D2.7, and Chapter C apply to C-OMF. All other requirements in Chapters A, B, D, I, J and K are not applicable to C-OMF.

User Note: Composite ordinary moment frames, comparable to reinforced concrete ordinary moment frames, are only permitted in seismic design categories B or below in ASCE/SEI 7. This is in contrast to steel ordinary moment frames, which are permitted in higher seismic design categories. The design requirements are commensurate with providing minimal ductility in the members and connections.

3. Analysis

There are no requirements specific to this system.

4. System Requirements

There are no requirements specific to this system.

5. Members

There are no additional requirements for steel or composite members beyond those in the *Specification*. Reinforced concrete columns shall meet the requirements of ACI 318, excluding Chapter 18.

5a. Protected Zones

There are no designated protected zones.

6. Connections

Connections shall be fully restrained (FR) and shall satisfy the requirements of Section D2.7.

6a. Demand Critical Welds

There are no requirements specific to this system.

G2. COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)

1. Scope

Composite intermediate moment frames (C-IMF) shall be designed in conformance with this section. This section is applicable to moment frames with fully restrained (FR) connections that consist of composite or reinforced concrete columns and structural steel, concrete-encased composite, or composite beams.

2. Basis of Design

C-IMF designed in accordance with these provisions are expected to provide limited inelastic deformation capacity through flexural yielding of the C-IMF beams and columns, and shear yielding of the column panel zones. Design of connections of beams to columns, including panel zones, continuity plates and diaphragms shall provide the performance required by Section G2.6b and demonstrate this conformance as required by Section G2.6c.

User Note: Composite intermediate moment frames, comparable to reinforced concrete intermediate moment frames, are only permitted in seismic design categories C or below in ASCE/SEI 7. This is in contrast to steel intermediate moment frames, which are permitted in higher seismic design categories. The design requirements are commensurate with providing limited ductility in the members and connections.

3. Analysis

There are no requirements specific to this system.

4. System Requirements

4a. Stability Bracing of Beams

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

In addition, unless otherwise indicated by testing, beam braces shall be placed near concentrated forces, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the C-IMF.

The required strength and stiffness of stability bracing provided adjacent to plastic hinges shall be in accordance with Section D1.2c.

5. Members

5a. Basic Requirements

Steel and composite members shall satisfy the requirements of Section D1.1 for moderately ductile members.

5b. Beam Flanges

Abrupt changes in the beam flange area are prohibited in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is not permitted unless testing or qualification demonstrates that the resulting configuration is able to develop stable plastic hinges to accommodate the required story drift angle.

5c. Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a protected zone and shall satisfy the requirements of Section D1.3.

User Note: The plastic hinge zones at the ends of C-IMF beams should be treated as protected zones. In general, the protected zone will extend from the face of the composite column to one-half of the beam depth beyond the plastic hinge point.

6. Connections

Connections shall be fully restrained (FR) and shall satisfy the requirements of Section D2 and this section.

6a. Demand Critical Welds

There are no requirements specific to this system.

6b. Beam-to-Column Connections

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

(a) The connection shall be capable of accommodating a story drift angle of at least 0.02 rad.

(b) The measured flexural resistance of the connection determined at the column face shall equal at least $0.80M_p$ of the connected beam at a story drift angle of 0.02 rad, where M_p is defined as the plastic bending moment of the steel, concrete-encased or composite beams and shall meet the requirements of *Specification* Chapter I.

6c. Conformance Demonstration

Beam-to-column connections used in the SFRS shall satisfy the requirements of Section G2.6b by one of the following:

(a) Use of C-IMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for C-IMF in accordance with Section K1.

(c) Results of at least two qualifying cyclic test results conducted in accordance with Section K2. The tests are permitted to be based on one of the following:

(1) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2.

(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Section K2.

(d) Calculations that are substantiated by mechanistic models and component limit state design criteria consistent with these provisions.

6d. Required Shear Strength

The required shear strength of the connection shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be taken as:

$$E_{cl} = 2(1.1\,M_{p,exp})/L_h \tag{G2-1}$$

where

$M_{p,exp}$ = expected flexural strength of the steel, concrete-encased or composite beam, kip-in. (N-mm)

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

For a concrete-encased or composite beam, $M_{p,exp}$ shall be calculated using the plastic stress distribution or the strain compatibility method. Applicable R_y and R_c factors shall be used for different elements of the cross section while establishing section force equilibrium and calculating the flexural strength.

User Note: For steel beams, $M_{p,exp}$ in Equation G2-1 may be taken as R_yM_p of the beam.

6e. Connection Diaphragm Plates

Connection diaphragm plates are permitted for filled composite columns both external to the column and internal to the column.

Where diaphragm plates are used, the thickness of the plates shall be at least the thickness of the beam flange.

The diaphragm plates shall be welded around the full perimeter of the column using either complete-joint-penetration (CJP) groove welds or two-sided fillet welds. The required strength of these joints shall not be less than the available strength of the contact area of the plate with the column sides.

Internal diaphragms shall have circular openings sufficient for placing the concrete.

6f. Column Splices

In addition to the requirements of Section D2.5, column splices shall comply with the requirements of this section. Where welds are used to make the splice, they shall be CJP groove welds. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to the plastic flexural strength, M_{pcc}, of the smaller composite column. The required shear strength of column web splices shall be at least equal to $\sum M_{pcc}/H$,

where

H = height of story, in. (mm)

$\sum M_{pcc}$ = sum of the plastic flexural strengths at the top and bottom ends of the composite column, kip-in. (N-mm)

For composite columns, the plastic flexural strength shall satisfy the requirements of *Specification* Chapter I including the required axial strength, P_{rc}.

G3. COMPOSITE SPECIAL MOMENT FRAMES (C-SMF)

1. Scope

Composite special moment frames (C-SMF) shall be designed in conformance with this section. This section is applicable to moment frames with fully restrained (FR) connections that consist of either composite or reinforced concrete columns and either structural steel or concrete-encased composite or composite beams.

2. Basis of Design

C-SMF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through flexural yielding of the C-SMF beams and limited yielding of the column panel zones. Except where otherwise permitted in this section, columns shall be designed to be stronger than the fully yielded and strain-hardened beams or girders. Flexural yielding of columns at the base is permitted. Design of connections of beams to columns, including panel zones, continuity plates and diaphragms, shall provide the performance required by Section G3.6b and demonstrate this conformance as required by Section G3.6c.

3. Analysis

For special moment-frame systems that consist of isolated planar frames, there are no additional analysis requirements.

For moment-frame systems that include columns that form part of two intersecting special moment frames in orthogonal or multi-axial directions, the column analysis of Section G3.4a shall consider the potential for beam yielding in both orthogonal directions simultaneously.

4. System Requirements

4a. Moment Ratio

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

$$\frac{\sum M^*_{pcc}}{\sum M^*_{p,exp}} > 1.0 \qquad \text{(G3-1)}$$

where

$\sum M^*_{pcc}$ = sum of the projections of the plastic flexural strengths, M_{pcc}, of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. For composite columns, the plastic flexural strength, M_{pcc}, shall satisfy the requirements of *Specification* Chapter I including the required axial strength, P_{rc}. For reinforced concrete columns, the plastic flexural strength, M_{pcc}, shall be calculated based on the provisions of ACI 318 including the required axial strength, P_{rc}. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

$\sum M^*_{p,exp}$ = sum of the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\sum M^*_{p,exp} = \sum(1.1M_{p,exp} + M_{uv})$, where $M_{p,exp}$ is calculated as specified in Section G2.6d.

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

Exception: The exceptions of Section E3.4a shall apply, except that the force limit in Exception (a) shall be $P_{rc} < 0.1P_c$.

4b. Stability Bracing of Beams

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

In addition, unless otherwise indicated by testing, beam braces shall be placed near concentrated forces, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the C-SMF.

The required strength and stiffness of stability bracing provided adjacent to plastic hinges shall be in accordance with Section D1.2c.

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

Composite columns with unbraced connections shall satisfy the requirements of Section E3.4c.2.

5. Members

5a. Basic Requirements

Steel and composite members shall meet the requirements of Section D1.1 for highly ductile members.

Exception: Reinforced concrete-encased beams shall meet the requirements for Section D1.1 for moderately ductile members if the reinforced concrete cover is at least 2 in. (50 mm) and confinement is provided by hoop reinforcement in regions where plastic hinges are expected to occur under seismic deformations. Hoop reinforcement shall satisfy the requirements of ACI 318 Section 18.6.4.

Concrete-encased composite beams that are part of C-SMF shall also meet the following requirement. The distance from the extreme concrete compression fiber to the plastic neutral axis shall not exceed:

$$Y_{PNA} = \frac{Y_{con} + d}{1 + \left(\frac{1{,}700\, F_y}{E} \right)} \tag{G3-2}$$

where

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

5b. Beam Flanges

Abrupt changes in beam flange area are prohibited in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is prohibited unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges to accommodate the required story drift angle.

5c. Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall meet the requirements of Section D1.3.

User Note: The plastic hinge zones at the ends of C-SMF beams should be treated as protected zones. In general, the protected zone will extend from the face of the composite column to one-half of the beam depth beyond the plastic hinge point.

6. Connections

Connections shall be fully restrained (FR) and shall meet the requirements of Section D2 and this section.

User Note: All subsections of Section D2 are relevant for C-SMF.

6a. Demand Critical Welds

The following welds are demand critical welds and shall meet the requirements of Section A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at the column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Complete-joint-penetration groove welds of beam flanges to columns, diaphragm plates that serve as a continuation of beam flanges, shear plates within the girder depth that transition from the girder to an encased steel shape, and beam webs to columns

6b. Beam-to-Column Connections

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

(a) The connection shall be capable of accommodating a story drift angle of at least 0.04 rad.

(b) The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at a story drift angle of 0.04 rad, where M_p is determined in accordance with Section G2.6b.

6c. Conformance Demonstration

Beam-to-composite column connections used in the SFRS shall meet the requirements of Section G3.6b by one of the following:

(a) Use of C-SMF connections designed in accordance with ANSI/AISC 358

(b) Use of a connection prequalified for C-SMF in accordance with Section K1.

(c) The connections shall be qualified using test results obtained in accordance with Section K2. Results of at least two cyclic connection tests shall be provided, and shall be based on one of the following:

(1) Tests reported in research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2.

(2) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified by Section K2.

(d) When beams are uninterrupted or continuous through the composite or reinforced concrete column, beam flange welded joints are not used, and the connection is not otherwise susceptible to premature fracture, other substantiating data is permitted to demonstrate conformance.

Connections that accommodate the required story drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified in Section G3.6d are permitted. In addition to satisfying the preceding requirements, the design shall demonstrate that any additional drift due to connection deformation is accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.

6d. Required Shear Strength

The required shear strength of the connection, V_u, shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be taken as:

$$E_{cl} = 2(1.1M_{p,exp})/L_h \quad \text{(G3-3)}$$

where

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

$M_{p,exp}$ = expected flexural strength of the steel, concrete-encased or composite beams, kip-in. (N-mm). For concrete-encased or composite beams, $M_{p,exp}$ shall be calculated according to Section G2.6d

6e. Connection Diaphragm Plates

The continuity plates or diaphragms used for infilled column moment connections shall satisfy the requirements of Section G2.6e.

6f. Column Splices

Composite column splices shall satisfy the requirements of Section G2.6f.

G4. COMPOSITE PARTIALLY RESTRAINED MOMENT FRAMES (C-PRMF)

1. Scope

Composite partially restrained moment frames (C-PRMF) shall be designed in conformance with this section. This section is applicable to moment frames that consist of structural steel columns and composite beams that are connected with partially restrained (PR) moment connections that satisfy the requirements in *Specification* Section B3.4b(b).

2. Basis of Design

C-PRMF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through yielding in the ductile components of the composite PR beam-to-column moment connections. Flexural yielding of columns at the base is permitted. Design of connections of beams to columns shall be based on connection tests that provide the performance required by Section G4.6c and demonstrate this conformance as required by Section G4.6d.

3. Analysis

Connection flexibility and composite beam action shall be accounted for in determining the dynamic characteristics, strength and drift of C-PRMF.

For purposes of analysis, the stiffness of beams shall be determined with an effective moment of inertia of the composite section.

4. System Requirements

There are no requirements specific to this system.

5. Members

5a. Columns

Steel columns shall meet the requirements of Sections D1.1 for moderately ductile members.

5b. Beams

Composite beams shall be unencased, fully composite, and shall meet the requirements of Section D1.1 for moderately ductile members. A solid slab shall be provided for a distance of 12 in. (300 mm) from the face of the column in the direction of moment transfer.

5c. Protected Zones

There are no designated protected zones.

6. Connections

Connections shall be partially restrained (PR) and shall meet the requirements of Section D2 and this section.

User Note: All subsections of Section D2 are relevant for C-PRMF.

6a. Demand Critical Welds

The following welds are demand critical welds and shall satisfy the requirements of Section A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at the column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied.

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

6b. Required Strength

The required strength of the beam-to-column PR moment connections shall be determined including the effects of connection flexibility and second-order moments.

6c. Beam-to-Column Connections

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

(a) The connection shall be capable of accommodating a connection rotation of at least 0.02 rad.

(b) The measured flexural resistance of the connection determined at the column face shall increase monotonically to a value of at least $0.5M_p$ of the connected beam at a connection rotation of 0.02 rad, where M_p is defined as the moment corresponding to plastic stress distribution over the composite cross section, and shall meet the requirements of *Specification* Chapter I.

6d. Conformance Demonstration

Beam-to-column connections used in the SFRS shall meet the requirements of Section G4.6c by provision of qualifying cyclic test results in accordance with Section K2. Results of at least two cyclic connection tests shall be provided and shall be based on one of the following:

(a) Tests reported in research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2.

(b) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified by Section K2.

6e. Column Splices

Column splices shall meet the requirements of Section G2.6f.

CHAPTER H

COMPOSITE BRACED-FRAME AND SHEAR-WALL SYSTEMS

This chapter provides the basis of design, the requirements for analysis, and the requirements for the system, members and connections for composite braced-frame and shear-wall systems.

The chapter is organized as follows:

H1. Composite Ordinary Braced Frames (C-OBF)
H2. Composite Special Concentrically Braced Frames (C-SCBF)
H3. Composite Eccentrically Braced Frames (C-EBF)
H4. Composite Ordinary Shear Walls (C-OSW)
H5. Composite Special Shear Walls (C-SSW)
H6. Composite Plate Shear Walls—Concrete Encased (C-PSW/CE)
H7. Composite Plate Shear Walls—Concrete Filled (C-PSW/CF)

User Note: The requirements of this chapter are in addition to those required by the *Specification* and the applicable building code.

H1. COMPOSITE ORDINARY BRACED FRAMES (C-OBF)

1. Scope

Composite ordinary braced frames (C-OBF) shall be designed in conformance with this section. Columns shall be structural steel, encased composite, filled composite or reinforced concrete members. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. This section is applicable to braced frames that consist of concentrically connected members where at least one of the elements (columns, beams or braces) is a composite or reinforced concrete member.

2. Basis of Design

This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if they are accounted for in the member design by determination of eccentric moments.

C-OBF designed in accordance with these provisions are expected to provide limited inelastic deformations in their members and connections.

The requirements of Sections A1, A2, A3.5, A4, B1, B2, B3, B4 and D2.7, and Chapter C apply to C-OBF. All other requirements in Chapters A, B, D, I, J and K do not apply to C-OBF.

User Note: Composite ordinary braced frames, comparable to other steel braced frames designed per the *Specification* using $R = 3$, are only permitted in seismic design categories A, B or C in ASCE/SEI 7. This is in contrast to steel ordinary braced frames, which are permitted in higher seismic design categories. The design requirements are commensurate with providing minimal ductility in the members and connections.

3. Analysis

There are no requirements specific to this system.

4. System Requirements

There are no requirements specific to this system.

5. Members

5a. Basic Requirements

There are no requirements specific to this system.

5b. Columns

There are no requirements specific to this system. Reinforced concrete columns shall satisfy the requirements of ACI 318, excluding Chapter 18.

5c. Braces

There are no requirements specific to this system.

5d. Protected Zones

There are no designated protected zones.

6. Connections

Connections shall satisfy the requirements of Section D2.7.

6a. Demand Critical Welds

There are no requirements specific to this system.

H2. COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAMES (C-SCBF)

1. Scope

Composite special concentrically braced frames (C-SCBF) shall be designed in conformance with this section. Columns shall be encased or filled composite. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. Collector beams that connect C-SCBF braces shall be considered to be part of the C-SCBF.

2. Basis of Design

This section is applicable to braced frames that consist of concentrically connected members. Eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.

C-SCBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension.

3. Analysis

The analysis requirements for C-SCBF shall satisfy the analysis requirements of Section F2.3 modified to account for the entire composite section in determining the expected brace strengths in tension and compression.

4. System Requirements

The system requirements for C-SCBF shall satisfy the system requirements of Section F2.4. Composite braces are not permitted for use in multi-tiered braced frames.

5. Members

5a. Basic Requirements

Composite columns and steel or composite braces shall satisfy the requirements of Section D1.1 for highly ductile members. Steel or composite beams shall satisfy the requirements of Section D1.1 for moderately ductile members.

User Note: In order to satisfy this requirement, the actual width-to-thickness ratio of square and rectangular filled composite braces may be multiplied by a factor, $(0.264 + 0.0082L_c/r)$, for L_c/r between 35 and 90; L_c/r being the effective slenderness ratio of the brace.

5b. Diagonal Braces

Structural steel and filled composite braces shall satisfy the requirements for SCBF of Section F2.5b. The radius of gyration in Section F2.5b shall be taken as that of the steel section alone.

5c. Protected Zones

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

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

(b) Elements that connect braces to beams and columns

6. Connections

Design of connections in C-SCBF shall be based on Section D2 and the provisions of this section.

6a. Demand Critical Welds

The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at the column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are met.

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Welds at beam-to-column connections conforming to Section H2.6b(b)

6b. Beam-to-Column Connections

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

(a) The connection shall be a simple connection meeting the requirements of *Specification* Section B3.4a where the required rotation is taken to be 0.025 rad; or

(b) Beam-to-column connections shall satisfy the requirements for fully-restrained (FR) moment connections as specified in Sections D2, G2.6d and G2.6e.

The required flexural strength of the connection shall be determined from analysis and shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength seismic load.

6c. Brace Connections

Brace connections shall satisfy the requirement of Section F2.6c, except that the required strength shall be modified to account for the entire composite section in determining the expected brace strength in tension and compression. Applicable R_y factors shall be used for different elements of the cross section for calculating the expected brace strength. The expected brace flexural strength shall be determined as $M_{p,exp}$, where $M_{p,exp}$ is calculated as specified in Section G2.6d.

6d. Column Splices

In addition to the requirements of Section D2.5, column splices shall comply with the requirements of this section. Where welds are used to make the splice, they shall

be complete-joint-penetration groove welds. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to the plastic flexural strength, M_{pcc}, of the smaller composite column. The required shear strength of column web splices shall be at least equal to $\sum M_{pcc}/H$, where $\sum M_{pcc}$ is the sum of the plastic flexural strengths at the top and bottom ends of the composite column and H is the height of story, in. (mm). The plastic flexural strength shall meet the requirements of *Specification* Chapter I including the required axial strength, P_{rc}.

H3. COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)

1. Scope

Composite eccentrically braced frames (C-EBF) shall be designed in conformance with this section. Columns shall be encased composite or filled composite. Beams shall be structural steel or composite beams. Links shall be structural steel. Braces shall be structural steel or filled composite members. This section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace or column.

2. Basis of Design

C-EBF shall satisfy the requirements of Section F3.2, except as modified in this section.

This section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace or column, forming a link that is subject to shear and flexure. Eccentricities less than the beam depth are permitted in the brace connection away from the link if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.

C-EBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through shear or flexural yielding in the links.

The available strength of members shall satisfy the requirements in the *Specification*, except as modified in this section.

3. Analysis

The analysis of C-EBF shall satisfy the analysis requirements of Section F3.3.

4. System Requirements

The system requirements for C-EBF shall satisfy the system requirements of Section F3.4.

5. Members

The member requirements of C-EBF shall satisfy the member requirements of Section F3.5.

6. Connections

The connection requirements of C-EBF shall satisfy the connection requirements of Section F3.6 except as noted in the following.

6a. Beam-to-Column Connections

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

(a) The connection shall be a simple connection meeting the requirements of *Specification* Section B3.4a where the required rotation is taken to be 0.025 rad; or

(b) Beam-to-column connections shall satisfy the requirements for FR moment connections as specified in Section D2, and Sections G2.6d and G2.6e shall apply.

The required flexural strength of the connection shall be determined from analysis and shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength seismic load.

H4. COMPOSITE ORDINARY SHEAR WALLS (C-OSW)

1. Scope

Composite ordinary shear walls (C-OSW) shall be designed in conformance with this section. This section is applicable to uncoupled reinforced concrete shear walls with composite boundary elements, and coupled reinforced concrete shear walls, with or without composite boundary elements, with structural steel or composite coupling beams that connect two or more adjacent walls.

2. Basis of Design

C-OSW designed in accordance with these provisions are expected to provide limited inelastic deformation capacity through yielding in the reinforced concrete walls and the steel or composite elements.

Reinforced concrete walls shall satisfy the requirements of ACI 318 excluding Chapter 18, except as modified in this section.

3. Analysis

Analysis shall satisfy the requirements of Chapter C as modified in this section.

(a) Uncracked effective stiffness values for elastic analysis shall be assigned in accordance with ACI 318 Chapter 6 for wall piers and composite coupling beams.

(b) When concrete-encased shapes function as boundary members, the analysis shall be based upon a transformed concrete section using elastic material properties.

4. System Requirements

In coupled walls, it is permitted to redistribute coupling beam forces vertically to adjacent floors. The shear in any individual coupling beam shall not be reduced by more than 20% of the elastically determined value. The sum of the coupling beam shear resistance over the height of the building shall be greater than or equal to the sum of the elastically determined values.

5. Members

5a. Boundary Members

Boundary members shall satisfy the following requirements:

(a) 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.

(b) When the concrete-encased structural steel boundary member qualifies as a composite column as defined in *Specification* Chapter I, it shall be designed as a composite column to satisfy the requirements of Chapter I of the *Specification*.

(c) Headed studs or welded reinforcement anchors shall be provided to transfer required shear strengths between the structural steel boundary members and reinforced concrete walls. Headed studs, if used, shall satisfy the requirements of *Specification* Chapter I. Welded reinforcement anchors, if used, shall satisfy the requirements of *Structural Welding Code—Reinforcing Steel* (AWS D1.4/D1.4M).

5b. Coupling Beams

1. Structural Steel Coupling Beams

Structural steel coupling beams that are used between adjacent reinforced concrete walls shall satisfy the requirements of the *Specification* and this section. The following requirements apply to wide-flange steel coupling beams.

(a) Steel coupling beams shall be designed in accordance with Chapters F and G of the *Specification*.

(b) The available connection shear strength, $\phi V_{n,connection}$, shall be computed from Equations H4-1 and H4-1M, with $\phi = 0.90$.

$$V_{n,connection} = 1.54\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left(\frac{0.58-0.22\beta_1}{0.88+\frac{g}{2L_e}}\right) \qquad \text{(H4-1)}$$

$$V_{n,connection} = 4.04\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left(\frac{0.58-0.22\beta_1}{0.88+\frac{g}{2L_e}}\right) \quad \text{(H4-1M)}$$

where

L_e = embedment length of coupling beam measured from the face of the wall, in. (mm)
b_w = thickness of wall pier, in. (mm)
b_f = width of beam flange, in. (mm)
f_c' = specified compressive strength of concrete, ksi (MPa)
β_1 = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, as defined in ACI 318
g = clear span of coupling beam, in. (mm)

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

2. Composite Coupling Beams

Encased composite sections serving as coupling beams shall satisfy the following requirements:

(a) Coupling beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the required shear strength, where the connection strength is calculated with Equation H4-1 or H4-1M.

The available shear strength of the composite beam, $\phi V_{n,comp}$, is computed from Equation H4-2 and H4-2M, with $\phi = 0.90$.

$$V_{n,comp} = V_p + \left(0.0632\sqrt{f_c'}\, b_{wc} d_c + \frac{A_{sr} F_{ysr} d_c}{s}\right) \quad \text{(H4-2)}$$

$$V_{n,comp} = V_p + \left(0.166\sqrt{f_c'}\, b_{wc} d_c + \frac{A_{sr} F_{ysr} d_c}{s}\right) \quad \text{(H4-2M)}$$

where

A_{sr} = area of transverse reinforcement, in.^{2} (mm^2)
F_{ysr} = specified minimum yield stress of transverse reinforcement, ksi (MPa)
V_p = $0.6F_yA_w$, kips (N)
A_w = area of steel beam web, in.^{2} (mm^2)

b_{wc} = width of concrete encasement, in. (mm)
d_c = effective depth of concrete encasement, in. (mm)
s = spacing of transverse reinforcement, in. (mm)

5c. Protected Zones

There are no designated protected zones.

6. Connections

There are no additional requirements beyond Section H4.5.

6a. Demand Critical Welds

There are no requirements specific to this system.

H5. COMPOSITE SPECIAL SHEAR WALLS (C-SSW)

1. Scope

Composite special shear walls (C-SSW) shall be designed in conformance with this section. This section is applicable when reinforced concrete walls are composite with structural steel elements, including structural steel or composite sections acting as boundary members for the walls and structural steel or composite coupling beams that connect two or more adjacent reinforced concrete walls.

2. Basis of Design

C-SSW designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through yielding in the reinforced concrete walls and the steel or composite elements. Reinforced concrete wall elements shall be designed to provide inelastic deformations at the design story drift consistent with ACI 318 including Chapter 18. Structural steel and composite coupling beams shall be designed to provide inelastic deformations at the design story drift through yielding in flexure or shear. Coupling beam connections and the design of the walls shall be designed to account for the expected strength including strain hardening in the coupling beams. Structural steel and composite boundary elements shall be designed to provide inelastic deformations at the design story drift through yielding due to axial force.

C-SSW systems shall satisfy the requirements of Section H4 and the shear wall requirements of ACI 318 including Chapter 18, except as modified in this section.

User Note: Steel coupling beams can be proportioned to be shear-critical or flexural-critical. Coupling beams with lengths $g \le 1.6M_p/V_p$ can be assumed to be shear-critical, where g, M_p and V_p are defined in Section H4.5b.1. Coupling beams with lengths $g \ge 2.6M_p/V_p$ may be considered to be flexure-critical. Coupling beam lengths between these two values are considered to yield in flexure and shear simultaneously.

3. Analysis

Analysis requirements of Section H4.3 shall be met with the following exceptions:

(a) Cracked effective stiffness values for elastic analysis shall be assigned in accordance with ACI 318 Chapter 6 practice for wall piers and composite coupling beams.

(b) Effects of shear distortion of the steel coupling beam shall be taken into account.

4. System Requirements

In addition to the system requirements of Section H4.4, the following shall be satisfied:

(a) In coupled walls, coupling beams shall yield over the height of the structure followed by yielding at the base of the wall piers.

(b) In coupled walls, the axial design strength of the wall at the balanced condition, P_b, shall equal or exceed the total required compressive axial strength in a wall pier, computed as the sum of the required strengths attributed to the walls from the gravity load components of the lateral load combination plus the sum of the expected beam shear strengths increased by a factor of 1.1 to reflect the effects of strain hardening of all the coupling beams framing into the walls.

5. Members

5a. Ductile Elements

Welding on steel coupling beams is permitted for attachment of stiffeners, as required in Section F3.5b.4.

5b. Boundary Members

Unencased structural steel columns shall satisfy the requirements of Section D1.1 for highly ductile members and Section H4.5a(a).

In addition to the requirements of Sections H4.3(b) and H4.5a(b), the requirements in this section shall apply to walls with concrete-encased structural steel boundary members. Concrete-encased structural steel boundary members that qualify as composite columns in *Specification* Chapter I shall meet the highly ductile member requirements of Section D1.4b.2. Otherwise, such members shall be designed as composite compression members to satisfy the requirements of ACI 318, including the special seismic requirements for boundary members in ACI 318 Section 18.10.6. Transverse reinforcement for confinement of the composite boundary member shall extend a distance of $2h$ into the wall, where h is the overall depth of the boundary member in the plane of the wall.

Headed studs or welded reinforcing anchors shall be provided as specified in Section H4.5a(c).

Vertical wall reinforcement as specified in Section H4.5b.1(c) shall be confined by transverse reinforcement that meets the requirements for boundary members of ACI 318 Section 18.10.6.

5c. Steel Coupling Beams

The design and detailing of steel coupling beams shall satisfy the following:

(a) The embedment length, L_e, of the coupling beam shall be computed from Equations H5-1 and H5-1M.

$$V_n = 1.54\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left(\frac{0.58-0.22\beta_1}{0.88+\frac{g}{2L_e}}\right) \quad \text{(H5-1)}$$

$$V_n = 4.04\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left(\frac{0.58-0.22\beta_1}{0.88+\frac{g}{2L_e}}\right) \quad \text{(H5-1M)}$$

where

L_e = embedment length of coupling beam, considered to begin inside the first layer of confining reinforcement, nearest to the edge of the wall, in the wall boundary member, in, (mm)

g = clear span of the coupling beam plus the wall concrete cover at each end of the beam, in, (mm)

V_n = expected shear strength of a steel coupling beam computed from Equation H5-2, kips (N)

$$= \frac{2\left(1.1R_y\right)M_p}{g} \le \left(1.1R_y\right)V_p \quad \text{(H5-2)}$$

where

A_{tw} = area of steel beam web, in.^{2} (mm^2)

F_y = specified minimum yield stress, ksi (MPa)

$M_p = F_y Z$, kip-in. (N-mm)

$V_p = 0.6F_y A_{tw}$, kips (N)

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

(b) Structural steel coupling beams shall satisfy the requirements of Section F3.5b, except that for built-up cross sections, the flange-to-web welds are permitted to be made with two-sided fillet, partial-joint-penetration, or complete-joint-penetration groove welds that develop the expected strength of the beam. When required in Section F3.5b.4, the coupling beam rotation shall be assumed as a 0.08 rad link rotation unless a smaller value is justified by rational analysis of the inelastic deformations that are expected under the design story drift. Face bearing plates shall be provided on both sides of the coupling beams at the face

of the reinforced concrete wall. These plates shall meet the detailing requirements of Section F3.5b.4.

(c) Steel coupling beams shall comply with the requirements of Section D1.1 for highly ductile members. Flanges of coupling beams with I-shaped sections with $g \le 1.6M_p/V_p$ are permitted to satisfy the requirements for moderately ductile members.

(d) Embedded steel members shall be provided with two regions of vertical transfer reinforcement attached to both the top and bottom flanges of the embedded member. The first region shall be located to coincide with the location of longitudinal wall reinforcing bars closest to the face of the wall. The second region shall be placed a distance no less than $d/2$ from the termination of the embedment length. All transfer reinforcement bars shall be fully developed where they engage the coupling beam flanges. It is permitted to use straight, hooked or mechanical anchorage to provide development. It is permitted to use mechanical couplers welded to the flanges to attach the vertical transfer bars. The area of vertical transfer reinforcement required is computed by Equation H5-3:

$$A_{tb} \ge 0.03 f_c' L_e b_f / F_{ysr} \tag{H5-3}$$

where

A_{tb} = area of transfer reinforcement required in each of the first and second regions attached to each of the top and bottom flanges, in.^{2} (mm^2)
F_{ysr} = specified minimum yield stress of transfer reinforcement, ksi (MPa)
b_f = width of beam flange, in. (mm)
f_c' = specified compressive strength of concrete, ksi (MPa)

The area of vertical transfer reinforcement shall not exceed that computed by Equation H5-4:

$$\sum A_{tb} < 0.08 L_e b_w - A_{sr} \tag{H5-4}$$

where

$\sum A_{tb}$ = total area of transfer reinforcement provided in both the first and second regions attached to either the top or bottom flange, in.^{2} (mm^2)
A_{sr} = area of longitudinal wall reinforcement provided over the embedment length, L_e, in.^{2} (mm^2)
b_w = width of wall, in. (mm)

5d. Composite Coupling Beams

Encased composite sections serving as coupling beams shall satisfy the requirements of Section H5.5c, except the requirements of Section F3.5b.4 need not be met, and Equation H5-5 or H5-5M shall be used instead of Equation H4-2 or H4-2M. For all encased composite coupling beams, the limiting expected shear strength, V_{comp}, is:

$$V_{comp} = 1.1 R_y V_p + 0.08 \sqrt{R_c f_c'}\, b_{wc} d_c + \frac{1.33 R_{yr} A_s F_{ysr} d_c}{s} \tag{H5-5}$$

$$V_{comp} = 1.1R_yV_p + 0.21\sqrt{R_c f_c'}\, b_{wc}d_c + \frac{1.33R_{yr}A_sF_{ysr}d_c}{s} \qquad \text{(H5-5M)}$$

where

F_{ysr} = specified minimum yield stress of transverse reinforcement, ksi (MPa)

R_c = factor to account for expected strength of concrete = 1.5

R_{yr} = ratio of the expected yield stress of the transverse reinforcement material to the specified minimum yield stress, F_{ysr}

5e. Protected Zones

The clear span of the coupling beam between the faces of the shear walls shall be designated as a protected zone and shall satisfy the requirements of Section D1.3. Attachment of stiffeners, and face bearing plates as required by Section H5.5c(b), are permitted.

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds and shall meet the requirements of Section A3.4b and I2.3.

(a) Groove welds at column splices

(b) Welds at the column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are satisfied.

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

6b. Column Splices

Column splices shall be designed in accordance with the requirements of Section G2.6f.

H6. COMPOSITE PLATE SHEAR WALLS—CONCRETE ENCASED (C-PSW/CE)

1. Scope

Composite plate shear walls-concrete encased (C-PSW/CE) shall be designed in accordance with this section. C-PSW/CE consist of steel plates with reinforced concrete encasement on one or both sides of the plate and structural steel or composite boundary members.

2. Basis of Design

C-PSW/CE designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through yielding in the plate webs. The horizontal boundary elements (HBE) and vertical boundary elements (VBE) adjacent to the composite webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded steel webs along with the reinforced concrete webs after the steel web has fully yielded, except that plastic hinging at the ends of HBE is permitted.

3. Analysis

3a. Webs

The analysis shall account for openings in the web.

3b. Other Members and Connections

Columns, beams and connections in C-PSW/CE shall be designed to resist seismic forces determined from an analysis that includes the expected strength of the steel webs in shear, $0.6R_yF_yA_{sp}$, and any reinforced concrete portions of the wall active at the design story drift,
where

A_{sp} = horizontal area of the stiffened steel plate, in.^{2} (mm^2)
F_y = specified minimum yield stress, ksi (MPa)
R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y

The VBE are permitted to yield at the base.

4. System Requirements

4a. Steel Plate Thickness

Steel plates with thickness less than ⅜ in. (10 mm) are not permitted.

4b. Stiffness of Vertical Boundary Elements

The VBEs shall satisfy the requirements of Section F5.4a.

4c. HBE-to-VBE Connection Moment Ratio

The beam-column moment ratio shall satisfy the requirements of Section F5.4b.

4d. Bracing

HBE shall be braced to satisfy the requirements for moderately ductile members.

4e. Openings in Webs

Boundary members shall be provided around openings in shear wall webs as required by analysis.

5. Members

5a. Basic Requirements

Steel and composite HBE and VBE shall satisfy the requirements of Section D1.1 for highly ductile members.

5b. Webs

The design shear strength, $\phi V_{n,}$ for the limit state of shear yielding with a composite plate conforming to Section H6.5c, shall be:

$$V_n = 0.6A_{sp}F_y \quad \text{(H6-1)}$$

$$\phi = 0.90 \quad \text{(LRFD)}$$

where

F_y = specified minimum yield stress of the plate, ksi (MPa)
A_{sp} = horizontal area of the stiffened steel plate, in.^{2} (mm^2)

The available shear strength of C-PSW/CE with a plate that does not meet the stiffening requirements in Section H6.5c shall be based upon the strength of the plate determined in accordance with Section F5.5 and shall satisfy the requirements of *Specification* Section G2.

5c. Concrete Stiffening Elements

The steel plate shall be stiffened by encasement or attachment to a reinforced concrete panel. Conformance to this requirement shall be demonstrated with an elastic plate buckling analysis showing that the composite wall is able to resist a nominal shear force equal to V_n, as determined in Section H6.5b.

The concrete thickness shall be a minimum of 4 in. (100 mm) on each side when concrete is provided on both sides of the steel plate and 8 in. (200 mm) when concrete is provided on one side of the steel plate. Steel headed stud anchors or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and reinforced concrete. Horizontal and vertical reinforcement shall be provided in the concrete encasement to meet or exceed the requirements in ACI 318 Sections 11.6 and 11.7. The reinforcement ratio in both directions shall not be less than 0.0025. The maximum spacing between bars shall not exceed 18 in. (450 mm).

5d. Boundary Members

Structural steel and composite boundary members shall be designed to resist the expected shear strength of steel plate and any reinforced concrete portions of the wall active at the design story drift. Composite and reinforced concrete boundary members shall also satisfy the requirements of Section H5.5b. Steel boundary members shall also satisfy the requirements of Section F5.

5e. Protected Zones

There are no designated protected zones.

6. Connections

6a. Demand Critical Welds

The following welds are demand critical welds and shall satisfy the requirements of Section A3.4b and I2.3:

(a) Groove welds at column splices

(b) Welds at the column-to-base plate connections

Exception: Welds need not be considered demand critical when both of the following conditions are met.

(1) Column hinging at, or near, the base plate is precluded by conditions of restraint.

(2) There is no net tension under load combinations including the overstrength seismic load.

(c) Welds at HBE-to-VBE connections

6b. HBE-to-VBE Connections

HBE-to-VBE connections shall satisfy the requirements of Section F5.6b.

6c. Connections of Steel Plate to Boundary Elements

The steel plate shall be continuously welded or bolted on all edges to the structural steel framing and/or steel boundary members, or the steel component of the composite boundary members. Welds and/or slip-critical high-strength bolts required to develop the nominal shear strength of the plate shall be provided.

6d. Connections of Steel Plate to Reinforced Concrete Panel

The steel anchors between the steel plate and the reinforced concrete panel shall be designed to prevent its overall buckling. Steel anchors shall be designed to satisfy the following conditions:

1. Tension in the Connector

The steel anchor shall be designed to resist the tension force resulting from inelastic local buckling of the steel plate.

2. Shear in the Connector

The steel anchors collectively shall be designed to transfer the expected strength in shear of the steel plate or reinforced concrete panel, whichever is smaller.

6e. Column Splices

In addition to the requirements of Section D2.5, column splices shall comply with the requirements of this section. Where welds are used to make the splice, they shall

be complete-joint-penetration groove welds. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to the plastic flexural strength, M_{pcc}, of the smaller composite column. The required shear strength of column web splices shall be at least equal to $\sum M_{pcc}/H$, where $\sum M_{pcc}$ is the sum of the plastic flexural strengths at the top and bottom ends of the composite column and H is the height of story. For composite columns, the plastic flexural strength shall satisfy the requirements of *Specification* Chapter I with consideration of the required axial strength, P_{rc}.

H7. COMPOSITE PLATE SHEAR WALLS—CONCRETE FILLED (C-PSW/CF)

1. Scope

Composite plate shear walls-concrete filled (C-PSW/CF) shall be designed in conformance with this section. This section is applicable to composite plate shear walls that consist of two planar steel web plates with concrete fill between the plates, with or without boundary elements. Composite action between the plates and concrete fill shall be achieved using either tie bars or a combination of tie bars and shear studs. The two steel web plates shall be of equal thickness and shall be placed at a constant distance from each other and connected using tie bars. When boundary members are included, they shall be either a half circular section of diameter equal to the distance between the two web plates or a circular concrete-filled steel tube.

2. Basis of Design

C-PSW/CF with boundary elements, designed in accordance with these provisions, are expected to provide significant inelastic deformation capacity through developing plastic moment strength of the composite C-PSW/CF cross section, by yielding of the entire skin plate and the concrete attaining its compressive strength. The cross section shall be detailed such that it is able to attain its plastic moment strength. Shear yielding of the steel web skin plates shall not be the governing mechanism.

C-PSW/CF without boundary elements designed in accordance to these provisions are expected to provide inelastic deformation capacity by developing yield moment strength of the composite C-PSW/CF cross section, by flexural tension yielding of the steel plates. The walls shall be detailed such that flexural compression yielding occurs before local buckling of the steel plates.

3. Analysis

Analysis shall satisfy the following:

(a) Effective flexural stiffness of the wall shall be calculated per *Specification* Equation I2-12, with C_3 taken equal to 0.40.

(b) The shear stiffness of the wall shall be calculated using the shear stiffness of the composite cross section.

4. System Requirements

4a. Steel Web Plate of C-PSW/CF with Boundary Elements

The maximum spacing of tie bars in vertical and horizontal directions, w_1, shall be:

$$w_1 = 1.8t\sqrt{\frac{E}{F_y}} \tag{H7-1}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_y = specified minimum yield stress, ksi (MPa)
t = thickness of the steel web plate, in. (mm)

When tie bars are welded with the web plate, the thickness of the plate shall develop the tension strength of the tie bars.

4b. Steel Plate of C-PSW/CF without Boundary Elements

The maximum spacing of tie bars in vertical and horizontal directions, w_1, shall be:

$$w_1 = 1.0t\sqrt{\frac{E}{F_y}} \tag{H7-2}$$

where

t = thickness of the steel web plate, in. (mm)

4c. Half Circular or Full Circular End of C-PSW/CF with Boundary Elements

The D/t_{HSS} ratio for the circular part of the C-PSW/CF cross section shall conform to:

$$\frac{D}{t_{HSS}} \le 0.044\frac{E}{F_y} \tag{H7-3}$$

where

D = outside diameter of round HSS, in. (mm)
t_{HSS} = thickness of HSS, in. (mm)

4d. Spacing of Tie Bars in C-PSW/CF with or without Boundary Elements

Tie bars shall be distributed in both vertical and horizontal directions, as specified in Equations H7-1 and H7-2.

4e. Tie Bar Diameter in C-PSW/CF with or without Boundary Elements

Tie bars shall be designed to elastically resist the tension force, T_{req}, determined as follows:

$$T_{req} = T_1 + T_2 \tag{H7-4}$$

T_1 is the tension force resulting from the locally buckled web plates developing plastic hinges on horizontal yield lines along the tie bars and at mid-vertical distance between tie-bars, and is determined as follows:

$$T_1 = 2\left(\frac{w_2}{w_1}\right)t_s^2 F_y \tag{H7-5}$$

where

t_s = thickness of steel web plate provided, in. (mm)
w_1, w_2 = vertical and horizontal spacing of tie bars, respectively, in. (mm)

T_2 is the tension force that develops to prevent splitting of the concrete element on a plane parallel to the steel plate.

$$T_2 = \left(\frac{t_s F_{y,plate} t_w}{4}\right)\left(\frac{w_2}{w_1}\right)\left[\frac{6}{18\left(\frac{t_w}{w_{min}}\right)^2 + 1}\right] \tag{H7-6}$$

where

t_w = total thickness of wall, in. (mm)
w_{min} = minimum of w_1 and w_2, in. (mm)

4f. Connection between Tie Bars and Steel Plates

Connection of the tie bars to the steel plate shall be able to develop the full tension strength of the tie bar.

4g. Connection between C-PSW/CF Steel Components

Welds between the steel web plate and the half-circular or full-circular ends of the cross section shall be complete-joint-penetration groove welds.

4h. C-PSW/CF and Foundation Connection

The connection between C-PSW/CF and the foundation shall be detailed such that the connection is able to transfer the base shear force and the axial force acting together with the overturning moment, corresponding to 1.1 times the plastic composite flexural strength of the wall, where the plastic flexural composite strength is obtained by the plastic stress distribution method described in *Specification* Section I1.2a assuming that the steel components have reached a stress equal to the expected yield strength, $R_y F_y$, in either tension or compression and that concrete components in compression due to axial force and flexure have reached a stress of f'_c.

5. Members

5a. Flexural Strength

The nominal plastic moment strength of the C-PSW/CF with boundary elements shall be calculated considering that all the concrete in compression has reached its specified compressive strength, f'_c, and that the steel in tension and compression has

reached its specified minimum yield strength, F_y, as determined based on the location of the plastic neutral axis.

The nominal moment strength of the C-PSW/CF without boundary elements shall be calculated as the yield moment, M_y, corresponding to yielding of the steel plate in flexural tension and first yield in flexural compression. The strength at first yield shall be calculated assuming a linear elastic stress distribution with maximum concrete compressive stress limited to $0.7f_c'$ and maximum steel stress limited to F_y.

User Note: The definition and calculation of the yield moment, M_y, for C-PSW/CF without boundary elements is very similar to the definition and calculation of yield moment, M_y, for noncompact filled composite members in *Specification* Section I3.4b(b).

5b. Shear Strength

The available shear strength of C-PSW/CF shall be determined as follows:

(a) The design shear strength, ϕV_{ni}, of the C-PSW/CF with boundary elements shall be determined as follows:

$$V_{ni} = \kappa F_y A_{sw} \tag{H7-7}$$

$$\phi = 0.90 \text{ (LRFD)}$$

where

$$\kappa = 1.11 - 5.16\overline{\rho} \le 1.0 \tag{H7-8}$$

$\overline{\rho}$ = strength adjusted reinforcement ratio

$$= \frac{A_{sw}F_{yw}}{A_{cw}\sqrt{1{,}000f_c'}} \tag{H7-9}$$

$$= \frac{1}{12}\frac{A_{sw}F_{yw}}{A_{cw}\sqrt{f_c'}} \tag{H7-9M}$$

A_{sw} = area of steel web plates, in.^{2} (mm^2)
A_{cw} = area of concrete between web plates, in.^{2} (mm^2)
F_{yw} = specified minimum yield stress of web skin plates, ksi (MPa)
f_c' = specified compressive strength of concrete, ksi (MPa)

User Note: For most cases, $0.9 \le \kappa \le 1.0$.

(b) The nominal shear strength of the C-PSW/CF without boundary elements shall be calculated for the steel plates alone, in accordance with Section D1.4c.

CHAPTER I
FABRICATION AND ERECTION

This chapter addresses requirements for fabrication and erection.

User Note: All requirements of *Specification* Chapter M also apply, unless specifically modified by these Provisions.

The chapter is organized as follows:

I1. Shop and Erection Drawings
I2. Fabrication and Erection

I1. SHOP AND ERECTION DRAWINGS

1. Shop Drawings for Steel Construction

Shop drawings shall indicate the work to be performed, and include items required by the *Specification*, the AISC *Code of Standard Practice for Steel Buildings and Bridges*, the applicable building code, the requirements of Sections A4.1 and A4.2, and the following, as applicable:

(a) Locations of pretensioned bolts

(b) Locations of Class A, or higher, faying surfaces

(c) Gusset plates drawn to scale when they are designed to accommodate inelastic rotation

(d) Weld access hole dimensions, surface profile and finish requirements

(e) Nondestructive testing (NDT) where performed by the fabricator

2. Erection Drawings for Steel Construction

Erection drawings shall indicate the work to be performed, and include items required by the *Specification*, the AISC *Code of Standard Practice for Steel Buildings and Bridges*, the applicable building code, the requirements of Sections A4.1 and A4.2, and the following, as applicable:

(a) Locations of pretensioned bolts

(b) Those joints or groups of joints in which a specific assembly order, welding sequence, welding technique or other special precautions are required

3. Shop and Erection Drawings for Composite Construction

Shop drawings and erection drawings for the steel components of composite steel-concrete construction shall satisfy the requirements of Sections I1.1 and I1.2. The

shop drawings and erection drawings shall also satisfy the requirements of Section A4.3.

User Note: For reinforced concrete and composite steel-concrete construction, the provisions of ACI 315 *Details and Detailing of Concrete Reinforcement* and ACI 315R *Manual of Engineering and Placing Drawings for Reinforced Concrete Structures* apply.

I2. FABRICATION AND ERECTION

1. Protected Zone

A protected zone designated by these Provisions or ANSI/AISC 358 shall comply with the following requirements:

(a) Within the protected zone, holes, tack welds, erection aids, air-arc gouging, and unspecified thermal cutting from fabrication or erection operations shall be repaired as required by the engineer of record.

(b) Steel headed stud anchors shall not be placed on beam flanges within the protected zone.

(c) Arc spot welds as required to attach decking are permitted.

(d) Decking attachments that penetrate the beam flange shall not be placed on beam flanges within the protected zone, except power-actuated fasteners up to 0.18 in. diameter are permitted.

(e) Welded, bolted, or screwed attachments or power-actuated fasteners for perimeter edge angles, exterior facades, partitions, duct work, piping or other construction shall not be placed within the protected zone.

Exception: Other attachments are permitted where designated or approved by the engineer of record. See Section D1.3.

User Note: AWS D1.8/D1.8M clause 6.18 contains requirements for weld removal and the repair of gouges and notches in the protected zone.

2. Bolted Joints

Bolted joints shall satisfy the requirements of Section D2.2.

3. Welded Joints

Welding and welded connections shall be in accordance with AWS D1.8/D1.8M and *Structural Welding Code—Steel* (AWS D1.1/D1.1M), hereafter referred to as AWS D1.1/D1.1M.

Welding procedure specifications (WPS) shall be approved by the engineer of record.

Weld tabs shall be in accordance with AWS D1.8/D1.8M clause 6.10, except at the outboard ends of continuity-plate-to-column welds, weld tabs and weld metal need not be removed closer than ¼ in. (6 mm) from the continuity plate edge.

AWS D1.8/D1.8M clauses relating to fabrication shall apply equally to shop fabrication welding and to field erection welding.

User Note: AWS D1.8/D1.8M was specifically written to provide additional requirements for the welding of seismic force-resisting systems, and has been coordinated wherever possible with these Provisions. AWS D1.8/D1.8M requirements related to fabrication and erection are organized as follows, including normative (mandatory) annexes:

1. General Requirements
2. Normative References
3. Terms and Definitions
4. Welded Connection Details
5. Welder Qualification
6. Fabrication

Annex A. WPS Heat Input Envelope Testing of Filler Metals for Demand Critical Welds
Annex B. Intermix CVN Testing of Filler Metal Combinations (where one of the filler metals is FCAW-S)
Annex D. Supplemental Welder Qualification for Restricted Access Welding
Annex E. Supplemental Testing for Extended Exposure Limits for FCAW Filler Metals

At continuity plates, these Provisions permit a limited amount of weld tab material to remain because of the reduced strains at continuity plates, and any remaining weld discontinuities in this weld end region would likely be of little significance. Also, weld tab removal sites at continuity plates are not subjected to MT.

AWS D1.8/D1.8M clause 6 is entitled "Fabrication," but the intent of AWS is that all provisions of AWS D1.8/D1.8M apply equally to fabrication and erection activities as described in the *Specification* and in these Provisions.

4. Continuity Plates and Stiffeners

Corners of continuity plates and stiffeners placed in the webs of rolled shapes shall be detailed in accordance with AWS D1.8/D1.8M clause 4.1.

CHAPTER J

QUALITY CONTROL AND QUALITY ASSURANCE

This chapter addresses requirements for quality control and quality assurance.

User Note: All requirements of *Specification* Chapter N also apply, unless specifically modified by these Provisions.

The chapter is organized as follows:

J1. Scope
J2. Fabricator and Erector Documents
J3. Quality Assurance Agency Documents
J4. Inspection and Nondestructive Testing Personnel
J5. Inspection Tasks
J6. Welding Inspection and Nondestructive Testing
J7. Inspection of High-Strength Bolting
J8. Other Steel Structure Inspections
J9. Inspection of Composite Structures
J10. Inspection of Piling

J1. SCOPE

Quality Control (QC) as specified in this chapter shall be provided by the fabricator, erector or other responsible contractor as applicable. Quality Assurance (QA) as specified in this chapter shall be provided by others when required by the authority having jurisdiction (AHJ), applicable building code (ABC), purchaser, owner or engineer of record (EOR). Nondestructive testing (NDT) shall be performed by the agency or firm responsible for Quality Assurance, except as permitted in accordance with *Specification* Section N6.

User Note: The quality assurance plan of this section is considered adequate and effective for most seismic force-resisting systems and should be used without modification. The quality assurance plan is intended to ensure that the seismic force resisting system is significantly free of defects that would greatly reduce the ductility of the system. There may be cases (for example, nonredundant major transfer members, or where work is performed in a location that is difficult to access) where supplemental testing might be advisable. Additionally, where the fabricator's or erector's quality control program has demonstrated the capability to perform some tasks this plan has assigned to quality assurance, modification of the plan could be considered.

J2. FABRICATOR AND ERECTOR DOCUMENTS

1. Documents to be Submitted for Steel Construction

In addition to the requirements of *Specification* Section N3.1, the following documents shall be submitted for review by the EOR or the EOR's designee, prior to fabrication or erection of the affected work, as applicable:

(a) Welding procedure specifications (WPS)

(b) Copies of the manufacturer's typical certificate of conformance for all electrodes, fluxes and shielding gasses to be used

(c) For demand critical welds, applicable manufacturer's certifications that the filler metal meets the supplemental notch toughness requirements, as applicable. When the filler metal manufacturer does not supply such supplemental certifications, the fabricator or erector, as applicable, shall have the necessary testing performed and provide the applicable test reports

(d) Manufacturer's product data sheets or catalog data for shielded metal arc welding (SMAW), flux cored arc welding (FCAW), and gas metal arc welding (GMAW) composite (cored) filler metals to be used

(e) Bolt installation procedures

(f) Specific assembly order, welding sequence, welding technique, or other special precautions for joints or groups of joints where such items are designated to be submitted to the engineer of record

2. Documents to be Available for Review for Steel Construction

Additional documents as required by the EOR in the contract documents shall be available by the fabricator and erector for review by the EOR or the EOR's designee prior to fabrication or erection, as applicable.

The fabricator and erector shall retain their document(s) for at least one year after substantial completion of construction.

3. Documents to be Submitted for Composite Construction

The following documents shall be submitted by the responsible contractor for review by the EOR or the EOR's designee, prior to concrete production or placement, as applicable:

(a) Concrete mix design and test reports for the mix design

(b) Reinforcing steel shop drawings

(c) Concrete placement sequences, techniques and restriction

4. Documents to be Available for Review for Composite Construction

The following documents shall be available from the responsible contractor for review by the EOR or the EOR's designee prior to fabrication or erection, as applicable, unless specified to be submitted:

(a) Material test reports for reinforcing steel

(b) Inspection procedures

(c) Nonconformance procedure

(d) Material control procedure

(e) Welder performance qualification records (WPQR) as required by S*tructural Welding Code—Reinforcing Steel* (AWS D1.4/D1.4M)

(f) QC Inspector qualifications

The responsible contractor shall retain their document(s) for at least one year after substantial completion of construction.

J3. QUALITY ASSURANCE AGENCY DOCUMENTS

The agency responsible for quality assurance shall submit the following documents to the authority having jurisdiction, the EOR, and the owner or owner's designee:

(a) QA agency's written practices for the monitoring and control of the agency's operations. The written practice shall include:

(1) The agency's procedures for the selection and administration of inspection personnel, describing the training, experience and examination requirements for qualification and certification of inspection personnel; and

(2) The agency's inspection procedures, including general inspection, material controls, and visual welding inspection

(b) Qualifications of management and QA personnel designated for the project

(c) Qualification records for inspectors and NDT technicians designated for the project

(d) NDT procedures and equipment calibration records for NDT to be performed and equipment to be used for the project

(e) For composite construction, concrete testing procedures and equipment

J4. INSPECTION AND NONDESTRUCTIVE TESTING PERSONNEL

In addition to the requirements of *Specification* Sections N4.1 and N4.2, visual welding inspection and NDT shall be conducted by personnel qualified in accordance with AWS D1.8/D1.8M clause 7.2. In addition to the requirements of *Specification* Section N4.3, ultrasonic testing technicians shall be qualified in accordance with AWS D1.8/D1.8M clause 7.2.4.

User Note: The recommendations of the International Code Council *Model Program for Special Inspection* should be considered a minimum requirement to establish the qualifications of a bolting inspector.

J5. INSPECTION TASKS

Inspection tasks and documentation for QC and QA for the seismic force-resisting system (SFRS) shall be as provided in the tables in Sections J6, J7, J8, J9 and J10. The following entries are used in the tables:

1. Observe (O)

The inspector shall observe these functions on a random, daily basis. Operations need not be delayed pending observations.

2. Perform (P)

These inspections shall be performed prior to the final acceptance of the item.

3. Document (D)

The inspector shall prepare reports indicating that the work has been performed in accordance with the contract documents. The report need not provide detailed measurements for joint fit-up, WPS settings, completed welds, or other individual items listed in the tables. For shop fabrication, the report shall indicate the piece mark of the piece inspected. For field work, the report shall indicate the reference grid lines and floor or elevation inspected. Work not in compliance with the contract documents and whether the noncompliance has been satisfactorily repaired shall be noted in the inspection report.

4. Coordinated Inspection

Where a task is stipulated to be performed by both QC and QA, coordination of the inspection function between QC and QA is permitted in accordance with *Specification* Section N5.3.

J6. WELDING INSPECTION AND NONDESTRUCTIVE TESTING

Welding inspection and nondestructive testing shall satisfy the requirements of the *Specification*, this section and AWS D1.8/D1.8M.

User Note: AWS D1.8/D1.8M was specifically written to provide additional requirements for the welding of seismic force resisting systems, and has been coordinated when possible with these Provisions. AWS D1.8/D1.8M requirements related to inspection and nondestructive testing are organized as follows, including normative (mandatory) annexes:

1. General Requirements
7. Inspection

Annex F. Supplemental Ultrasonic Technician Testing
Annex G. Supplemental Magnetic Particle Testing Procedures
Annex H. Flaw Sizing by Ultrasonic Testing

1. Visual Welding Inspection

All requirements of the *Specification* shall apply, except as specifically modified by AWS D1.8/D1.8M.

Visual welding inspection shall be performed by both quality control and quality assurance personnel. As a minimum, tasks shall be as listed in Tables J6.1, J6.2 and J6.3.

2. NDT of Welded Joints

In addition to the requirements of *Specification* Section N5.5, nondestructive testing of welded joints shall be as required in this section.

2a. CJP Groove Weld NDT

Ultrasonic testing (UT) shall be performed on 100% of complete-joint-penetration (CJP) groove welds in materials 5/16 in. (8 mm) thick or greater. UT in materials less than 5/16 in. (8 mm) thick is not required. Weld discontinuities shall be accepted or rejected on the basis of AWS D1.1/D1.1M Table 6.2. Magnetic particle testing (MT) shall be performed on 25% of all beam-to-column CJP groove welds. The rate of UT and MT is permitted to be reduced in accordance with Sections J6.2g and J6.2h, respectively.

Exception: For ordinary moment frames in structures in risk categories I or II, UT and MT of CJP groove welds are required only for demand critical welds.

User Note: For structures in risk category III or IV, *Specification* Section N5.5b requires that the UT be performed by QA on all CJP groove welds subject to transversely applied tension loading in butt, T- and corner joints, in material 5/16 in. (8 mm) thick or greater.

2b. Column Splice and Column to Base Plate PJP Groove Weld NDT

UT shall be performed by QA on 100% of partial-joint-penetration (PJP) groove welds in column splices and column to base plate welds. The rate of UT is permitted to be reduced in accordance with Section J6.2g.

UT shall be performed using written procedures and UT technicians qualified in accordance with AWS D1.8/D1.8M. The weld joint mock-ups used to qualify procedures and technicians shall include at least one single-bevel PJP groove welded joint and one double-bevel PJP groove welded joint, detailed to provide transducer access limitations similar to those to be encountered at the weld faces and by the column web. Rejection of discontinuities outside the groove weld throat shall be considered false indications in procedure and personnel qualification. Procedures qualified using mock-ups with artificial flaws 1/16 in. (1.5 mm) in their smallest dimension are permitted.

TABLE J6.1
Visual Inspection Tasks Prior to Welding

Visual Inspection Tasks Prior to Welding	QC		QA	
	Task	Doc.	Task	Doc.
Material identification (Type/Grade)	O	—	O	—
Welder identification system	O	—	O	—
Fit-up of Groove Welds (including joint geometry) –Joint preparation –Dimensions (alignment, root opening, root face, bevel) –Cleanliness (condition of steel surfaces) –Tacking (tack weld quality and location) –Backing type and fit (if applicable)	P/O**	—	O	—
Configuration and finish of access holes	O	—	O	—
Fit-up of Fillet Welds –Dimensions (alignment, gaps at root) –Cleanliness (condition of steel surfaces) –Tacking (tack weld quality and location)	P/O**	—	O	—

** Following performance of this inspection task for ten welds to be made by a given welder, with the welder demonstrating understanding of requirements and possession of skills and tools to verify these items, the Perform designation of this task shall be reduced to Observe, and the welder shall perform this task. Should the inspector determine that the welder has discontinued performance of this task, the task shall be returned to Perform until such time as the Inspector has re-established adequate assurance that the welder will perform the inspection tasks listed.

TABLE J6.2
Visual Inspection Tasks During Welding

Visual Inspection Tasks During Welding	QC		QA	
	Task	Doc.	Task	Doc.
WPS followed –Settings on welding equipment –Travel speed –Selected welding materials –Shielding gas type/flow rate –Preheat applied –Interpass temperature maintained (min/max.) –Proper position (F, V, H, OH) –Intermix of filler metals avoided unless approved	O	—	O	—
Use of qualified welders	O	—	O	—
Control and handling of welding consumables –Packaging –Exposure control	O	—	O	—
Environmental conditions –Wind speed within limits –Precipitation and temperature	O	—	O	—
Welding techniques –Interpass and final cleaning –Each pass within profile limitations –Each pass meets quality requirements	O	—	O	—
No welding over cracked tacks	O	—	O	—

TABLE J6.3
Visual Inspection Tasks After Welding

Visual Inspection Tasks After Welding	QC		QA	
	Task	Doc.	Task	Doc.
Welds cleaned	O	—	O	—
Size, length, and location of welds	P	—	P	—
Welds meet visual acceptance criteria –Crack prohibition –Weld/base-metal fusion –Crater cross section –Weld profiles and size –Undercut –Porosity	P	D	P	D
k-area[1]	P	D	P	D
Placement of reinforcing or contouring fillet welds (if required)	P	D	P	D
Backing removed, weld tabs removed and finished, and fillet welds added (if required)	P	D	P	D
Repair activities	P	—	P	D

[1] When welding of doubler plates, continuity plates or stiffeners has been performed in the *k*-area, visually inspect the web *k*-area for cracks within 3 in. (75 mm) of the weld. The visual inspection shall be performed no sooner than 48 hours following completion of the welding.

UT examination of welds using alternative techniques in compliance with AWS D1.1/D1.1M Annex Q is permitted.

Weld discontinuities located within the groove weld throat shall be accepted or rejected on the basis of criteria of AWS D1.1/D1.1M Table 6.2, except when alternative techniques are used, the criteria shall be as provided in AWS D1.1/D1.1M Annex Q.

2c. Base Metal NDT for Lamellar Tearing and Laminations

After joint completion, base metal thicker than 1½ in. (38 mm) loaded in tension in the through-thickness direction in T- and corner-joints, where the connected material is greater than ¾ in. (19 mm) and contains CJP groove welds, shall be ultrasonically tested for discontinuities behind and adjacent to the fusion line of such welds. Any base metal discontinuities found within $t/4$ of the steel surface shall be accepted or rejected on the basis of criteria of AWS D1.1/D1.1M Table 6.2, where t is the thickness of the part subjected to the through-thickness strain.

2d. Beam Cope and Access Hole NDT

At welded splices and connections, thermally cut surfaces of beam copes and access holes shall be tested using magnetic particle testing or penetrant testing, when the flange thickness exceeds 1½ in. (38 mm) for rolled shapes, or when the web thickness exceeds 1½ in. (38 mm) for built-up shapes.

2e. Reduced Beam Section Repair NDT

MT shall be performed on any weld and adjacent area of the reduced beam section (RBS) cut surface that has been repaired by welding, or on the base metal of the RBS cut surface if a sharp notch has been removed by grinding.

2f. Weld Tab Removal Sites

At the end of welds where weld tabs have been removed, MT shall be performed on the same beam-to-column joints receiving UT as required under Section J6.2a. The rate of MT is permitted to be reduced in accordance with Section J6.2h. MT of continuity plate weld tab removal sites is not required.

2g. Reduction of Percentage of Ultrasonic Testing

The reduction of percentage of UT is permitted to be reduced in accordance with *Specification* Section N5.5e, except no reduction is permitted for demand critical welds.

2h. Reduction of Percentage of Magnetic Particle Testing

The amount of MT on CJP groove welds is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The MT rate for an individual welder or welding operator is permitted to be reduced to 10%, provided the reject rate is demonstrated to be 5% or less of the welds tested for the welder or welding operator. A sampling of at least 20 completed welds for a job shall be made for such reduction evaluation. Reject rate is the number of welds containing rejectable defects divided by the number of welds completed. This reduction is prohibited on welds in the *k*-area, at repair sites, backing removal sites, and access holes.

J7. INSPECTION OF HIGH-STRENGTH BOLTING

Bolting inspection shall satisfy the requirements of *Specification* Section N5.6 and this section. Bolting inspection shall be performed by both quality control and quality assurance personnel. As a minimum, the tasks shall be as listed in Tables J7.1, J7.2 and J7.3.

J8. OTHER STEEL STRUCTURE INSPECTIONS

Other inspections of the steel structure shall satisfy the requirements of *Specification* Section N5.8 and this section. Such inspections shall be performed by both quality control and quality assurance personnel. Where applicable, the inspection tasks listed in Table J8.1 shall be performed.

User Note: The protected zone should be inspected by others following completion of the work of other trades, including those involving curtainwall, mechanical, electrical, plumbing and interior partitions. See Section A4.1.

TABLE J7.1
Inspection Tasks Prior To Bolting

Inspection Tasks Prior To Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Proper fasteners selected for the joint detail	O	—	O	—
Proper bolting procedure selected for joint detail	O	—	O	—
Connecting elements, including the faying surface condition and hole preparation, if specified, meet applicable requirements	O	—	O	—
Pre-installation verification testing by installation personnel observed for fastener assemblies and methods used	P	D	O	D
Proper storage provided for bolts, nuts, washers and other fastener components	O	—	O	—

TABLE J7.2
Inspection Tasks During Bolting

Inspection Tasks During Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Fastener assemblies placed in all holes and washers (if required) are positioned as required	O	—	O	—
Joint brought to the snug tight condition prior to the pretensioning operation	O	—	O	—
Fastener component not turned by the wrench prevented from rotating	O	—	O	—
Bolts are pretensioned progressing systematically from the most rigid point toward the free edges	O	—	O	—

TABLE J7.3
Inspection Tasks After Bolting

Inspection Tasks After Bolting	QC		QA	
	Task	Doc.	Task	Doc.
Document accepted and rejected connections	P	D	P	D

TABLE J8.1
Other Inspection Tasks

Other Inspection Tasks	QC		QA	
	Task	Doc	Task	Doc.
RBS requirements, if applicable –Contour and finish –Dimensional tolerances	P	D	P	D
Protected zone—no holes and unapproved attachments made by fabricator or erector, as applicable	P	D	P	D

J9. INSPECTION OF COMPOSITE STRUCTURES

Where applicable, inspection of composite structures shall satisfy the requirements of the *Specification* and this section. These inspections shall be performed by the responsible contractor's quality control personnel and by quality assurance personnel.

Where applicable, inspection of structural steel elements used in composite structures shall comply with the requirements of this Chapter. Where applicable, inspection of reinforced concrete shall comply with the requirements of ACI 318, and inspection of welded reinforcing steel shall comply with the applicable requirements of Section J6.1.

Where applicable to the type of composite construction, the minimum inspection tasks shall be as listed in Tables J9.1, J9.2 and J9.3.

J10. INSPECTION OF H-PILES

Where applicable, inspection of piling shall satisfy the requirements of this section. These inspections shall be performed by both the responsible contractor's quality control personnel and by quality assurance personnel. Where applicable, the inspection tasks listed in Table J10.1 shall be performed.

TABLE J9.1
Inspection of Composite Structures Prior to Concrete Placement

Inspection of Composite Structures Prior to Concrete Placement	QC		QA	
	Task	Doc.	Task	Doc.
Material identification of reinforcing steel (Type/Grade)	O	—	O	—
Determination of carbon equivalent for reinforcing steel other than ASTM A706/A706M	O	—	O	—
Proper reinforcing steel size, spacing and orientation	O	—	O	—
Reinforcing steel has not been rebent in the field	O	—	O	—
Reinforcing steel has been tied and supported as required	O	—	O	—
Required reinforcing steel clearances have been provided	O	—	O	—
Composite member has required size	O	—	O	—

TABLE J9.2
Inspection of Composite Structures during Concrete Placement

Inspection of Composite Structures during Concrete Placement	QC		QA	
	Task	Doc.	Task	Doc.
Concrete: Material identification (mix design, compressive strength, maximum large aggregate size, maximum slump)	O	D	O	D
Limits on water added at the truck or pump	O	D	O	D
Proper placement techniques to limit segregation	O	—	O	—

TABLE J9.3
Inspection of Composite Structures after Concrete Placement

Inspection of Composite Structures After Concrete Placement	QC		QA	
	Task	Doc	Task	Doc.
Achievement of minimum specified concrete compressive strength at specified age	—	D	—	D

TABLE J10.1
Inspection of H-Piles

Inspection of Piling	QC		QA	
	Task	Doc.	Task	Doc.
Protected zone—no holes and unapproved attachments made by the responsible contractor, as applicable	P	D	P	D

CHAPTER K

PREQUALIFICATION AND CYCLIC QUALIFICATION TESTING PROVISIONS

This chapter addresses requirements for qualification and prequalification testing.

This chapter is organized as follows:

K1. Prequalification of Beam-to-Column and Link-to-Column Connections
K2. Cyclic Tests for Qualification of Beam-to-Column and Link-to-Column Connections
K3. Cyclic Tests for Qualification of Buckling Restrained Braces

K1. PREQUALIFICATION OF BEAM-TO-COLUMN AND LINK-TO-COLUMN CONNECTIONS

1. Scope

This section contains minimum requirements for prequalification of beam-to-column moment connections in special moment frames (SMF), intermediate moment frames (IMF), composite special moment frames (C-SMF), and composite intermediate moment frames (C-IMF), and link-to-column connections in eccentrically braced frames (EBF). Prequalified connections are permitted to be used, within the applicable limits of prequalification, without the need for further qualifying cyclic tests. When the limits of prequalification or design requirements for prequalified connections conflict with the requirements of these Provisions, the limits of prequalification and design requirements for prequalified connections shall govern.

2. General Requirements

2a. Basis for Prequalification

Connections shall be prequalified based on test data satisfying Section K1.3, supported by analytical studies and design models. The combined body of evidence for prequalification must be sufficient to ensure that the connection is able to supply the required story drift angle for SMF, IMF, C-SMF, and C-IMF systems, or the required link rotation angle for EBF, on a consistent and reliable basis within the specified limits of prequalification. All applicable limit states for the connection that affect the stiffness, strength and deformation capacity of the connection and the seismic force-resisting system (SFRS) must be identified. The effect of design variables listed in Section K1.4 shall be addressed for connection prequalification.

2b. Authority for Prequalification

Prequalification of a connection and the associated limits of prequalification shall be

established by a connection prequalification review panel (CPRP) approved by the authority having jurisdiction.

3. Testing Requirements

Data used to support connection prequalification shall be based on tests conducted in accordance with Section K2. The CPRP shall determine the number of tests and the variables considered by the tests for connection prequalification. The CPRP shall also provide the same information when limits are to be changed for a previously prequalified connection. A sufficient number of tests shall be performed on a sufficient number of nonidentical specimens to demonstrate that the connection has the ability and reliability to undergo the required story drift angle for SMF, IMF, C-SMF, and C-IMF, and the required link rotation angle for EBF, where the link is adjacent to columns. The limits on member sizes for prequalification shall not exceed the limits specified in Section K2.3b.

4. Prequalification Variables

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

4a. Beam and Column Parameters for SMF and IMF, Link and Column Parameters for EBF

(a) Cross-section shape: wide flange, box or other

(b) Cross-section fabrication method: rolled shape, welded shape or other

(c) Depth

(d) Weight per foot

(e) Flange thickness

(f) Material specification

(g) Beam span-to-depth ratio (for SMF or IMF), or link length (for EBF)

(h) Width-to-thickness ratio of cross-section elements

(i) Lateral bracing

(j) Column orientation with respect to beam or link: beam or link is connected to column flange; beam or link is connected to column web; beams or links are connected to both the column flange and web; or other

(k) Other parameters pertinent to the specific connection under consideration

4b. Beam and Column Parameters for C-SMF and C-IMF

(a) For structural steel members that are part of a composite beam or column: specify parameters required in Section K1.4a

(b) Overall depth of composite beam and column

(c) Composite beam span-to-depth ratio

(d) Reinforcing bar diameter

(e) Reinforcement material specification

(f) Reinforcement development and splice requirements

(g) Transverse reinforcement requirements

(h) Concrete compressive strength and density

(i) Steel anchor dimensions and material specification

(j) Other parameters pertinent to the specific connection under consideration

4c. Beam-to-Column or Link-to-Column Relations

(a) Panel zone strength for SMF, IMF, and EBF

(b) Joint shear strength for C-SMF and C-IMF

(c) Doubler plate attachment details for SMF, IMF and EBF

(d) Joint reinforcement details for C-SMF and C-IMF

(e) Column-to-beam (or column-to-link) moment ratio

4d. Continuity and Diaphragm Plates

(a) Identification of conditions under which continuity plates or diaphragm plates are required

(b) Thickness, width and depth

(c) Attachment details

4e. Welds

(a) Location, extent (including returns), type (CJP, PJP, fillet, etc.) and any reinforcement or contouring required

(b) Filler metal classification strength and notch toughness

(c) Details and treatment of weld backing and weld tabs

(d) Weld access holes: size, geometry and finish

(e) Welding quality control and quality assurance beyond that described in Chapter J, including nondestructive testing (NDT) method, inspection frequency, acceptance criteria and documentation requirements

4f. Bolts

(a) Bolt diameter

(b) Bolt grade: ASTM F3125 Grades A325, A325M, A490, A490M, F1852, F2280 or other

(c) Installation requirements: pretensioned, snug-tight or other

(d) Hole type: standard, oversize, short-slot, long-slot or other

(e) Hole fabrication method: drilling, punching, sub-punching and reaming, or other

(f) Other parameters pertinent to the specific connection under consideration

4g. Reinforcement in C-SMF and C-IMF

(a) Location of longitudinal and transverse reinforcement

(b) Cover requirements

(c) Hook configurations and other pertinent reinforcement details

4h. Quality Control and Quality Assurance

Requirements that exceed or supplement requirements specified in Chapter J, if any.

4i. Additional Connection Details

All variables and workmanship parameters that exceed AISC, RCSC and AWS requirements pertinent to the specific connection under consideration, as established by the CPRP.

5. Design Procedure

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

6. Prequalification Record

A prequalified connection shall be provided with a written prequalification record with the following information:

(a) General description of the prequalified connection and drawings that clearly identify key features and components of the connection

(b) Description of the expected behavior of the connection in the elastic and inelastic ranges of behavior, intended location(s) of inelastic action, and a description of limit states controlling the strength and deformation capacity of the connection

(c) Listing of systems for which connection is prequalified: SMF, IMF, EBF, C-SMF, or C-IMF.

(d) Listing of limits for all applicable prequalification variables listed in Section K1.4

(e) Listing of demand critical welds

(f) Definition of the region of the connection that comprises the protected zone

(g) Detailed description of the design procedure for the connection, as required in Section K1.5

(h) List of references of test reports, research reports and other publications that provided the basis for prequalification

(i) Summary of quality control and quality assurance procedures

K2. CYCLIC TESTS FOR QUALIFICATION OF BEAM-TO-COLUMN AND LINK-TO-COLUMN CONNECTIONS

1. Scope

This section provides requirements for qualifying cyclic tests of beam-to-column moment connections in SMF, IMF, C-SMF, and C-IMF; and link-to-column connections in EBF, when required in these Provisions. The purpose of the testing described in this section is to provide evidence that a beam-to-column connection or a link-to-column connection satisfies the requirements for strength and story drift angle or link rotation angle in these Provisions. Alternative testing requirements are permitted when approved by the engineer of record and the authority having jurisdiction.

2. Test Subassemblage Requirements

The test subassemblage shall replicate, as closely as is practical, the conditions that will occur in the prototype during earthquake loading. The test subassemblage shall include the following features:

(a) The test specimen shall consist of at least a single column with beams or links attached to one or both sides of the column.

(b) Points of inflection in the test assemblage shall coincide with the anticipated points of inflection in the prototype under earthquake loading.

(c) Lateral bracing of the test subassemblage is permitted near load application or reaction points as needed to provide lateral stability of the test subassemblage. Additional lateral bracing of the test subassemblage is not permitted, unless it replicates lateral bracing to be used in the prototype.

3. Essential Test Variables

The test specimen shall replicate as closely as is practical the pertinent design, detailing, construction features and material properties of the prototype. The following variables shall be replicated in the test specimen.

3a. Sources of Inelastic Rotation

The inelastic rotation shall be computed based on an analysis of test specimen deformations. Sources of inelastic rotation include, but are not limited to, yielding of members, yielding of connection elements and connectors, yielding of reinforcing steel, inelastic deformation of concrete, and slip between members and connection elements. For beam-to-column moment connections in SMF, IMF, C-SMF and C-IMF, inelastic rotation is computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the beam with the centerline of the column. For link-to-column connections in EBF, inelastic rotation shall be computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the link with the face of the column.

Inelastic rotation shall be developed in the test specimen by inelastic action in the same members and connection elements as anticipated in the prototype (in other words, in the beam or link, in the column panel zone, in the column outside of the panel zone, or in connection elements) within the limits described below. The percentage of the total inelastic rotation in the test specimen that is developed in each member or connection element shall be within 25% of the anticipated percentage of the total inelastic rotation in the prototype that is developed in the corresponding member or connection element.

3b. Members

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% of the depth of the prototype beam or link.

(b) For SMF, IMF and EBF, the weight per foot of the test beam or link shall be no less than 75% of the weight per foot of the prototype beam or link.

(c) For C-SMF and C-IMF, the weight per foot of the structural steel member that forms part of the test beam shall be no less than 75% of the weight per foot of the structural steel member that forms part of the prototype beam.

The size of the column used in the test specimen shall correctly represent the inelastic action in the column, as per the requirements in Section K2.3a. In addition, in SMF, IMF and EBF, the depth of the test column shall be no less than 90% of the depth of the prototype column. In C-SMF and C-IMF, the depth of the structural steel member that forms part of the test column shall be no less than 90% of the depth of the structural steel member that forms part of the prototype column.

The width-to-thickness ratios of compression elements of steel members of the test specimen shall meet the width-to-thickness limitations as specified in these Provisions for members in SMF, IMF, C-SMF, C-IMF or EBF, as applicable.

Exception: The width-to-thickness ratios of compression elements of members in the test specimen are permitted to exceed the width-to-thickness limitations specified in these Provisions if both of the following conditions are met:

(a) The width-to-thickness ratios of compression elements of the members of the test specimen are no less than the width-to-thickness ratios of compression elements in the corresponding prototype members.

(b) Design features that are intended to restrain local buckling in the test specimen, such as concrete encasement of steel members, concrete filling of steel members, and other similar features are representative of the corresponding design features in the prototype.

Extrapolation beyond the limitations stated in this section is permitted subject to qualified peer review and approval by the authority having jurisdiction.

3c. Reinforcing Steel Amount, Size and Detailing

The total area of the longitudinal reinforcing bars shall not be less than 75% of the area in the prototype, and individual bars shall not have an area less than 70% of the maximum bar size in the prototype.

Design approaches and methods used for anchorage and development of reinforcement, and for splicing reinforcement in the test specimen shall be representative of the prototype.

The amount, arrangement and hook configurations for transverse reinforcement shall be representative of the bond, confinement and anchorage conditions of the prototype.

3d. 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.

3e. 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.

3f. Steel Strength for Steel Members and Connection Elements

The following additional requirements shall be satisfied for each steel member or connection element of the test specimen that supplies inelastic rotation by yielding:

(a) The yield strength shall be determined as specified in Section K2.6a. The use of yield stress values that are reported on certified material test reports in lieu of physical testing is prohibited for the purposes of this section.

(b) The yield strength of the beam flange as tested in accordance with Section K2.6a shall not be more than 15% below R_yF_y for the grade of steel to be used for the corresponding elements of the prototype.

(c) The yield strength of the columns and connection elements shall not be more than 15% above or below R_yF_y for the grade of steel to be used for the corresponding elements of the prototype. R_yF_y shall be determined in accordance with Section A3.2.

User Note: Based upon the preceding criteria, steel of the specified grade with a specified minimum yield stress, F_y, of up to and including 1.15 times the R_yF_y for the steel tested should be permitted in the prototype. In production, this limit should be checked using the values stated on the steel manufacturer's material test reports.

3g. Steel Strength and Grade for Reinforcing Steel

Reinforcing steel in the test specimen shall have the same ASTM designation as the corresponding reinforcing steel in the prototype. The specified minimum yield stress of reinforcing steel in the test specimen shall not be less than the specified minimum yield stress of the corresponding reinforcing steel in the prototype.

3h. Concrete Strength and Density

The specified compressive strength of concrete in members and connection elements of the test specimen shall be at least 75% and no more than 125% of the specified compressive strength of concrete in the corresponding members and connection elements of the prototype.

The compressive strength of concrete in the test specimen shall be determined in accordance with Section K2.6d.

The density classification of the concrete in the members and connection elements of the test specimen shall be the same as the density classification of concrete in the corresponding members and connection elements of the prototype. The density classification of concrete shall correspond to either normal weight, lightweight, all-lightweight, or sand-lightweight as defined in ACI 318.

3i. Welded Joints

Welds on the test specimen shall satisfy the following requirements:

(a) Welding shall be performed in conformance with Welding Procedure Specifications (WPS) as required in AWS D1.1/D1.1M. The WPS essential variables shall satisfy the requirements in AWS D1.1/D1.1M and shall be within the parameters established by the filler-metal manufacturer. The tensile strength and Charpy V-notch (CVN) toughness of the welds used in the test specimen shall be determined by tests as specified in Section K2.6e, made using the same filler metal classification, manufacturer, brand or trade name, diameter, and average heat input for the WPS used on the test specimen. The use of tensile strength and CVN toughness values that are reported on the manufacturer's typical certificate of conformance, in lieu of physical testing, is not permitted for purposes of this section.

(b) The specified minimum tensile strength of the filler metal used for the test specimen shall be the same as that to be used for the welds on the corresponding prototype. The tensile strength of the deposited weld as tested in accordance with Section K2.6c shall not exceed the tensile strength classification of the filler metal specified for the prototype by more than 25 ksi (170 MPa).

User Note: Based upon the criteria in (b), should the tested tensile strength of the weld metal exceed 25 ksi (170 MPa) above the specified minimum tensile strength, the prototype weld should be made with a filler metal and WPS that will provide a tensile strength no less than 25 ksi (170 MPa) below the tensile strength measured in the material test plate. When this is the case, the tensile strength of welds resulting from use of the filler metal and the WPS to be used in the prototype should be determined by using an all-weld-metal tension specimen. The test plate is described in AWS D1.8/D1.8M clause A6 and shown in AWS D1.8/D1.8M Figure A.1.

(c) The specified minimum CVN toughness of the filler metal used for the test specimen shall not exceed that to be used for the welds on the corresponding prototype. The tested CVN toughness of the weld as tested in accordance with Section K2.6c shall not exceed the minimum CVN toughness specified for the prototype by more than 50%, nor 25 ft-lb (34 J), whichever is greater.

User Note: Based upon the criteria in (c), should the tested CVN toughness of the weld metal in the material test specimen exceed the specified CVN toughness for the test specimen by 25 ft-lb (34 J) or 50%, whichever is greater, the prototype weld can be made with a filler metal and WPS that will provide a CVN toughness that is no less than 25 ft-lb (34 J) or 33% lower, whichever is lower, below the CVN toughness measured in the weld metal material test plate. When this is the case, the weld properties resulting from the filler metal and WPS to be used in the prototype can be determined using five CVN test specimens. The test plate is described in AWS D1.8/D1.8M clause A6 and shown in AWS D1.8/D1.8M Figure A.1.

(d) 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.

(e) Weld details such as backing, tabs and access holes 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.

(f) Methods of inspection and nondestructive testing and standards of acceptance used for test specimen welds shall be the same as those to be used for the prototype welds.

User Note: The filler metal used for production of the prototype may be of a different classification, manufacturer, brand or trade name, and diameter, if Sections K2.3i(b) and K2.3i(c) are satisfied. To qualify alternate filler metals, the tests as prescribed in Section K2.6e should be conducted.

3j. Bolted Joints

The bolted portions of the test specimen shall replicate the bolted portions of the prototype connection as closely as possible. Additionally, bolted portions of the test specimen shall satisfy the following requirements:

(a) The bolt grade (for example, ASTM F3125 Grades A325, A325M, A490, A490M, F1852, F2280) used in the test specimen shall be the same as that to be used for the prototype, except that heavy hex bolts are permitted to be substituted for twist-off-type tension control bolts of equal specified minimum tensile strength, and vice versa.

(b) 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.

(c) 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.

(d) Bolts in the test specimen shall have the same installation (pretensioned or other) and faying surface preparation (no specified slip resistance, Class A or B slip resistance, or other) as that to be used for the corresponding bolts in the prototype.

3k. Load Transfer Between Steel and Concrete

Methods used to provide load transfer between steel and concrete in the members and connection elements of the test specimen, including direct bearing, shear connection, friction and others, shall be representative of the prototype.

4. Loading History

4a. General Requirements

The test specimen shall be subjected to cyclic loads in accordance with the requirements prescribed in Section K2.4b for beam-to-column moment connections in SMF, IMF, C-SMF, and C-IMF, and in accordance with the requirements prescribed in Section K2.4c for link-to-column connections in EBF.

Loading sequences to qualify connections for use in SMF, IMF, C-SMF or C-IMF with columns loaded orthogonally shall be applied about both axes using the loading sequence specified in Section K2.4b. Beams used about each axis shall represent the most demanding combination for which qualification or prequalification is sought. In lieu of concurrent application about each axis of the loading sequence specified in Section K2.4b, the loading sequence about one axis shall satisfy requirements of Section K2.4b, while a concurrent load of constant magnitude, equal to the expected strength of the beam connected to the column about its orthogonal axis, shall be applied about the orthogonal axis.

Loading sequences other than those specified in Sections K2.4b and K2.4c are permitted to be used when they are demonstrated to be of equivalent or greater severity.

4b. Loading Sequence for Beam-to-Column Moment Connections

Qualifying cyclic tests of beam-to-column moment connections in SMF, IMF, C-SMF and C-IMF shall be conducted by controlling the story drift angle, θ, imposed on the test specimen, as specified below:

(a) 6 cycles at $\theta = 0.00375$ rad

(b) 6 cycles at $\theta = 0.005$ rad

(c) 6 cycles at $\theta = 0.0075$ rad

(d) 4 cycles at $\theta = 0.01$ rad

(e) 2 cycles at $\theta = 0.015$ rad

(f) 2 cycles at $\theta = 0.02$ rad

(g) 2 cycles at $\theta = 0.03$ rad

(h) 2 cycles at $\theta = 0.04$ rad

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

4c. Loading Sequence for Link-to-Column Connections

Qualifying cyclic tests of link-to-column moment connections in EBF shall be conducted by controlling the total link rotation angle, γ_{total}, imposed on the test specimen, as follows:

(a) 6 cycles at $\gamma_{total} = 0.00375$ rad

(b) 6 cycles at $\gamma_{total} = 0.005$ rad

(c) 6 cycles at $\gamma_{total} = 0.0075$ rad

(d) 6 cycles at $\gamma_{total} = 0.01$ rad

(e) 4 cycles at $\gamma_{total} = 0.015$ rad

(f) 4 cycles at $\gamma_{total} = 0.02$ rad

(g) 2 cycles at $\gamma_{total} = 0.03$ rad

(h) 1 cycle at $\gamma_{total} = 0.04$ rad

(i) 1 cycle at $\gamma_{total} = 0.05$ rad

(j) 1 cycle at $\gamma_{total} = 0.07$ rad

(k) 1 cycle at $\gamma_{total} = 0.09$ rad

Continue loading at increments of $\gamma_{total} = 0.02$ rad, with one cycle of loading at each step.

5. Instrumentation

Sufficient instrumentation shall be provided on the test specimen to permit measurement or calculation of the quantities listed in Section K2.7.

6. Testing Requirements for Material Specimens

6a. Tension Testing Requirements for Structural Steel Material Specimens

Tension testing shall be conducted on samples taken from material test plates in accordance with Section K2.6c. The material test plates shall be taken from the steel of the same heat as used in the test specimen. Tension-test results from certified material test reports shall be reported, but shall not be used in lieu of physical testing for the purposes of this section. Tension testing shall be conducted and reported for the following portions of the test specimen:

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

(b) Any element of the connection that supplies inelastic rotation by yielding

6b. Tension Testing Requirements for Reinforcing Steel Material Specimens

Tension testing shall be conducted on samples of reinforcing steel in accordance with Section K2.6c. Samples of reinforcing steel used for material tests shall be taken from the same heat as used in the test specimen. Tension-test results from certified material test reports shall be reported, but shall not be used in lieu of physical testing for the purposes of this section.

6c. Methods of Tension Testing for Structural and Reinforcing Steel Material Specimens

Tension testing shall be conducted in accordance with ASTM A6/A6M, ASTM A370, and ASTM E8, as applicable, with the following exceptions:

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

(b) The loading rate for the tension test shall replicate, as closely as practical, the loading rate to be used for the test specimen.

6d. Testing Requirements for Concrete

Test cylinders of concrete used for the test specimen shall be made and cured in accordance with ASTM C31. At least three cylinders of each batch of concrete used in a component of the test specimen shall be tested within five days before or after of the end of the cyclic qualifying test of the test specimen. Tests of concrete cylinders shall be in accordance with ASTM C39. The average compressive strength of the three cylinders shall be no less than 90% and no greater than 150% of the specified compressive strength of the concrete in the corresponding member or connection element of the test specimen. In addition, the average compressive strength of the three cylinders shall be no more than 3000 psi (20.7 MPa) greater than the specified

compressive strength of the concrete in the corresponding member or connection element of the test specimen.

Exception: If the average compressive strength of three cylinders is outside of these limits, the specimen is still acceptable if supporting calculations or other evidence is provided to demonstrate how the difference in concrete strength will affect the connection performance.

6e. Testing Requirements for Weld Metal Material Specimens

Weld metal testing shall be conducted on samples extracted from the material test plate, made using the same filler metal classification, manufacturer, brand or trade name and diameter, and using the same average heat input as used in the welding of the test specimen. The tensile strength and CVN toughness of weld material specimens shall be determined in accordance with *Standard Methods for Mechanical Testing of Welds* (AWS B4.0/B4.0M). The use of tensile strength and CVN toughness values that are reported on the manufacturer's typical certificate of conformance in lieu of physical testing is not permitted for use for purposes of this section.

The same WPS shall be used to make the test specimen and the material test plate. The material test plate shall use base metal of the same grade and type as was used for the test specimen, although the same heat need not be used. If the average heat input used for making the material test plate is not within ±20% of that used for the test specimen, a new material test plate shall be made and tested.

7. Test Reporting Requirements

For each test specimen, a written test report meeting the requirements of the authority having jurisdiction and the requirements of this section shall be prepared. The report shall thoroughly document all key features and results of the test. The report shall include the following information:

(a) A drawing or clear description of the test subassemblage, including key dimensions, boundary conditions at loading and reaction points, and location of lateral braces.

(b) 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, specified compressive strength and density of concrete, reinforcing bar sizes and grades, reinforcing bar locations, reinforcing bar splice and anchorage details, and all other pertinent details of the connection.

(c) A listing of all other essential variables for the test specimen, as listed in Section K2.3.

(d) A listing or plot showing the applied load or displacement history of the test specimen.

(e) A listing of all welds to be designated demand critical.

(f) Definition of the region of the member and connection to be designated a protected zone.

(g) 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.

(h) A plot of beam moment versus story drift angle for beam-to-column moment connections; or a plot of link shear force versus link rotation angle for link-to-column connections. For beam-to-column connections, the beam moment and the story drift angle shall be computed with respect to the centerline of the column.

(i) The story drift angle and the total inelastic rotation developed by the test specimen. The components of the test specimen contributing to the total inelastic rotation 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.

(j) A chronological listing of test observations, including observations of yielding, slip, instability, cracking and rupture of steel elements, cracking of concrete, and other damage of any portion of the test specimen as applicable.

(k) 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.

(l) The results of the material specimen tests specified in Section K2.6.

(m) The welding procedure specifications (WPS) and welding inspection reports.

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

8. Acceptance Criteria

The test specimen must satisfy the strength and story drift angle or link rotation angle requirements of these Provisions for the SMF, IMF, C-SMF, C-IMF or EBF connection, as applicable. The test specimen must sustain the required story drift angle or link rotation angle for at least one complete loading cycle.

K3. CYCLIC TESTS FOR QUALIFICATION OF BUCKLING-RESTRAINED BRACES

1. Scope

This section includes requirements for qualifying cyclic tests of individual buckling-restrained braces and buckling-restrained brace subassemblages, when required in these Provisions. The purpose of the testing of individual braces is to provide evidence that a buckling-restrained brace satisfies the requirements for strength and inelastic deformation by these provisions; it also permits the determination of maximum brace forces for design of adjoining elements. The purpose of testing of the

brace subassemblage is to provide evidence that the brace-design is able to satisfactorily accommodate the deformation and rotational demands associated with the design. Further, the subassemblage test is intended to demonstrate that the hysteretic behavior of the brace in the subassemblage is consistent with that of the individual brace elements tested uniaxially.

Alternative testing requirements are permitted when approved by the engineer of record and the authority having jurisdiction. This section provides only minimum recommendations for simplified test conditions.

2. Subassemblage Test Specimen

The subassemblage test specimen shall satisfy the following requirements:

(a) The mechanism for accommodating inelastic rotation in the subassemblage test specimen brace shall be the same as that of the prototype. The rotational deformation demands on the subassemblage test specimen brace shall be equal to or greater than those of the prototype.

(b) The axial yield strength of the steel core, P_{ysc}, of the brace in the subassemblage test specimen shall not be less than 90% of that of the prototype where both strengths are based on the core area, A_{sc}, multiplied by the yield strength as determined from a coupon test.

(c) The cross-sectional shape and orientation of the steel core projection of the subassemblage test specimen brace shall be the same as that of the brace in the prototype.

(d) The same documented design methodology shall be used for design of the subassemblage as used for the prototype, to allow comparison of the rotational deformation demands on the subassemblage brace to the prototype. In stability calculations, beams, columns and gussets connecting the core shall be considered parts of this system.

(e) The calculated margins of safety for the prototype connection design, steel core projection stability, overall buckling and other relevant subassemblage test specimen brace construction details, excluding the gusset plate, for the prototype, shall equal or exceed those of the subassemblage test specimen construction. If the qualification brace test specimen required in Section K3.3 was also tested including the subassemblage requirements of this section, the lesser safety factor for overall buckling between that required in Section K3.3a(a) and that required in this section may be used.

(f) Lateral bracing of the subassemblage test specimen shall replicate the lateral bracing in the prototype.

(g) The brace test specimen and the prototype shall be manufactured in accordance with the same quality control and assurance processes and procedures.

Extrapolation beyond the limitations stated in this section is permitted subject to qualified peer review and approval by the authority having jurisdiction.

3. Brace Test Specimen

The brace test specimen shall replicate as closely as is practical the pertinent design, detailing, construction features and material properties of the prototype.

3a. Design of Brace Test Specimen

The same documented design methodology shall be used for the brace test specimen and the prototype. The design calculations shall demonstrate, at a minimum, the following requirements:

(a) The calculated margin of safety for stability against overall buckling for the prototype shall equal or exceed that of the brace test specimen.

(b) The calculated margins of safety for the brace test specimen and the prototype shall account for differences in material properties, including yield and ultimate stress, ultimate elongation, and toughness.

3b. Manufacture of Brace Test Specimen

The brace test specimen and the prototype shall be manufactured in accordance with the same quality control and assurance processes and procedures.

3c. Similarity of Brace Test Specimen and Prototype

The brace test specimen shall meet the following requirements:

(a) The cross-sectional shape and orientation of the steel core shall be the same as that of the prototype.

(b) The axial yield strength of the steel core, P_{ysc}, of the brace test specimen shall not be less than 30% nor more than 120% of the prototype where both strengths are based on the core area, A_{sc}, multiplied by the yield strength as determined from a coupon test.

(c) The material for, and method of, separation between the steel core and the buckling restraining mechanism in the brace test specimen shall be the same as that in the prototype.

Extrapolation beyond the limitations stated in this section is permitted subject to qualified peer review and approval by the authority having jurisdiction.

3d. Connection Details

The connection details used in the brace test specimen shall represent the prototype connection details as closely as practical.

3e. Materials

1. Steel Core

The following requirements shall be satisfied for the steel core of the brace test specimen:

(a) The specified minimum yield stress of the brace test specimen steel core shall be the same as that of the prototype.

(b) The measured yield stress of the material of the steel core in the brace test specimen shall be at least 90% of that of the prototype as determined from coupon tests.

(c) The specified minimum ultimate stress and strain of the brace test specimen steel core shall not exceed those of the prototype.

2. Buckling-Restraining Mechanism

Materials used in the buckling-restraining mechanism of the brace test specimen shall be the same as those used in the prototype.

3f. Connections

The welded, bolted and pinned joints on the test specimen shall replicate those on the prototype as close as practical.

4. Loading History

4a. General Requirements

The test specimen shall be subjected to cyclic loads in accordance with the requirements prescribed in Sections K3.4b and K3.4c. Additional increments of loading beyond those described in Section K3.4c are permitted. Each cycle shall include a full tension and full compression excursion to the prescribed deformation.

4b. Test Control

The test shall be conducted by controlling the level of axial or rotational deformation, Δ_b, imposed on the test specimen. As an alternate, the maximum rotational deformation is permitted to be applied and maintained as the protocol is followed for axial deformation.

4c. Loading Sequence

Loads shall be applied to the test specimen to produce the following deformations, where the deformation is the steel core axial deformation for the test specimen and the rotational deformation demand for the subassemblage test specimen brace:

(a) 2 cycles of loading at the deformation corresponding to $\Delta_b = \Delta_{by}$

(b) 2 cycles of loading at the deformation corresponding to $\Delta_b = 0.50\ \Delta_{bm}$

(c) 2 cycles of loading at the deformation corresponding to $\Delta_b = 1.0\ \Delta_{bm}$

(d) 2 cycles of loading at the deformation corresponding to $\Delta_b = 1.5\ \Delta_{bm}$

(e) 2 cycles of loading at the deformation corresponding to $\Delta_b = 2.0\ \Delta_{bm}$

(f) Additional complete cycles of loading at the deformation corresponding to $\Delta_b = 1.5\ \Delta_{bm}$, as required for the brace test specimen to achieve a cumulative inelastic

axial deformation of at least 200 times the yield deformation (not required for the subassemblage test specimen)

where

Δ_{bm} = value of deformation quantity, Δ_b, at least equal to that corresponding to the design story drift, in. (mm)

Δ_{by} = value of deformation quantity, Δ_b, at first yield of test specimen, in. (mm)

The design story drift shall not be taken as less than 0.01 times the story height for the purposes of calculating Δ_{bm}. Other loading sequences are permitted to be used to qualify the test specimen when they are demonstrated to be of equal or greater severity in terms of maximum and cumulative inelastic deformation.

5. Instrumentation

Sufficient instrumentation shall be provided on the test specimen to permit measurement or calculation of the quantities listed in Section K3.7.

6. Materials Testing Requirements

6a. Tension Testing Requirements

Tension testing shall be conducted on samples of steel taken from the same heat of steel as that used to manufacture the steel core. Tension test results from certified material test reports shall be reported but are prohibited in place of material specimen testing for the purposes of this Section. Tension test results shall be based upon testing that is conducted in accordance with Section K3.6b.

6b. Methods of Tension Testing

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

(a) The yield stress that is reported from the test shall be based upon the yield strength definition in ASTM A370, using the offset method of 0.002 strain.

(b) The loading rate for the tension test shall replicate, as closely as is practical, the loading rate used for the test specimen.

(c) The coupon shall be machined so that its longitudinal axis is parallel to the longitudinal axis of the steel core.

7. Test Reporting Requirements

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

(a) A drawing or clear description of the test specimen, including key dimensions, boundary conditions at loading and reaction points, and location of lateral bracing, if any.

(b) A drawing of the connection details showing member sizes, grades of steel, the sizes of all connection elements, welding details including filler metal, the size and location of bolt or pin holes, the size and grade of connectors, and all other pertinent details of the connections.

(c) A listing of all other essential variables as listed in Sections K3.2 or K3.3.

(d) A listing or plot showing the applied load or displacement history.

(e) A plot of the applied load versus the deformation, Δ_b. The method used to determine the deformations shall be clearly shown. The locations on the test specimen where the loads and deformations were measured shall be clearly identified.

(f) A chronological listing of test observations, including observations of yielding, slip, instability, transverse displacement along the test specimen and rupture of any portion of the test specimen and connections, as applicable.

(g) The results of the material specimen tests specified in Section K3.6.

(h) The manufacturing quality control and quality assurance plans used for the fabrication of the test specimen. These shall be included with the welding procedure specifications and welding inspection reports.

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

8. Acceptance Criteria

At least one subassemblage test that satisfies the requirements of Section K3.2 shall be performed. At least one brace test that satisfies the requirements of Section K3.3 shall be performed. Within the required protocol range, all tests shall satisfy the following requirements:

(a) The plot showing the applied load versus displacement history shall exhibit stable, repeatable behavior with positive incremental stiffness.

(b) There shall be no rupture, brace instability, or brace end connection failure.

(c) For brace tests, each cycle to a deformation greater than Δ_{by}, the maximum tension and compression forces shall not be less than the nominal strength of the core.

(d) For brace tests, each cycle to a deformation greater than Δ_{by}, the ratio of the maximum compression force to the maximum tension force shall not exceed 1.5.

Other acceptance criteria are permitted to be adopted for the brace test specimen or subassemblage test specimen subject to qualified peer review and approval by the authority having jurisdiction.

COMMENTARY on the Seismic Provisions for Structural Steel Buildings

July 12, 2016

(The Commentary is not a part of ANSI/AISC 341-16, *Seismic Provisions for Structural Steel Buildings*, and is included for informational purposes only.)

INTRODUCTION

The Provisions are intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations and limits of the Provisions.

The Provisions and Commentary are intended for use by design professionals with demonstrated engineering competence.

COMMENTARY PREFACE

Experience from the 1994 Northridge and 1995 Kobe earthquakes significantly expanded knowledge regarding the seismic response of structural steel building systems, particularly welded steel moment frames. Shortly after the Northridge earthquake, the SAC Joint Venture* initiated a comprehensive study of the seismic performance of steel moment frames. Funded by the Federal Emergency Management Agency (FEMA), SAC developed 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 actively participated in the SAC activities.

These 2016 AISC *Seismic Provisions for Structural Steel Buildings*, hereinafter referred to as the Provisions, continues the practice of incorporating recommendations from the NEHRP Provisions, most recently FEMA P-750 (FEMA, 2009a), and other research. While research is ongoing, the Committee has prepared this revision of the Provisions using the best available knowledge to date. These Provisions were being developed in the same time frame as a revision of *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7 (ASCE, 2016) was being accomplished.

It is also anticipated that these Provisions will be adopted by the 2018 *International Building Code* (IBC), and the National Fire Protection Association (NFPA) Building Construction and Safety Code, NFPA 5000 (NFPA, 2018). It is expected that both of these model building codes will reference ASCE/SEI 7 for seismic loading and neither code will contain seismic requirements.

Where there is a desire to use these Provisions with a model code that has not yet adopted these Provisions, it is essential that the AISC *Specification for Structural Steel Buildings* (AISC, 2016a), hereafter referred to as the *Specification*, be used in conjunction with these Provisions, as they are companion documents. Where the provisions for intermediate or special moment frame systems are used, the use of AISC *Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications,* ANSI/AISC 358 (AISC, 2016b) may be warranted. In addition, users should also concurrently use ASCE/SEI 7 for a fully coordinated package.

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

CHAPTER A
GENERAL REQUIREMENTS

A1. SCOPE

The scope of the *Specification* and the Provisions includes buildings and other structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral force-resisting elements. For simplicity, the commentary refers to steel buildings and structures interchangeably.

However, it should be noted that these provisions were developed specifically for buildings. The Provisions, therefore, may not be applicable, in whole or in part, to some nonbuilding structures that do not have the building-like characteristics described in the preceding paragraph. Extrapolation of their use to such nonbuilding structures should be done with due consideration of the inherent differences between the response characteristics of buildings and these nonbuilding structures.

Structural steel systems in seismic regions are generally expected to dissipate seismic input energy through controlled inelastic deformations of the structure. The Provisions supplement the *Specification* for such applications. The seismic design loads specified in the building codes have been developed considering the energy dissipation generated during inelastic response.

The Provisions are intended to be mandatory for structures where they have been specifically referenced when defining a seismic response modification coefficient, R, in *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7 (ASCE, 2016). For steel structures, typically this occurs in seismic design category D, E and F, where R is greater than 3. However, there are instances where R of less than 3 is assigned to a system and the Provisions are still required. These limited cases occur in ASCE/SEI 7 Table 12.2-1 for cantilevered column systems and Table 15.4-1 for nonbuilding structures similar to buildings. For these systems with R less than 3, the use of the Provisions is required. In general, for structures in seismic design categories B and C, the designer is given a choice to either solely use the *Specification* and the R given for structural steel buildings not specifically detailed for seismic resistance (typically, a value of 3) or the designer may choose to assign a higher R to a system detailed for seismic resistance and follow the requirements of the Provisions. Additionally, for composite steel-concrete structures, there are cases where the Provisions are required in seismic design categories B and C, as specified in Table 12.2-1 of ASCE/SEI 7. This typically occurs for composite systems designated as "ordinary" where the counterpart reinforced concrete systems have designated R and design requirements for seismic design categories B and C.

The Provisions include requirements for columns not part of the seismic force-resisting system (SFRS) in Sections D2.5 and D2.6.

The 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 later editions of those provisions and in ASCE/SEI 7. Because composite systems are assemblies of steel and concrete components, the portions of these Provisions pertaining to steel, the *Specification* and *Building Code Requirements for Structural Concrete*, ACI 318-14 (ACI, 2014), form an important basis for provisions related to composite construction.

There is at present limited experience in the U.S. with composite building systems subjected to extreme seismic loads and many of the recommendations herein are necessarily of a conservative and/or qualitative nature. Extensive design and performance experience with this type of building in Japan clearly indicates that composite systems, due to their inherent rigidity and toughness, can equal or exceed the performance of reinforced concrete only or structural steel only buildings (Deierlein and Noguchi, 2004; Yamanouchi et al., 1998). Composite systems have been extensively used in tall buildings throughout the world.

Careful attention to all aspects of the design is necessary in the design of composite systems, particularly with respect to 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 force transfer in composite systems. However, these details should not necessarily be treated as design standards. The cited references provide more specific information on the design of composite connections. For a general discussion of these issues and some specific design examples, refer to Viest et al. (1997).

The design and construction of composite elements and systems continues to evolve in practice. Except where explicitly stated, these Provisions are not intended to limit the application of new systems 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 fully restrained (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 composite elements can vary considerably due to the effects of cracking.

When systems have both ductile and nonductile elements, the relative stiffness of each should be properly modeled; the ductile elements can deform inelastically while the nonductile 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.

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

The specifications, codes and standards referenced herein are listed with the appropriate revision date in this section or in *Specification* Section A2. Since the Provisions act as a supplement to the *Specification*, the references listed in *Specification* Section A2 are not repeated again in the Provisions.

A3. MATERIALS

1. Material Specifications

The structural steels that are explicitly permitted for use in seismic applications have been selected based upon their inelastic properties and weldability. In general, they meet the following characteristics: (1) a pronounced stress-strain plateau at the yield stress; (2) a large inelastic strain capability [e.g., tensile elongation of 20% or greater in a 2 in. (50 mm) gage length]; and (3) good weldability. Other steels should not be used without evidence that the above criteria are met. For structural wide-flange shapes, ASTM A992/A992M and ASTM A913/A913M contain additional supplementary requirements that provide a limitation on the ratio of yield stress to tensile stress to be not greater than 0.85.

The limitation on the specified minimum yield stress for members expecting inelastic action refers to inelastic action under the effects of the design earthquake. The 50 ksi (345 MPa) limitation on the specified minimum yield stress for members was restricted to those systems in Chapters E, F, G and H expected to undergo moderate to significant inelastic action, while a 55 ksi (380 MPa) limitation was assigned to the systems in Sections E1, F1, G1, H1 and H4, since those systems are expected to undergo limited inelastic action. The listed steels conforming to ASTM A1011/A1011M with a specified minimum yield stress of 55 ksi (380 MPa) are included as they have adequate ductility considering their limited thickness range. This steel is commonly used by the metal building industry in built-up sections.

An exception allows the yield stress limits to be exceeded where testing or rational criteria permit. An example of testing that would permit higher strength steels for elements would be cyclic tests per Sections K2 and K3 where the element is subject to the anticipated level of inelastic strain for the intended use.

Modern steels of higher strength, such as ASTM A913/A913M Grades 65 (450) and 70 (485), are generally considered to have properties acceptable for seismic column applications where limited inelastic action may occur. An exception permits structural steel with a specified minimum yield stress up to 70 ksi (485 MPa) for columns in those designated systems where the anticipated level of inelastic yielding will be minor.

Conformance with the material requirements of the *Specification* is satisfied by the testing performed in accordance with ASTM provisions by the manufacturer. Supplemental or independent material testing is only required for material that cannot be identified or traced to a material test report and materials used in qualification testing, according to Section K2.

While ASTM A709/A709M steel is primarily used in the design and construction of bridges, it could also be used in building construction. Written as an umbrella specification, its grades are essentially the equivalent of other approved ASTM specifications. For example, ASTM A709/A709M Grade 50 (345) is essentially ASTM A572/A572M Grade 50 (345) and ASTM A709/A709M Grade 50W (345W) is essentially ASTM A588/A588M Grade 50 (345). Thus, if used, ASTM A709/A709M material should be treated as would the corresponding approved ASTM material grade.

ASTM A1085/A1085M, a new specification for the production of hollow structural sections (HSS) has been added as an approved steel for the SFRS. Benefits of this new material specification include tighter mass tolerances, a maximum specified yield stress, minimum specified CVN requirements, and a reduced variability of material yield strength and tensile stress versus the ASTM A500/A500M Grades B and C HSS and ASTM A53/A53M Grade B pipe materials.

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/web fillet as illustrated in Figure C-A3.1. Recommendations issued by AISC (AISC, 1997a) were followed up by a series of industry sponsored research projects (Kaufmann et al., 2001; Uang and Chi, 2001; Kaufmann and Fisher, 2001; Lee et al., 2002; Bartlett et al., 2001). This research generally corroborates AISC's initial findings and recommendations.

2. Expected Material Strength

The Provisions employ a methodology for many seismic systems (e.g., special moment frames, special concentrically braced frames, and eccentrically braced frames) that can be characterized as "capacity design." That is, the required strength of elements which are intended to behave essentially elastically is defined by forces corresponding to the capacity (expected strength) of certain members or components intended to undergo inelastic deformations (e.g., the link in eccentrically braced

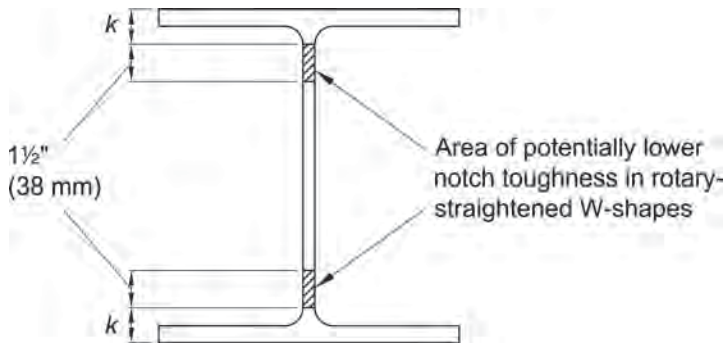


Fig. C-A3.1. "k-area."

frames). This methodology serves to confine ductility demands to members or components that have specific requirements to ensure their ductile behavior. Furthermore, the methodology serves to ensure that within that member or component the desired ductile mode of yielding governs and other nonductile modes are precluded.

Such a capacity-design methodology requires a realistic estimate of the expected strength of the members or components intended to undergo inelastic deformations (designated yielding members). To this end, the expected yield stresses of various steel materials have been established by a survey of mill certificates, and the ratio of expected to nominal yield stress has been included in the Provisions as R_y. The expected capacity of the designated yielding member is defined as R_y times the nominal strength of the member based on the desired yield mode. This expected strength is amplified to account for strain-hardening in some cases. For determination of the required strength of adjoining elements and their connection to the designated yielding members, neither the resistance factor (LRFD), nor the safety factor (ASD), are applied to the strength of the designated yielding members.

Where the capacity-design methodology is employed to preclude nonductile or unintended yielding modes of failure within the designated yielding member, it is reasonable to use the expected material strength in the determination of the element capacity. For unintended yield limit states, the factor R_y applies to the determination of available strength just as it applies to the determination capacity for the designated yielding member capacity used to compute the required strength and to the strength with respect to the limit states to be precluded. An example of this condition is the design of the beam outside the link in an eccentrically braced frame for the yield limit states. The required strength is based on the capacity of the link beam. The yield limit states of the beam outside the link, such as combined flexure and compression, can be expected to be similarly affected by increased material strength, thus the factor R_y is applied when determining the available strength. The factor R_y is not applied to elements other than the designated yielding element.

Similarly, rupture limit states within the designated yielding element are affected by increased material strength. An example of such limit states include block shear rupture and net section rupture of braces in special concentrically braced frames, where the required strength is calculated based on the brace capacity in tension. The ratio of expected tensile strength to specified minimum tensile strength is often different from that of expected yield stress to specified minimum yield stress, so a separate factor was created called R_t. This factor applies only to rupture limit states in designated yielding members. As is the case with R_y, R_t is applied in the determination of the expected strength of designated yielding members and not the available strength of other members.

The specified values of R_y for rolled shapes are somewhat lower than those that can be calculated using the mean values reported in a survey conducted by the Structural Shape Producers Council. Those values were skewed somewhat by the inclusion of a large number of smaller members, which typically have a higher measured yield stress 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. The expected yield stress, R_yF_y, can be determined by testing conducted in accordance with the requirements for the specified grade of steel. Such an approach should only be followed in unusual cases where there is extensive evidence that the values of R_y are significantly unconservative. It is not expected that this would be the approach followed for typical building projects. Refer to ASTM A370 for testing requirements. The higher values of R_y for ASTM A36/A36M ($R_y = 1.5$) shapes are indicative of the most recently reported properties of these grades of steel. The values of R_y will be periodically monitored to ensure that current production practice is properly reflected.

Two studies (Liu et al., 2007 and Liu, 2016) were used in determining the R_t values shown in Table A3.1. These values are based on the mean value of R_t/R_y for individual samples. Mean values are considered to be sufficiently conservative for these calculations considering that they are applied along with a ϕ factor of 0.75. An additional analysis of tensile data was carried out (Harrold, 2004) to determine appropriate R_y and R_t factors for ASTM A529/A529M Grade 50 (345), A529/A529M Grade 55 (380), A1011/A1011M HSLAS Grade 55 (380), and A572/A572M Grade 55 (380) steels that were added to Table A3.1.

In this edition of the Provisions, R_y and R_t values for HSS members have been refined based on the most recent research (Liu, 2016). ASTM A500/A500M Grade B, ASTM A500/A500M Grade C, and ASTM A501/A501M have been given individual values and ASTM A1085/A1085M has been added to Table A3.1. ASTM A501/A501M material has shown through limited testing to have R_y values less than those specified in Table A3.1 as this material is not cold worked as is ASTM A500/A500M material. Presently, ASTM A501/A501M material is not as commonly used nor as readily available as ASTM A500/A500M (Grades B or C). Due to the limited production data available for ASTM A501/A501M, these Provisions continue to conservatively use R_y and R_t values for ASTM A501/A501M based primarily on ASTM A500/A500M (Grades B or C) production data.

ASTM A572/A572M Grade 42 (290) shapes are no longer commonly produced. However, thick plate sections of this material grade are still used for connections, built-up shapes, and column bases. As limited production data is available for plates of this material grade, a value of R_y of 1.3 is specified corresponding to approximately the same 55 ksi (380 MPa) expected yield stress as ASTM A572/A572M Grade 50 (345) plate. The R_t value of 1.0 specified for plates of this material grade considers the expected tensile strength, R_tF_u, of the material to be the same as the specified tensile strength, F_u, which is conservative when used for determining nominal strength, R_n, limit states.

Values of R_y and R_t for ASTM A1043/A1043M Grades 36 (250) and 50 (345) are included based on a survey of production data.

Recent extensive unpublished data from American reinforcing bar producers indicate $R_y = 1.18$, $R_t = 1.17$, and $F_u/F_y = 1.50$ for A615/A615M Grade 60 (420), and $R_y = 1.11$, $R_t = 1.16$, and $F_u/F_y = 1.39$ for A615/A615M Grade 75 (520). Similarly, $R_y =$

1.14, R_t = 1.18, and F_u/F_y = 1.38 for A706/A706M Grade 60 (420) are expected. These values are meant for new construction and American-produced bars, and do not apply to other grades or specifications.

3. Heavy Sections

The *Specification* requirements for notch toughness cover hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and plate elements with thickness that is greater than or equal to 2 in. (50 mm) in tension applications. In the Provisions, this requirement is extended to cover: (1) shapes that are part of the SFRS with flange thickness greater than or equal to 1½ in. (38 mm); and (2) plate elements with thickness greater than or equal to 2 in. (50 mm) that are part of the SFRS, such as the flanges of built-up girders and connection material subject to inelastic strain under seismic loading. Because smaller shapes and thinner plates are generally subjected to sufficient cross-sectional reduction during the rolling process such that the resulting notch toughness will exceed that required (Cattan, 1995), specific requirements have not been included herein.

Connection plates in which inelastic strain under seismic loading may be expected include, but are not limited to:

1. Gusset plates for diagonal braces that are designed to allow rotation capacity per Section F2.6c.3(b)
2. Bolted flange plates for moment connections such as per ANSI/AISC 358 Chapter 7 (bolted flange plate moment connection) and similar flange plate moment connections in ordinary moment frame (OMF) systems
3. Bolted end plates for moment connections such as per ANSI/AISC 358 Chapter 6
4. Base plates of column bases designed to yield inelastically to limit forces on anchor rods or to allow column rotation

Early investigations of connection fractures in the 1994 Northridge earthquake identified a number of fractures that some speculated were the result of inadequate through-thickness strength of the column flange material. As a result, in the period immediately following the Northridge earthquake, a number of recommendations were promulgated that suggested limiting the value of through-thickness stress demand on column flanges to ensure that through-thickness yielding did not initiate in the column flanges. This limit state often controlled the overall design of these connections. However, the actual cause for the fractures that were initially thought to be through-thickness failures of the column flange are now considered to be unrelated to this material property. Detailed fracture mechanics investigations conducted as part of the FEMA/SAC project confirm that damage initially identified as through-thickness failures is likely to have occurred as a result of certain combinations of filler metal and base material strength and notch toughness, conditions of stress in the connection, and the presence of critical flaws in the welded joint. In addition to the analytical studies, extensive through-thickness testing conducted specifically to determine the susceptibility to through-thickness failures of column materials

meeting ASTM A572/A572M Grade 50 and ASTM A913/A913M Grade 65 specifications did not result in significant through-thickness fractures (FEMA, 2000g).

In addition, none of the more than 100 full-scale tests on "post-Northridge" connection details have demonstrated any through-thickness column fractures. This combined analytical and laboratory research clearly shows that due to the high restraint inherent in welded beam flange-to-column flange joints, the through-thickness yield and tensile strengths of the column material are significantly elevated in the region of the connection. For the materials tested, these strengths significantly exceed those loads that can be delivered to the column by the beam flange. For this reason, no limits are suggested for the through-thickness strength of the base material by the FEMA/SAC program or in these Provisions.

The preceding discussion assumes that no significant laminations, inclusions or other discontinuities occur in regions adjacent to welded beam flange-to-column flange joints and other tee and corner joints. Section J6.2c checks the integrity of this material after welding. A more conservative approach would be to ultrasonically test the material for laminations prior to welding. A similar requirement has been included in the Los Angeles City building code since 1973; however, in practice the base material prior to welding generally passes the ultrasonic examination, and interior defects, if any, are found only after heating and cooling during the weld process. Should a concern exist, the ultrasonic inspection prior to welding should be conducted in accordance with ASTM A435/A435M for plates and ASTM A898/A898M, level 1, for shapes.

4. Consumables for Welding

As in previous Provisions, specified levels of filler metal and weld metal Charpy V-notch (CVN) toughness are required in all member and connection welds in the load path of the SFRS.

The Provisions designate certain welds as demand critical welds, and require that these welds be made with filler metals that meet minimum levels of CVN toughness using two different test temperatures and specified test protocols, unless otherwise exempted from testing. Welds designated as demand critical welds are identified in the section of the Provisions applicable to the specific SFRS. Demand critical welds are generally complete-joint-penetration groove (CJP) welds so designated because they are subjected to yield level or higher stress demand and located in a joint whose failure would result in significant degradation in the strength or stiffness of the SFRS.

For demand critical welds, FEMA 350 (FEMA, 2000a) and 353 (FEMA, 2000b) recommended filler metal that complied with minimum Charpy V-notch (CVN) requirements using two test temperatures and specified test protocols. Previous editions of the Provisions included the dual CVN requirement suggested in the FEMA documents but required a lower temperature than the FEMA recommendations for the filler metal classification [–20°F (–29°C) rather than 0°F (–18°C)]. The use of this lower temperature was consistent with the filler metal used in the SAC/FEMA tests and matched the filler metals frequently used for such welds at the time the

testing was conducted. The filler metal classification requirement was revised in the 2010 edition of the Provisions to reflect the original FEMA recommendation and AWS D1.8/D1.8M requirements because filler metals classified at either temperature ensure that some ductile tearing would occur before final fracture, and because the more critical CVN weld metal property is the minimum of 40 ft-lb (54 J) at 70°F (21°C), as determined in AWS D1.8/D1.8M Annex A. This change now permits the use of common welding processes and filler metals, such as GMAW and SAW filler metals that are frequently classified for 20 ft-lb (27 J) at 0°F (−18°C).

In a structure with exposed structural steel, an unheated building, or a building used for cold storage, the demand critical welds may be subject to service temperatures less than 50°F (10°C) on a regular basis. In these cases, the Provisions require that the minimum qualification temperature for AWS D1.8/D1.8M Annex A be adjusted such that the test temperature for the Charpy V-notch toughness qualification tests be no more than 20°F (11°C) above the lowest anticipated service temperature (LAST). For example, weld metal in a structure with a LAST of 0°F (−18°C) would need to be qualified at a test temperature less than or equal to 20°F (−7°C) and −50°F (−46°C) in lieu of 70°F (21°C) and 0°F (−18°C), respectively. For purposes of the Provisions, the LAST may be considered to be the lowest one-day mean temperature (LODMT) compiled from National Oceanic and Atmospheric Administration (NOAA) data.

All other welds in members and connections in the load path of the SFRS require filler metal with a minimum specified CVN toughness of 20 ft-lbs (27 J) at 0°F (−18°C) using the AWS A5 classification. Manufacturer certification may also be used to meet this CVN requirement. Welds carrying only gravity loads, such as filler beam connections and welds for collateral members of the SFRS such as deck welds, minor collectors, and lateral bracing, do not require filler metal meeting these notch toughness requirements.

It is not the intent of the Provisions to require project-specific CVN testing of either the welding procedure specification (WPS) or any production welds. Further, these weld notch toughness requirements are not intended to apply to electric resistance welding (ERW) and submerged arc welding (SAW) when these welding processes are used in the production of hollow structural sections and pipe, such as ASTM A500/A500M and A53/A53M.

5. Concrete and Steel Reinforcement

The limitations on structural steel grades used in composite construction are the same as those given in Sections A3.1 and D2. The limitations in Section A3.5 on concrete and reinforcing bars are the same as those specified for the seismic design of reinforced concrete structures in the *Building Code Requirements for Structural Concrete*, ACI 318 Chapter 18 (ACI, 2014). While these limitations are particularly appropriate for construction in seismic design categories D, E and F, they apply in any seismic design category when systems are designed with the assumption that inelastic deformation will be required.

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

1. General

To ensure proper understanding of the contract requirements and the application of the design, it is necessary to identify the specific types of seismic force-resisting system (SFRS) or systems used on the project.

The special design, construction and quality requirements of the Provisions Chapter J, compared to the general requirements of the *Specification* Chapter N, are applicable to the SFRS. The additional quality control and quality assurance requirements of Chapter J are prepared to address the additional requirements for the SFRS, not the structure as a whole. Therefore, it is necessary to clearly designate which members and connections comprise the SFRS.

The protected zone includes regions anticipated to undergo significant inelastic deformations and often the areas immediately around those regions. Unanticipated connections, attachments or notches may interfere with the anticipated location and distribution of inelastic deformations, or initiate a fracture. Because the location of the protected zone may vary depending on member and connection configuration, the extent of the protected zone must be identified.

Fabricators commonly have shop drawings that show the locations of the protected zones with the piece during the time on the shop floor. Those working on the piece are expected to be knowledgeable of protected zones and their restrictions. Similarly, the locations of protected zones are shown on the erection drawings. Should the fabricator's or erector's personnel fail to heed the protected zone restrictions, the quality control inspector (QCI) is expected to identify the error. When required, quality assurance (QA) inspection of protected zones also is performed, using the design drawings that identify the protected zones.

AISC *Code of Standard Practice for Steel Buildings and Bridges*, ANSI/AISC 303 (AISC, 2016c) Section 1.11 requires that protected zones be permanently marked by the fabricator and re-marked by the owner's designated representative for construction if those markings are obscured in the field, such as by application of fireproofing. Marking and re-marking is important because the structural steel quality control inspector (QCI) and quality assurance inspector (QAI) have finished their tasks and are no longer present as the work of other trades (e.g., curtainwall, plumbing, electrical, HVAC, column covers, and partitions) is being performed. It also is important for subsequent remodeling or renovation of the structure over its life, particularly when design drawings are no longer available.

Floor and roof decks may be designed to serve as diaphragms and transfer seismic loads, and additional connection details may be needed to provide this load transfer. Consideration should also be made for other floor and roof deck connections when the deck has not been specifically designed and detailed as a diaphragm, as the system may behave as one.

2. Steel Construction

(a) It is necessary to designate working points and connection types, and any other detailing requirements for the connections in the SFRS.

(b) Information should be provided as to the steel specification and grade of the steel elements that comprise the connection, the size and thickness of those elements, weld material size, strength classification and required CVN toughness, and bolt material diameter and grade, as well as bolted joint type.

(c) Demand critical welds are identified in the Provisions for each type of SFRS. Demand critical welds have special Charpy V-notch (CVN) toughness and testing requirements to ensure that this notch toughness will be provided.

(d) Where SCBF brace connections are designed to provide rotation capacity to accommodate buckling in accordance with Section F2.6c.3(b), special detailing is required. These connections must be identified in the structural design drawings.

(f) The majority of welded connection applications in buildings are in temperature-controlled settings. Where connections are subjected to temperatures of less than 50°F (10°C) during service, additional requirements for welding filler metals are necessary for demand critical welds to ensure adequate resistance to fracture at the lower service temperatures.

(g) The presence of backing may affect the flow of stresses within the connection and contribute to stress concentrations. Therefore, backing removal may be required at some locations. Removal of backing should be evaluated on a joint specific basis, based upon connection prequalification requirements or qualification testing. AWS D1.8/D1.8M provides details for weld backing removal, additional fillet welds, weld tab removal, tapered transitions, and weld access holes.

(h) Where steel backing remains in place in tee and corner joints with the load applied perpendicular to the weld axis, a fillet weld between the backing and the flange element of the tee or corner joint reduces the stress concentration at the weld root. The requirement for this fillet weld should be evaluated on a joint specific basis, based upon connection prequalification requirements or qualification testing for moment connections, and the requirements of the Provisions for column-to-base plate connections. AWS D1.8/D1.8M provides details for additional fillet welds at weld backing.

(i) In tee and corner joints where loads are perpendicular to the weld axis, a reinforcing fillet weld applied to a CJP groove weld reduces the stress concentration at the corner between the weld face or root and the member. AWS D1.8/D1.8M provides details for reinforcing fillet welds. Such reinforcement is not required for most groove welds in tee or corner joints.

(j) The presence of weld tabs may affect the flow of stresses within the connection and contribute to stress concentrations. In addition, weld starts and stops made on weld tabs typically contain welds of lesser quality and are not subjected to

nondestructive testing. Therefore, complete or partial weld tab removal may be required at some locations. Removal of weld tabs should be evaluated on a joint-specific basis, based upon connection prequalification requirements or qualification testing. AWS D1.8/D1.8M provides details for weld tab removal.

(k) AWS D1.8/D1.8M provides details for tapered transition when required for welded butt joints between parts of unequal thickness and width.

(l) Analysis and research regarding the use of weld access holes have shown that the shape of the weld access hole can have a significant effect on the behavior of moment connections. The selection of weld access hole configuration should be evaluated on a joint-specific basis, based upon connection prequalification requirements or qualification testing. The use of different weld access holes other than those prescribed by AWS D1.1/D1.1M or the *Specification* has not been found necessary for specific moment connection types, nor necessary for locations such as column splices and column-to-base plate connections. Care should be exercised to avoid specifying special weld access hole geometries when not justified. In some situations, weld access holes are undesirable, such as in end plate moment connections.

(m) In typical structural frame systems, the specification of specific assembly order, welding sequence, welding technique, or other special precautions beyond those provided in this document should not be necessary. Such additional requirements would only be required for special cases, such as those of unusually high restraint.

3. Composite Construction

Structural design drawings and specifications, shop drawings and erection drawings for composite steel-concrete construction are basically similar to those given for all-steel structures. For the reinforced concrete portion of the work, in addition to the requirements in ACI 318 Chapter 26, attention is called to the ACI *Detailing Manual* (ACI, 2004b), with emphasis on Section 2.10, which contains requirements for seismic design of frames, joints, walls, diaphragms and two-way slabs.

CHAPTER B
GENERAL DESIGN REQUIREMENTS

B1. GENERAL SEISMIC DESIGN REQUIREMENTS

When designing structures to resist earthquake motions, each structure is categorized based upon its occupancy and use to establish the potential earthquake hazard that it represents. Determining the available strength differs significantly in each specification or building code. The primary purpose of these Provisions is to provide information necessary to determine the required and available strengths of steel structures. The following discussion provides a basic overview of how several seismic codes or specifications categorize structures and how they determine 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 *Minimum Design Loads for Buildings and Other Structures,* ASCE/SEI 7 (ASCE, 2016), structures are assigned to one of four risk categories. Category IV, for example, includes essential facilities. Structures are then assigned to a seismic design category based upon the risk categories and the seismicity of the site adjusted by soil type. Seismic design categories B and C are generally applicable to structures with moderate seismic risk, and special seismic provisions like those in these Provisions are optional. However, special seismic provisions are mandatory in seismic design categories D, E and F, which cover areas of high seismic risk, unless stated otherwise in ASCE/SEI 7.

B2. LOADS AND LOAD COMBINATIONS

The Provisions give member and element load requirements that supplement those in the applicable building code. In order to accommodate both LRFD and ASD, the 2005 edition of the Provisions (AISC, 2005) was the first to provide two "available strengths," one for LRFD and one for ASD. "Available strength" is the term used in the *Specification* to cover both design strength (LRFD) and allowable strength (ASD).

In some instances, the load effect defined in the Provisions must be combined with other loads. In such cases, the Provisions simply define the seismic load effect, which is combined with other loads using the appropriate load factor from the seismic load combinations in the applicable building code, and thus both LRFD and ASD are supported.

The Provisions are intended for use with load combinations given in the applicable building code. However, since they are written for consistency with the load combinations given in ASCE/SEI 7 and the 2018 *International Building Code* (ICC, 2018), consistency with the applicable building code should be confirmed if another building code is applicable.

The engineer is expected to use these Provisions in conjunction with the *Specification*. Typically, the Provisions do not define available strengths as these are given in the *Specification*. Additionally, the designer is directed to specific limit states or provisions in the *Specification* in certain cases.

An overstrength factor, Ω_o, applied to the horizontal portion of the earthquake load, E, is prescribed in ASCE/SEI 7, the IBC, the NEHRP Provisions (FEMA, 2015) and the *Building Construction and Safety Code*, NFPA 5000 provisions (NFPA, 2018). However, these codes do not all express the load combinations that incorporate this factor in exactly the same format. In the future, if all codes adopt ASCE/SEI 7 by reference, it will be possible to directly reference the appropriate combinations within these Provisions.

These Provisions require the consideration of system overstrength for many elements. System overstrength effects on the required strength of such elements are addressed in two ways. For some elements, it is sufficient to approximate the effect using the overstrength factor for the system given in ASCE/SEI 7 Table 12.2-1. For other elements, this approximate method is not sufficient and a more explicit calculation of required strength based on the expected or probable strength of adjoining elements is required. This latter approach has been used in previous editions of these Provisions and is now addressed by ASCE/SEI 7 Section 12.4.3.2 and termed the "capacity-limited horizontal seismic load effect." Per ASCE/SEI 7 Section 12.4.3.1, where consideration of overstrength is required but the capacity-limited seismic load is not, the approximate method based on the system's overstrength factor is permitted. Loads determined using this approximate method need never be taken as larger than those calculated using the capacity-limited seismic load. In either method of addressing system overstrength, the horizontal seismic load effects are combined with vertical seismic and gravity load effects using the load combinations in ASCE/SEI 7 to obtain the required strength. The capacity-limited horizontal seismic load effect, E_{cl}, is intended to have a load factor of 1.0 for LRFD and 0.7 for ASD applied in the applicable ASCE/SEI 7 load combinations.

In some cases, the total load on an element (typically a connection) is limited by the yielding of an adjacent member. In such cases, these provisions directly specify the required strength of the element (both for ASD and for LRFD terms) and no combination is made with gravity loads.

The calculation of seismic loads for composite systems per the ASCE/SEI 7 provisions is the same as is described previously for steel structures. The seismic response modification coefficient, R, and the deflection amplification factor, C_d, for some structural systems have been changed in ASCE/SEI 7 to make them more consistent with similar systems in structural steel only and reinforced concrete only systems. 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, where specific loading requirements are not specified in the applicable building code for composite systems, appropriate values for the seismic response modification coefficient can be

inferred from specified values for steel and/or reinforced concrete systems. These are predicated upon meeting the design and detailing requirements for the composite systems specified in these Provisions. Unlike the requirements for steel systems, for composite systems that include reinforced concrete members, the design loads and the corresponding design strengths are limited to those defined based on load and resistance factor design. This is done to ensure consistency between provisions for steel, composite and reinforced concrete members that are designed in accordance with the *Specification* and the *Building Code Requirements for Structural Concrete,* ACI 318 (ACI, 2014).

B3. DESIGN BASIS

2. Available Strength

It is intended that nominal strengths, resistance and safety factors, and available strengths of steel and composite members in the seismic force resisting system (SFRS) be determined in accordance with the *Specification*, unless noted otherwise in the Provisions. For reinforced concrete members in the SFRS, it is intended that they be designed in accordance with ACI 318.

B5. DIAPHRAGMS, CHORDS AND COLLECTORS

1. General

Seismic design requires that components of the structure be connected or tied together in such a manner that they behave as a unit. Diaphragms and their connections are an important structural element for creating this interconnection and contribute to lateral force resisting system performance in the following ways:

- connect the distributed mass of the building to the vertical elements of the seismic force resisting system (braced frames, moment frames or shear walls);
- interconnect the vertical elements of the seismic force resisting system, thus completing the system for resistance to building torsion;
- provide lateral stability to columns and beams including non-seismic force-resisting system columns and beams; and
- provide out-of-plane support for walls and cladding.

The elements that make up a diaphragm are generally already present in a building to carry other loads, such as gravity loads.

For recommendations on the design of diaphragms, see Sabelli et al. (2011).

In order for the seismic systems defined in the *Provisions* to provide ductility, the system must have capacity to deliver forces to the frames corresponding to the frame strength. For this reason ASCE/SEI 7 requires collectors to be designed for the over-strength seismic load in seismic design categories C through F.

2. Truss Diaphragms

In some structure types, a horizontal truss is used in lieu of a steel deck or composite diaphragm. In such cases, there is typically an orthogonal grid of beams with diaphragm-shear deformations resisted by members that are diagonal in plan.

ASCE/SEI 7 does not provide prescriptive direction on how to consider horizontal truss diaphragms. Although there is a school of thought that diagonal and cross brace members could be allowed to buckle or hinge as a source of additional energy absorption, the Provisions requires that these elements be designed for the overstrength seismic load in accordance with the capacity-limited design approach of the Provisions, unless the exceptions of Section B5.2 are met.

Two exceptions are provided to the requirement in Section B5.2. In the first exception, the horizontal truss is expected to provide ductility. In this case the members that are diagonal in plan are treated similarly to braces in SCBF, with the orthogonal beam system acting as the SCBF beams and columns. Under this exception, the beams are designed using the overstrength seismic load and the diagonal members for the basic load combinations. The second exception is for a three-dimensional analysis for ordinary systems (OMF and OCBF) in which the diaphragm is treated similarly to an OCBF and the diagonal members are treated similarly to braces.

CHAPTER C
ANALYSIS

C1. GENERAL REQUIREMENTS

For nonseismic applications, story drift limits like deflection limits are commonly used in design to ensure the serviceability of the structure. These limits vary because they depend upon the structural usage and contents. As an example, for wind loads 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 *Specification*.

The situation is somewhat different when considering seismic effects. Research has shown that story drift limits improve frame stability (P-Δ effects) and seismic performance because of the resulting 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 how the limit is specified and applied. 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 analytical model used to estimate building drift should accurately account for the stiffness of the frame elements and connections and other structural and nonstructural elements that materially affect the drift. Recent research on steel moment frame connections indicates that in most cases the effect of panel zone deformations on elastic drift can be adequately accounted for by modeling beams to extend between column centerlines without rigid end offsets, and that explicit panel zone modeling is not required (FEMA, 2000f). In cases where nonlinear element deformation demands are of interest, panel zone shear behavior should be represented in the analytical model whenever it significantly affects the state of deformation at a beam-to-column connection. Mathematical models for the behavior of the panel zone in terms of shear force-shear distortion relationships have been proposed by many researchers. FEMA 355C presents a good discussion of how to incorporate panel zone deformations into the analytical model (FEMA, 2000d).

Adjustment of connection stiffness is usually not required for connections traditionally considered as fully restrained, although FEMA 350 (FEMA, 2000a) contains recommendations for adjusting calculated drift for frames with reduced beam sections. Nonlinear models should contain nonlinear elements where plastic hinging is expected to properly capture the inelastic deformation of the frame. Where partially restrained connections are used, analytical models must adequately reflect connection stiffness in both the elastic and inelastic range.

For composite systems that include composite members or steel members combined with reinforced concrete, the properties of the composite and concrete members

should be modeled to represent the effects of concrete cracking. For design by elastic analysis, the composite and concrete member properties should reflect the effective stiffness of the members at the onset of significant yielding. The following guidance is provided for calculating effective stiffness values for design by elastic analysis:

(1) In concrete beam and column members, stiffness properties for elastic analysis are typically specified as a fraction of the flexural stiffness, EI_g, where E is the elastic modulus of concrete and I_g is the gross moment of inertia. For concrete frames, ACI 318 Section 6.6.3.1.1 (ACI, 2014) recommends effective stiffness values ($EI_{effective}$) in the range of 0.25 to 0.50EI_g for beams and 0.35 to 0.875EI_g for columns, or as justified by rigorous analysis. More detailed recommendations that account explicitly for axial load are given in ASCE 41, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2013) which recommends effective stiffness values of (a) 0.70EI_g for columns with unfactored gravity compressive loads that are greater than $0.5A_g f_c'$ (where A_g is the gross member area and f_c' is the concrete compressive strength) and (b) 0.30EI_g for columns (and beams) with axial gravity loads less than $0.1A_g f_c'$. Linear interpolation of stiffness is suggested for axial loads between 0.1 and $0.5A_g f_c'$.

(2) For concrete walls, ACI 318 Section 6.6.3.1.1 recommends effective stiffness values between 0.35EI_g and 0.875EI_g, or as justified by rigorous analysis. The walls above the hinged region are typically expected to remain essentially elastic. For these regions and walls that are anticipated to remain in the elastic range, the cracked section properties for the walls may be taken as 0.70EI_g and 1.0EA_g. ASCE 41 also includes recommendations, which are deemed to be conservative for new composite ordinary shear walls.

(3) For concrete-encased or concrete-filled beam-columns, the effective stiffness may be specified based on the use of a cracked transformed section [see, e.g., Ricles and Paboojian (1994); Varma et al. (2002)]. Attention should be paid to the relative values of the girder versus beam-column effective stiffnesses.

(4) For steel beams with composite slabs in which the shear connection between the beam and slab is such that the contribution of the composite slab can be included in the stiffness and subject to reverse curvature due to earthquake loading, a reasonable assumption is to specify a flexural stiffness that is equal to the average of the composite beam stiffness in positive bending and bare steel beam stiffness in negative bending. Assuming that the beams are designed to have full composite action, it is suggested to take the effective stiffness as equal to $0.5(E_s I_s + E_s I_{tr})$, where E_s is the steel modulus, I_s is the moment of inertia of the bare steel beam, and I_{tr} is the transformed moment of inertia of the beam and slab. The effective width of the slab can be determined in accordance with *Specification* Chapter H.

Any of the elastic methods in *Specification* Chapter C or Appendix 7 can be used to assess the stability of frames in high seismic regions. When using the equivalent lateral load procedure for seismic design and the direct analysis provisions in *Specification* Chapter C, the reduced stiffness and notional load provisions should not be

included in the calculation of the fundamental period of vibration or the evaluation of seismic drift limits.

Like most of the provisions in the *Specification*, the stability requirements are intended for cases where the strength limit state is based on the nominal elastic-plastic limit in the most critical members and connections (e.g., the "first hinge" limit point), not to ensure stability under seismic loads where large inelastic deformations are expected. Thus, the provisions of *Specification* Chapter C do not alone ensure stability under seismic loads. Stability under seismic loads is synonymous with collapse prevention, which is provided for in the prescriptive design requirements given for each system, including such elements as:

(1) The basic determination of the seismic design force (R factors, site effects, ρ factors, etc.)

(2) The drift limits under the seismic lateral load (a factor of both the limiting drift and the specified C_d factor)

(3) The "theta" limits (sidesway stability collapse prevention)

(4) Other design requirements, such as strong-column weak-beam requirements, limitations on bracing configurations, etc.

C2. ADDITIONAL REQUIREMENTS

The analysis requirements of ASCE/SEI 7 are general with the primary intent of provisioning for stability, in part by developing minimum design forces for a variety of systems. Required strength relates to a sufficient first-yield strength within the system. While limitations on system irregularity help to avoid unexpected or known undesirable behavior, the requirements of ASCE/SEI 7 do not ensure a well-proportioned system with controlled or distributed yielding. The Provisions are intended to expand on the basic requirements of ASCE/SEI 7 to provide a well-proportioned system with controlled yielding and large inelastic drift capacity. This is accomplished to varying degrees depending on the intended ductility of the system by promoting inelastic activity in designated components, while limiting inelastic activity elsewhere. The required strength of designated yielding members (DYM) or components is determined by elastic analysis methods for the prescribed load combinations, while that of other elements which are intended to remain essentially elastic is determined by pseudo-capacity design approach which varies from system to system.

An alternative to using elastic analysis is to use the plastic design method as a more direct way to achieve the objective of a desired yield mechanism for the structural system (Goel and Chao, 2008). In the plastic design approach, the desired yield mechanism is first selected by identifying the DYM and those that are intended to remain elastic, designated as non-DYM. The required strength of the DYM is determined by using a mechanism-based plastic analysis for each appropriate load combination. Any expected overstrength of the DYM or structure beyond the elastic limit up to the formation of targeted yield mechanism (within its maximum deformation limit) must

be properly considered in the analysis. The second step of determining the required strength of non-DYM can be carried out by one of the following possible methods:

(1) A static elastic analysis of suitably selected structural subassemblages consisting of non-DYM with loads applied to keep them in equilibrium under the expected forces from the DYM and other applicable loads.

(2) A nonlinear static pushover analysis of the entire structure up to a target drift level by modeling the DYM to behave inelastically, while the non-DYM are modeled (or "forced") to behave elastically in order to be able to determine their required strength.

(3) A nonlinear dynamic analysis of the structure as modeled for the pushover analysis mentioned previously, using an appropriately selected ensemble of ground motions.

Typical seismic analysis of structures uses applied external loads. The Specification requires that second-order effects be considered in order to arrive at appropriate member design forces. These second-order effects consist of magnification of member forces due to the presence of gravity load acting through the sidesway displacement of the structure (P-Δ effect) and magnification of member moments due to the presence of member axial force (P-δ effect).

Determining the required strength of non-DYM is the same in the capacity design and plastic mechanism design methods. In a static elastic analysis approach, a set of forces that represent the fully yielded capacity of the DYM, applicable gravity loads, and lateral forces (as required for equilibrium) are applied on appropriately selected portions of the structure. P-Δ corrections (such as notional lateral loads or the B_2 factor) are not applicable as those effects are represented in the calculated lateral forces. The P-Δ effect can be thought of as having contributed to the formation of the fully yielded condition. P-δ effects are not relieved by the formation of the plastic mechanism, and where such effects occur, adjustments (such as the B_1 factor) must be applied in order to arrive at appropriate design forces.

C3. NONLINEAR ANALYSIS

Nonlinear analysis may be used in the Provisions in certain situations (e.g., exception in Section E3.6g). Procedures such as those given in ASCE/SEI 7 should be followed unless a more rational method can be justified.

CHAPTER D

GENERAL MEMBER AND CONNECTION DESIGN REQUIREMENTS

D1. MEMBER REQUIREMENTS

1. Classification of Sections for Ductility

Members of the seismic force-resisting system (SFRS) that are anticipated to undergo inelastic deformation have been classified as either moderately ductile members or highly ductile members. During the design earthquake, moderately ductile members are anticipated to undergo moderate plastic rotation of 0.02 rad or less, whereas highly ductile members are intended to withstand significant plastic rotation of 0.04 rad or more. Member rotations result from either flexure or flexural buckling. The requirements for moderately ductile and highly ductile members apply only to those members designated as such in the Provisions.

1a. Section Requirements for Ductile Members

To provide for reliable inelastic deformations in those SFRS members that require moderate to high levels of inelasticity, the member flanges must be continuously connected to the web(s). This requirement does not preclude the use of members built up from plates or shapes. Built-up members shall comply with the requirements in the *Specification* and any additional requirements of these Provisions or ANSI/AISC 358 (AISC, 2016b) that are specific to the system or connection type being used.

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

Local buckling can result in very high localized strains that when repeated, such as in low-cycle fatigue caused by an earthquake, can result in premature fracture of a member that is intended to behave in a ductile manner. To provide for reliable inelastic deformations in those members of the SFRS that require moderate to high levels of inelasticity, the width-to-thickness ratios of compression elements should be less than or equal to those that are resistant to local buckling when stressed into the inelastic range. Table D1.1 provides width-to-thickness ratios that correspond to the anticipated level of inelastic behavior for both moderately ductile and highly ductile members. The limiting width-to-thickness ratios for moderately ductile members generally correspond to λ_p values in *Specification* Table B4.1b with exceptions for round and rectangular HSS, stems of WTs, and webs in flexural compression. Although the limiting width-to-thickness ratios for compact compression elements, λ_p, given in *Specification* Table B4.1b, are sufficient to prevent local buckling before the onset of strain-hardening, the available test data suggests that these limits are not adequate for the required inelastic performance of highly ductile members in the SFRS. The limiting width-to-thickness ratios for highly ductile members, λ_{hd}, given

in Table D1.1 are deemed adequate for the large ductility demands to which these members may be subjected (Sawyer, 1961; Lay, 1965; Kemp, 1986; Bansal, 1971).

This edition of the Provisions adds the R_y term to adjust the material strength to the expected material strength in the width-to-thickness equations in Table D1.1. It is common practice for materials to be certified for various material grades, some of which have significantly different yield strengths. I-shaped beams can be obtained with "dual certification" both as ASTM A36/A36M products and A992/A992M products. A36/A36M material that is not certified with multiple grades is still likely to have a yield stress near 50 ksi (345 MPa). A member sized using A36/A36M specified minimum yield stress might use a shape that meets width-to thickness requirements for a steel with $F_y = 36$ ksi (250 MPa), but not for a steel with $F_y = 50$ ksi (345 MPa). Given the likelihood the shape used in a structure might have an actual yield stress near 50 ksi (345 MPa), it could be subject to premature local buckling when experiencing inelastic deformations due to a significant seismic event. To account for this possibility, the R_y term has been incorporated into the width-to-thickness limits. The width-to-thickness equations have been recalibrated to provide nearly identical results with the expected yield strengths of the commonly used materials such as ASTM A572/A572M Grade 50 (345), ASTM A992/A992M, ASTM A913/A913M Grades 65 (450) and 70 (485), and ASTM A500/A500M Grade B.

For highly ductile members, the limiting width-to-thickness ratios for webs of rolled or I-shaped built-up beams and webs of built-up shapes used as beams or columns are based primarily on research on the effects of web slenderness on ductility under combined bending and axial compression under monotonic loading. The basis includes work by Haaijer and Thurlimann (1958), Perlynn and Kulak (1974), and Dawe and Kulak (1986). The current web slenderness limits were chosen to be consistent with those suggested by Dawe and Kulak (1986) with minor modifications.

For special moment frame (SMF) beams, the modifications provide results consistent with the recommendations of Uang and Fan (2001) and FEMA 350 (FEMA, 2000a) for cases where the axial force is zero. The limiting width-to-thickness ratios of stiffened webs for moderately ductile beam or column members correspond to those in *Specification* Appendix 1. For I-shaped beams in SMF and intermediate moment frames (IMF), the effects of axial compression on the limiting web slenderness ratio can be neglected when C_a is less than or equal to 0.114 (see footnote b of Table D1.1). This exception is provided because it is believed that small levels of axial compression, and its consequent effect on web buckling in beams, will be less detrimental to system performance than in columns.

Axial forces caused by the design earthquake ground motion may approach the available tensile strength of diagonal braces. In order to preclude local buckling of the webs of I-shaped members used as diagonal braces, the web width-to-thickness limit for nonslender elements for members subject to axial compression per *Specification* Table B4.1a must be met.

HSS members used as beams or columns designated as moderately ductile members are not anticipated to experience flexural buckling. Therefore, exceptions have

been added relaxing the width-to-thickness ratios to the λ_p values of *Specification* Table B4.1b (see footnote c of Table D1.1).

A small relaxation in the width-to-thickness ratio of the stem of tees used as highly ductile members is permitted for two cases (see footnote a of Table D1.1). The relaxed value corresponds to the λ_p value in *Specification* Table B4.1b. For the first case, where buckling is anticipated to occur about the plane of the stem, little inelastic deformation should occur in the stem itself. The second case takes advantage of a common practice for the connection of tees which is to bolt or weld a connection plate only to the outside of the flange of the tee with no connection to the web. Because the axial load is applied eccentrically to the neutral axis of the tee, a bending stress occurs that reduces the compressive stresses at the tip of the stem. Currently there is insufficient data or research on buckling of stems of tees to permit a more substantial relaxation for highly ductile members, nor to permit a relaxation for tees used as moderately ductile members.

During the service life of a steel H-pile, it is primarily subjected to axial compression and acts as an axially loaded column. Therefore, the b/t ratio limitations given in *Specification* Table B4.1 suffice. During a major earthquake, because of lateral movements of the pile cap and foundation, the steel H-pile becomes a beam-column and may have to resist large bending moments and uplift. Cyclic tests (Astaneh-Asl and Ravat, 1997) indicated that local buckling of piles satisfying the width-to-thickness limitations in Table D1.1 occurred after many cycles of loading. However, this local buckling did not have much effect on the cyclic performance of the pile during cyclic testing or after cyclic testing stopped and the piles were once again under only axial load. Previous editions of these Provisions required highly ductile sections for H-pile members. This requirement has been relaxed in this edition of the Provisions based on the width-to-thickness ratios of H-pile sections that performed well in tests (Astaneh-Asl et al., 1994; Astaneh-Asl and Ravat, 1997). See Commentary Section D4.1 for further discussion.

Previous editions of these Provisions required the link cross section in eccentrically braced frames (EBF) to meet the same width-to-thickness criteria as is specified for beams in SMF. Exceptions have been provided in Section F3.5b.1 that allow links to meet the width-to-thickness limits for moderately ductile members in certain conditions. See Commentary Section F3.5b.1 for further discussion.

The width-to-thickness criteria for composite members remain unchanged from the requirements in the 2010 Provisions.

2. Stability Bracing of Beams

The requirements for stability bracing of beams designated as moderately ductile members and highly ductile members are a function of the anticipated levels of inelastic yielding as discussed in Commentary Section D1.1 for members with these two designations.

2a. Moderately Ductile Members

The limiting requirement for spacing of stability bracing of $0.17r_yE/F_y$ for moderately ductile beam members has been modified to $0.19r_yE/(R_yF_y)$. For materials with an R_y of 1.1, there will be minimal change. For materials with a higher R_y, the equation will increase the requirement to reflect the higher expected yield stress. The revised equation results in the same limit specified in the 2010 Provisions for IMF beams, as the level of inelastic behavior in IMF beams is considered representative of moderately ductile beams. Since the minimum required story drift angle of an SMF system is twice that of an IMF system, the use of a less severe maximum stability spacing requirement for IMF beams that is twice that of SMF beams is appropriate. The commentary to Section D1.2b gives further discussion on stability bracing of beams.

In addition to point bracing, these provisions allow both point torsional bracing and panel bracing per *Specification* Appendix 6. While point torsional bracing is appropriate for beams with minimal or no compressive axial loads, beams with significant axial loads may require lateral bracing or lateral bracing combined with point torsional bracing to preclude axial buckling.

For calculating required bracing strength according to Equations A-6-5 and A-6-7 of *Specification* Appendix 6, the use of C_d =1 is justified because the Appendix 6 equations have an implicit assumption that the beams will be subjected to top flange loading. One can see this by comparing the *Specification* Equations A-6-5 and A-6-7 to the *Specification* Commentary Equations C-A-6-8a and C-A-6-8b, where the *Specification* equations are based on a conservative assumption of $C_t = 2$. In the case of seismic frames, where the moments are introduced via the beam-column connections, $C_t = 1$. Strictly speaking, the correct solution would be to use the commentary equation with $C_t = 1$ and $C_d = 1$ at all locations except for braces at the inflection point where $C_d = 2$. The current Provisions imply that the product of $C_t(C_d) = 2$ by the implied value of $C_t = 2$ and $C_d = 1$.

2b. Highly Ductile Members

Spacing of stability braces for highly ductile members is specified not to exceed $0.095r_yE/(R_yF_y)$. The R_y modifier has been incorporated to decrease the spacing of materials with R_y factors greater than 1.1 to adjust for their higher expected yield stress. This adjusted limitation provides identical results to the requirement in previous Provisions for beams in SMF as the degree of inelastic behavior is representative of highly ductile members. The spacing requirement for beams in SMF was originally based on an examination of lateral bracing requirements from early work on plastic design and based on limited experimental data on beams subject to cyclic loading. Lateral bracing requirements for SMF beams have since been investigated in greater detail in Nakashima et al. (2002). This study indicates that a beam lateral support bracing of $0.095Er_y/(R_yF_y)$ is appropriate, and slightly conservative, to achieve a story drift angle of 0.04 rad.

2c. Special Bracing at Plastic Hinge Locations

In addition to bracing along the beam length, the provisions of this section call for the placement of stability bracing to be near the location of expected plastic hinges of highly ductile members. Such guidance dates to the original development of plastic design procedures in the early 1960s. In moment frame structures, many connection details attempt to move the plastic hinge a short distance away from the beam-to-column connection. Testing carried out as part of the SAC program (FEMA, 2000a) indicated that the bracing provided by typical composite floor slabs is adequate to avoid excessive strength deterioration up to the required story drift angle of 0.04 rad. Therefore, the FEMA recommendations do not require the placement of supplemental lateral bracing at plastic hinge locations adjacent to column connections for beams with composite floor construction. These provisions allow the placement of lateral or torsional braces to be consistent with the tested connections that are used to justify the design. For conditions where drifts larger than 0.04 rad are anticipated or improved performance is desired, the designer may decide to provide additional stability bracing near these plastic hinges. If lateral braces are used, they should provide an available strength of 6% of the expected strength of the beam flange at the plastic hinge location. If a reduced beam section connection detail is used, the reduced flange width may be considered in calculating the bracing force. If point torsional braces are used, they should provide an available strength of 6% of the expected flexural strength of the beam at the plastic hinge. Placement of bracing connections should consider the protected zone requirements of Section D1.3.

3. Protected Zones

The FEMA/SAC testing has demonstrated the sensitivity of regions undergoing large inelastic strains to discontinuities caused by welding, rapid change of section, penetrations, or flaws caused during construction. For this reason, operations as specified in Section I2.1 that cause discontinuities are prohibited in regions subject to large inelastic strains. These provisions designate these regions as protected zones. The protected zones are designated in the Provisions in the sections applicable to the designated type of system and in ANSI/AISC 358. Some examples of protected zones include moment frame hinging zones, links of eccentrically braced frames (EBF), and the ends and center of SCBF diagonal braces.

Not all regions experiencing inelastic deformation are designated protected zones. For example, the beam-column panel zone of moment frame systems is not a protected zone. It should be noted that yield level strains are not strictly limited to the plastic hinge zones and caution should also be exercised in creating discontinuities in all regions.

4. Columns

4a. Required Strength

Columns in the SFRS are required to have adequate strength to resist specific loading requirements where specified in the applicable system chapter. Where the system

chapter does not have specific requirements, the columns must be adequate for load combinations of the applicable building code. In addition to meeting the system chapter and/or applicable building code requirements, the columns must also satisfy the requirements of Section D1.4a(b).

It is imperative that columns that are part of the SFRS have adequate strength to avoid global buckling or tensile rupture. Since the late 1980s, previous editions of the Provisions and other codes and standards have included requirements that are similar to those included in this section. The required forces for design of the columns are intended to represent reasonable limits on the axial forces that can be imposed. Design for these forces is expected to prevent global column failure. These axial forces are permitted to be applied without consideration of concurrent bending moments that may occur at column ends. Research has shown that columns can withstand high axial forces (up to $0.75F_y$) with significant end rotations due to story drift (Newell and Uang, 2008). The column design using these forces is typically checked using $K = 1.0$. This approach is based on the recognition that in the SFRS, column bending moments would be largest at the column ends and would normally result in reverse curvature in the column. This being the case, the bending moments would not contribute to column buckling, and the assumption of $K = 1.0$ would be conservative. However, bending moments resulting from a load applied between points of lateral support can contribute to column buckling and are therefore required to be considered concurrently with axial loads.

Clearly, the previously described approach provides no assurance that columns will not yield and the combination of axial load and bending is often capable of causing yielding at the ends of columns. Column yielding may be caused by a combination of high bending moments and modest axial loads, as is normal in moment frames; or by a combination of high axial load and bending due to the end rotations from story drift, as is normal in braced frame structures. While yielding of columns may result in damage that is significant and difficult to repair, it is judged that, in general, it will not result in column ruptures or global buckling, either of which would threaten life safety.

Although the provisions in Section D1.4a are believed to provide reasonable assurance of adequate performance, it should be recognized that these are minimum standards and there may be additional concerns where higher levels of performance, or greater levels of reliability are merited. For example, nonlinear analyses often indicate conditions wherein column end moments are not reversed and may contribute to buckling.

Where columns are part of intersecting frames in seismic design category (SDC) D, E and F, ASCE/SEI 7 requires that analyses include the effects of 100% of the design motions in one direction in conjunction with 30% of those in the orthogonal direction, or the simultaneous application of orthogonal pairs of ground motion acceleration histories. For systems with high R values, even the 30% design motion is likely capable of yielding the structure, and considering that the 100% motion may occur in any direction relative to a given axis of the structure, it is clear that simultaneous yielding of orthogonal systems is likely and should be considered in the design.

Determination of the need to combine axial forces from simultaneous yielding of intersecting frames is left as a matter of judgment. The extent to which simultaneous yielding of orthogonal lateral frames is of concern is a matter of configuration and design, and depends upon the expected deformations and the story drift at which the system used is expected to start yielding. Depending upon stiffness and overstrength, moment frames generally remain elastic until they reach 1% story drift, whereas braced frames generally will yield before reaching half that drift.

4b. Encased Composite Columns

The basic requirements and limitations for determining the design strength of reinforced concrete encased composite columns are the same as those in the *Specification*. Additional requirements for reinforcing bar details of composite columns that are not covered in the *Specification* are included based on provisions in ACI 318 (ACI, 2014). Examples for determining the effective shear width, b_w, of the reinforced concrete encasement are given in Figure C-D1.1.

Composite columns can be an ideal solution for use in seismic regions because of their inherent structural redundancy (Viest et al., 1997; El-Tawil and Deierlein, 1999). 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 stud anchors can cause difficulty in reinforcing bar placement and a potential for honeycombing of the concrete.

The required level of detailing is specified in Chapters G and H of the Provisions. Moderately ductile requirements are intended for seismic systems permitted in seismic design category C, and highly ductile requirements are intended for seismic systems permitted in seismic design categories D, E and F. Note that the highly

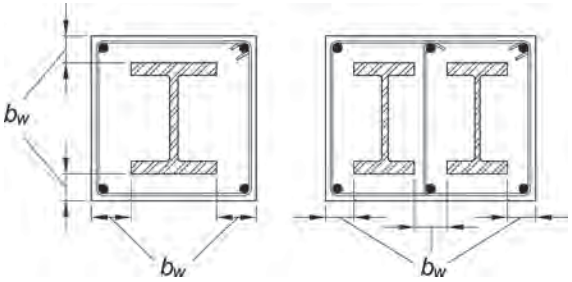


Fig. C-D1.1. Effective widths for shear strength calculation of encased composite columns.

ductile requirements apply to members of special seismic systems permitted in seismic design category D, E and F even if the systems are employed for use in lower seismic design categories.

1. Moderately Ductile Members

The more stringent tie spacing requirements for moderately ductile encased composite columns follow those for reinforced concrete columns in regions of moderate seismicity as specified in ACI 318 Chapter 18. 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, 2000e; ICC, 2015).

2. Highly Ductile Members

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 composite systems permitted in seismic design categories D, E and F (FEMA, 2009a; ICC, 2015). For additional explanation of these requirements, see Commentary Section D1.4a and ACI 318 Chapter 18.

The minimum area of tie reinforcement requirement in Equation D1-8 is based upon a similar provision in ACI 318 Chapter 18, except that the required tie area is reduced to take into account the steel core. The tie area requirement in Equation D1-8 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 composite special moment frames (C-SMF) are based upon similar requirements for steel and reinforced concrete columns in SMF (FEMA, 2009a; ICC, 2015). For additional commentary, see Commentary Section E3 and ASCE/SEI 7.

The strong-column/weak-beam 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-D1.2 to facilitate the placement of the reinforcement around the steel core. Ties are required to be anchored into the confined core of the column to provide effective confinement.

4c. 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 *Specification* Chapter I.

The shear strength of the filled member is conservatively limited to the nominal shear yield strength of the hollow structural section (HSS) 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. (750 mm). 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 relatively low ratios of structural steel to concrete areas (Furlong, 1997).

5. Composite Slab Diaphragms

In composite construction, floor and roof slabs typically consist of either composite or noncomposite metal deck slabs that are connected to the structural framing to provide an in-plane composite diaphragm that collects and distributes seismic loads. Generally, composite action is distinguished from noncomposite action on the basis of the out-of-plane shear and flexural behavior and design assumptions.

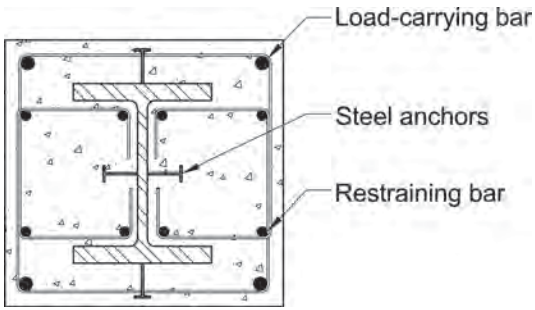


Fig. C-D1.2. Example of a closed hoop detail for an encased composite column.

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 due to vertical floor and roof loads. Design procedures for determining flexural and shear strength and codes of practice for such slabs are well established (ASCE, 1991a, 1991b; AISI, 2007; SDI, 2001, 2007, 2011).

Noncomposite 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. Noncomposite metal deck slabs, particularly those used as roofs, can be formed with metal deck that is capable of carrying all vertical loads and is overlaid with insulating concrete fill that is not relied upon for out-of-plane strength and stiffness. 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 plays a key role in collecting and distributing seismic loads to the seismic force-resisting systems and its design requires careful attention to establishing proper load paths and coherent detailing (Sabelli et al., 2011). In some cases, loads from other floors should also be included, such as at a level where a change in the structural stiffness results in 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 the applicable building code (SDI, 2001, 2004, 2007, 2011). In addition, research on composite diaphragms has been reported in the literature (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. (50 mm) and 6 in. (150 mm), measured shear stresses on the order of $3.5\sqrt{f_c'}$ (where $\sqrt{f_c'}$ is 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, 2014) using the concrete and reinforcement above the metal deck ribs and ignoring the beneficial effect of the concrete in the flutes.

Shear forces are typically transferred through welds and/or shear anchors in the collector and boundary elements. Where concrete fill is present, it is generally advisable to use mechanical devices such as steel headed stud anchors 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.

6. Built-Up Structural Steel Members

Shapes and plates may be joined to form built-up shapes where the combined shape behaves as an integral member for the magnitude and type of loading expected. ANSI/AISC 358 provides direction for built-up I-shapes and box columns when forming part of moment connections using prequalified connections. Section F2 provides

direction for built-up diagonal braces. Section F3 provides direction for built-up I-shaped and built-up box sections used as links.

Other systems may use built-up members comprised of joined plates and/or shapes provided that their connections are designed for the anticipated forces. Where inelastic deformation is expected in a member during a significant earthquake, the connections between elements shall be based on the forces due to that inelastic force level. The basis of design section in the system chapters typically indicates when inelastic deformation is expected and in which members or elements.

For example, an SCBF diagonal brace is typically required to be connected for its expected axial tension strength, $R_yF_yA_g/\alpha_s$. Furthermore, connections must accommodate brace buckling. Therefore, the direction of brace buckling must be determined. Interconnection of brace elements must address both the magnitude of load and the direction of loading.

The connection design strength requirement of diagonal braces in an ordinary concentrically braced frame (OCBF) is typically governed by forces arising from the load combinations including the overstrength seismic load. These end connection forces can therefore be used to determine the interconnection between the elements. Brace end gussets are not required to be designed for buckling in or out of plane.

For moment frames subject primarily to flexure, the horizontal shear between elements is a function of the vertical shear at the connection to the column face. The system chapters provide direction to determine this force. For example, Section E1 provides direction to determine the shear in the beam at the column face. This shear force can be used to determine the horizontal shear force between the flanges and web. Connections between elements of columns in moment frames must also be designed both for the horizontal shears between floors, and for the high horizontal shear in the column panel zone.

Where protected zones are specified, inelastic deformation is typically expected at that location. An example is the protected zone in a moment frame beam near the column face. The connection should develop the strength of the weaker element, typically the beam web. This can be accomplished by complete-joint-penetration groove welds or by two-sided fillet welds proportioned to develop the expected strength of the weaker element. Note that the fillet weld option is not permitted for built up shapes in moment connections governed by ANSI/AISC 358. An example of where fillet welds are permitted is in the protected zone of a special cantilever column systems column per Section E6.

D2. CONNECTIONS

1. General

Adequate behavior of connections of members in various systems in the SFRS is ensured by satisfying one of the following general conditions:

(1) Connections in some systems are verified by testing to ensure adequate performance (IMF, SMF beam-to-column connections, and BRBF brace-to-gusset connections, for example).

(2) Connections of members in some systems are designed to resist the required strength of the connected member or an adjoining member and therefore the maximum connection forces are limited by expected strength of a member (SCBF and BRBF diagonal braces and EBF links, for example).

(3) Connections of some members must be designed to resist forces based on the load combinations including the overstrength seismic load (column splices, collectors, and OCBF diagonal braces, for example).

A review of the requirements of these Provisions and ASCE/SEI 7 indicates that connections in the SFRS satisfy at least one of the preceding conditions. Therefore, the requirement in the 2005 Provisions that the design of a connection ensures a ductile limit state was deleted in the 2010 Provisions.

2. Bolted Joints

The potential for full reversal of design load and the likelihood of inelastic deformations of members and/or connected parts necessitates that pretensioned bolts be used in bolted joints in the SFRS. However, earthquake motions are such that slip cannot and need not be prevented in all cases, even with slip-critical connections. Accordingly, the Provisions call for bolted joints to be proportioned as pretensioned bearing joints but with faying surfaces prepared as for Class A or better slip-critical connections. That is, bolted connections can be proportioned with available strengths for bearing connections as long as the faying surfaces are still prepared to provide a minimum slip coefficient, $\mu = 0.30$. The resulting nominal amount of slip resistance may minimize damage in more moderate seismic events. This requirement is intended for joints where the faying surface is primarily subjected to shear. Where the faying surface is primarily subjected to tension or compression from seismic load effects, for example, in a bolted end plate moment connection, the requirement for preparation of the faying surfaces may be relaxed.

It is an acceptable practice to designate bolted joints as slip-critical as a simplified means of specifying the requirements for pretensioned bolts with slip-critical faying surfaces. However when the fabricator is permitted to design the connections, specifying that bolted joints must be designed as slip-critical may result needlessly in additional and/or larger bolts.

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 SFRS is limited to standard holes (including the new standard ⅛-in. hole clearance for bolts 1-in. diameter and larger) and short-slotted holes with the direction of the slot perpendicular to the line of force. For connections where there is no transfer of seismic load effect by shear in the bolts in the joint, oversized holes, short-slotted holes, and slotted holes are permitted. An example is a collector beam end connection using

an end-plate connection. The axial force in the beam due to seismic load effects is transferred by either tension in the end connection or by bearing of the beam end through the connection. Gravity loads are transferred by bolt shear, but not seismic load effects.

An exception is provided for alternative hole types that are justified as a part of a tested assembly. Additionally, an exception allows the use of oversized holes in one ply of connections of diagonal bracing members in Sections F1, F2, F3 and F4 when the connection is designed as a slip-critical joint. The required strength for the limit state of bolt slip for the connection is specified in the applicable section. As reported in FEMA 355D (FEMA, 2000d), bolted joints with oversized holes in tested moment connections were found to behave as fully restrained connections for most practical applications. Bolted connections of diagonal bracing with oversized holes should behave similarly. Oversized holes in diagonal bracing connections with slip-critical bolts will provide additional tolerance for field connections, yet should remain as slip-resistant for most seismic events. If the bolts did slip in the oversized holes in an extreme situation, the connections should still behave similarly to fully restrained connections. Story drifts may also increase slightly if bolts slip, and the effect of bolt slip should be considered in drift calculations. In order to minimize the amount of slip, oversized holes for bolts are limited to one ply of the connection. For large diameter bolts, the amount of slippage can also be minimized by limiting the oversized bolt hole size to a maximum of 3⁄16 in. (5 mm) greater than the bolt diameter, rather than the maximum diameter permitted by the *Specification.* The available slip resistance of bolts in oversized holes is reflected in the reduced available strength for oversized holes per *Specification* Section J3.8. While there is no loss of pretension with bolts properly installed in oversized holes, the *Specification* for static applications reduces the available strength because of the larger slip that occurs at strength loads. The overall behavior of connections with oversized holes has been shown to be similar to those with standard holes (Kulak et al., 1987).

To prevent excessive deformations of bolted joints due to bearing on the connected material, the bearing and tearout strengths are limited to the option where deformation is a design consideration in *Specification* Section J3.10. The philosophical intent of this limitation in the *Specification* is to limit the bearing/tearout deformation to an approximate maximum of ¼ in. (6 mm). It should be recognized, however, that the actual bearing load 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 should effectively minimize damage in moderate seismic events. An exception is permitted for those bolted connections where the required force is determined by the capacity of a member or an adjacent one. For this condition the connection force is unlikely to be exceeded significantly. Therefore for this restriction, the bearing and tearout strengths may be increased to the values allowed in *Specification* Section J3.10 where deformation is not a design consideration. The consequences of the additional deformation should still be considered. For example, additional frame drift could occur in a moment frame with shallow beams and bolted

flange plate connections where additional beam rotation is caused by the increased bolt deformation.

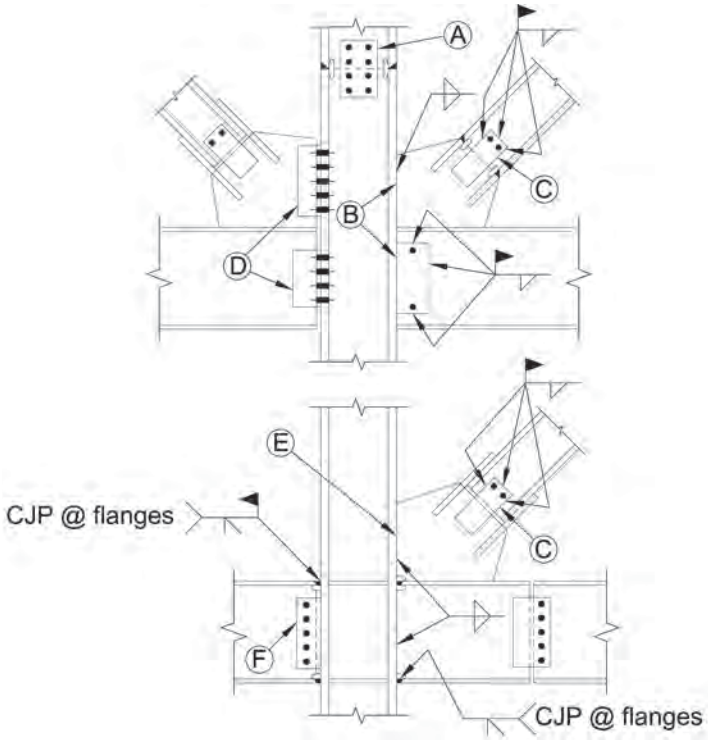
Connections or joints in which bolts in combination with welds resist a common force in a common shear plane are prohibited. Due to the potential for full load reversal and the likelihood of inelastic deformations in connecting plate elements, bolts may exceed their slip resistances under significant seismic loads. Welds that are in a common shear plane to these bolts will likely not deform sufficiently to allow the bolts to slip into bearing, particularly if subject to cyclic load reversal. Consequently, the welds will tend to resist the entire force and may fail if they are not designed as such. These provisions prohibit bolts from sharing a force with welds in a common shear plane in all situations. In addition to prohibiting sharing of loads on a common faying surface, sharing of a common force between different elements in other conditions is also prohibited. For example, bracing connections at beam-to-column joints are often configured such that the vertical component of the brace is resisted by a combination of both the beam web and the gusset connections to the columns (see Figure C-D2.1 for desirable details and Figure C-D2.2 for problematic connections). Since these two elements are in a common shear plane with limited deformation capability, if one element were welded and the other bolted, the welded joint would likely resist all the force. By making the connections of these elements to the column either both bolted or both welded when considering an individual shear plane, both elements would likely participate in resisting the force. Similarly, wide-flange bracing connections should not be designed such that bolted web connections share in resisting the axial loads with welded flanges (or vice versa).

Bolts in one element of a member may be designed to resist a force in one direction while other elements may be connected by welds to resist a force in a different direction or shear plane. For example, a beam-to-column moment connection may use welded flanges to transfer flexure and/or axial loads, while a bolted web connection transfers the beam shear. Similarly, column splices may transfer axial loads and/or flexure through flange welds with horizontal shear in the column web transferred through a bolted web connection. In both of these cases there should be adequate deformation capability between the flange and web connections to allow the bolts to resist loads in bearing independent of the welds.

The Provisions do not prohibit the use of erection bolts on a field-welded connection such as a shear tab in the web of a wide-flange beam moment connection. In this instance the bolts would resist the temporary erection loads, but the welds would need to be designed to resist the entire anticipated force in that element.

3. Welded Joints

The general requirements for design of welded joints are specified in *Specification* Chapter J. Additional design requirements for specific systems or connection types are specified elsewhere in the Provisions. The 2005 Provisions also invoked certain requirements for weld filler metal toughness and welding procedures. In these Provisions, the requirements are specified in Sections A3.4 and I2.3.



Ⓐ A bolted web connection may be designed to resist column shear while welded flanges resist axial and/or flexural forces.

Ⓑ Connection using both gusset and beam web welded to column allows both elements to participate in resisting the vertical component of the brace force. Note that erection bolts may be used to support beam temporarily.

Ⓒ Flanges and web are both welded to resist axial force in combination. Bolts are for erection only.

Ⓓ Both web of beam and gusset are bolted to column allowing sharing of vertical and horizontal forces.

Ⓔ A stub detail allows both gusset and beam web to be shop welded to column. Flanges of supported beam may be welded to transfer flexural and axial forces.

Ⓕ For beam moment connections, bolted webs can resist shear while welded flanges resist flexural and axial forces. (Moment connections must meet the requirements of Chapter E of the *Seismic Provisions*, as required.)

Fig. C-D2.1. Desirable details that avoid shared forces between welds and bolts.

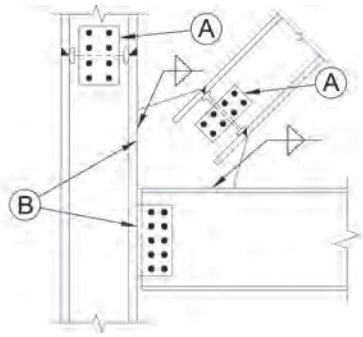
4. Continuity Plates and Stiffeners

The available lengths for welds of continuity plates and stiffeners to the web and flanges of rolled shapes are reduced by the detailing requirements of AWS D1.8/D1.8M clause 4.1 as specified in Section I2.4 of the Provisions. See Figures C-D2.3(a) and (b). These large corner clips are necessary to avoid welding into the *k*-area of wide-flange shapes. See Commentary Section A3.1 and AWS D1.8/D1.8M clause 4 commentary for discussion.

5. Column Splices

5a. Location of Splices

Column splices should be located away from the beam-to-column connection to reduce the effects of flexure. For typical buildings, the 4 ft (1.2 m) minimum distance requirement will control. When splices are located 4 to 5 ft (1.2 to 1.5 m) above the floor level, field erection and construction of the column splice will generally be



Ⓐ Brace or column members should not be designed with a combination of bolted web and welded flanges resisting axial forces.

Ⓑ Brace connections to columns with gussets welded to the column and the beam web bolted to the column will transfer forces differently from all-welded or all-bolted connections. The welded joint of the gusset to the column will tend to resist the entire vertical force at the column face (the vertical component of the brace force, plus the beam reaction). Also, the transfer of horizontal force through the bolted web to the column face will be precluded by the stiffer path through the welded joints of the gusset, so the gusset-to-beam joint will tend to resist the entire horizontal component of the brace force. Pass-through forces at beam-column connection will bypass the shear plate and go through the gusset. Equilibrium of the connection requires additional moments in both the beam and column, as well as higher forces in the welds of the gusset to the column and to the beam to transfer these forces.

Fig. C-D2.2. Problematic bolted/welded member connections.

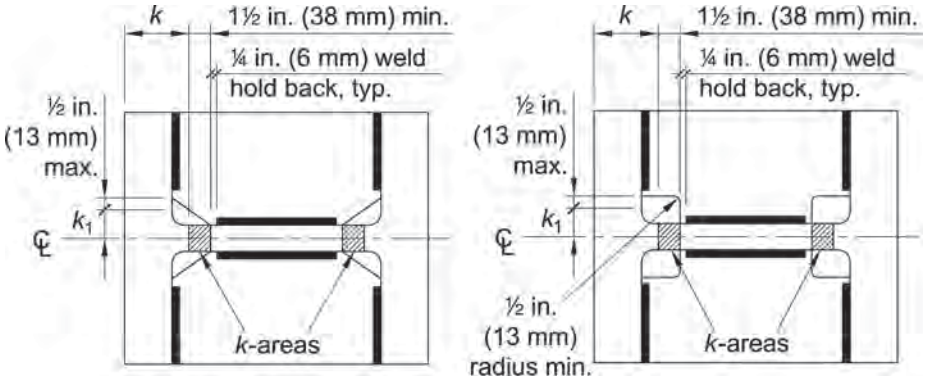
simplified due to improved accessibility and convenience. In general, it is recommended that the splice be within the middle third of the story height from a design perspective. For less typical buildings, where the floor-to-floor height is insufficient to accommodate this requirement, the splice should be placed as close as practicable to the midpoint of the clear distance between the finished floor and the bottom flange of the beam above. It is not intended that these column splice requirements be in conflict with applicable safety regulations, such as the OSHA *Safety Standards for Steel Erection* (OSHA, 2010) developed by the Steel Erection Negotiated Rulemaking Advisory Committee (SENRAC). This requirement is not intended to apply at columns that begin at a floor level, such as a transfer column, or columns that are interrupted at floor levels by cantilevered beams. However, the splice connection strength requirements of Section D2.5 still apply.

5b. Required Strength

Except for moment frames, the available strength of a column splice is required to equal or exceed both the required strength determined in Section D2.5b and the required strength for axial, flexural and shear effects at the splice location determined from load combinations stipulated by the applicable building code.

Partial-joint-penetration groove welded splices of thick column flanges exhibit virtually no ductility under tensile loading (Popov and Stephen, 1977; Bruneau et al., 1987). Consequently, column splices made with partial-joint-penetration groove welds require a 100% increase in required strength and must be made using weld metal with minimum Charpy V-notch (CVN) toughness properties.

The calculation of the minimum available strength in Section D2.5b(2)(b) includes the ratio R_y. This results in a minimum available strength that is not less than 50% of the expected yield strength of the column flanges. A complete-joint-penetration (CJP) groove weld may be considered as satisfying this requirement. However, when



(a) Straight corner clip *(b) Curved corner clip*

Fig. C-D2.3. Configuration of continuity plates.

applicable, tapered transitions are required in order to relieve stress concentrations where local yielding could occur at changes in column flange width or thickness per Section D2.5b(2)(c). Tensile stresses are to be calculated by adding the uniform axial stress with the elastic bending stress or stresses, using the elastic section modulus, *S*.

The possible occurrence of tensile loads in column splices utilizing partial-joint-penetration (PJP) groove welds during a maximum considered earthquake should be evaluated. When tensile loads are possible, it is suggested that some restraint be provided against relative lateral movement between the spliced column shafts because the strength of the PJP welds is potentially exhausted in resisting the tensile forces. 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.

Where CJP groove welds are not used, the connection is likely to consist of PJP groove welds. The unwelded portion of the PJP groove weld forms a discontinuity that acts like a notch that can induce stress concentrations. A PJP groove weld made from one side could produce an edge crack-like notch (Barsom and Rolfe, 1999). A PJP groove weld made from both sides would produce a buried crack-like notch. The strength of such internal crack-like notches may be computed by using fracture mechanics methodology. Depending on the specific characteristics of the particular design configuration, geometry and deformation, the analysis may warrant elastic-plastic or plastic finite element analysis of the joint. The accuracy of the computed strength will depend on the finite element model and mesh size used, the assumed strength and fracture toughness of the base metal, heat affected zone and weld metal, and on the residual stress magnitude and distribution in the joint.

5c. Required Shear Strength

Inelastic analyses (FEMA, 2000f) of moment frame buildings have shown the importance of the columns that are not part of the SFRS in helping to distribute the seismic shears between the floors. Even columns that have beam connections considered to be pinned connections may develop large bending moments and shears due to non-uniform drifts of adjacent levels. For this reason, it is recommended that splices of such columns be adequate to develop the shear forces corresponding to these large column moments in both orthogonal directions. Accordingly, columns that are part of

the SFRS must be connected for the greater of the forces resulting from these drifts, or the requirements specific to the applicable system in Chapters E, F, G or H.

FEMA 350 (FEMA, 2000a) recommends that: "Splices of columns that are not part of the seismic force-resisting system should be made in the center one-third of the column height, and should have sufficient shear capacity in both orthogonal directions to maintain the alignment of the column at the maximum shear force that the column is capable of producing." The corresponding commentary suggests that this shear should be calculated assuming plastic hinges at the ends of the columns in both orthogonal directions.

Further review (Krawinkler, 2001) of nonlinear analyses cited in FEMA 355C (FEMA, 2000d) showed that, in general, shears in such columns will be less than one-half of the shear calculated from $2M_{pc}/H$, where M_{pc} is the nominal plastic flexural strength of the column and H is the height of the story. For this reason, Section D2.5c requires that the calculated shear in the splices be $M_{pc}/(\alpha_s H)$.

5d. Structural Steel Splice Configurations

Bolted web connections are preferred by many engineers and contractors because they have advantages for erection, and when plates are placed on both sides of the web, whether they are bolted or welded, they are expected to maintain alignment of the column in the event of a flange splice fracture. A one-sided web plate may be used when it is designed as a back-up plate for a CJP web weld. This plate is also commonly used as a column erection aid. In most cases, partial-joint-penetration (PJP) groove welded webs are not recommended because fracture of a flange splice would likely lead to fracture of the web splice, considering the stress concentrations inherent in such welded joints. An exception allowing the use of PJP groove welds at the web splice in IMF, SMF and special truss moment frames (STMF) is given.

Weld backing for groove welds in column splices may remain. The justification for this is that unlike beam-to-column connections, splices of column flanges and webs using weld backing result in no transversely loaded notch.

6. Column Bases

Column bases must have adequate strength to permit the expected ductile behavior for which the system is designed in order for the anticipated performance to be achieved.

Column bases are required to be designed for the same forces as those required for the members and connections framing into them. If the connections of the system are required to be designed for the amplified seismic loads or loads based on member strengths, the connection to the column base must also be designed for those loads.

Column bases are considered to be column splices. The required strength of column bases includes the requirements prescribed in Section D2.5.

It is necessary to decompose the required tension strength of connections of diagonal brace members to determine the axial and shear forces imparted on the column base.

The requirement for removal of weld tabs and weld backing at column-to-base plate connections made with groove welds has been added to Section D2.6 as it is applicable to all SFRS systems in Chapters E, F, G and H. The use of weld backing for a CJP weld of a column to a base plate creates a transverse notch. Consequently, weld backing must be removed. For OMF, IMF and SMF systems, weld backing is allowed to remain at the CJP welds of the top flange of beam-to-column moment connections if a fillet weld is added per ANSI/AISC 358 Chapter 3 (AISC, 2016b). Similarly, an exception has been added for column bases to permit weld backing to remain at the inside flanges and at the webs of wide-flange shapes when a reinforcing fillet weld is added between the backing bar and the base plate.

6a. Required Axial Strength

The required axial (vertical) strength of the column base is computed from the column required strength in Sections D1.4a and D2.5b, in combination with the vertical component of the required connection strength of any braces present.

6b. Required Shear Strength

The required shear (horizontal) strength of the column base in the SFRS is computed from a mechanism in which the column forms plastic hinges at the top and bottom of the first story, in combination with the horizontal component of the required connection strength of any braces present. The component of shear in the column need not exceed the load effect corresponding to the overstrength seismic load. As noted in Commentary Section D2.5c, columns that are not part of the SFRS may be subject to significant shear loads from relative displacement between floors particularly if there are nonuniform drifts between floors. Similarly, bases of columns that are not part of the SFRS will be subject to high shear demand. A minimum shear requirement is present for all column bases including columns that are not part of the SFRS. The required shear force for column bases is less than that for column splices given that the base level of gravity columns is typically pinned. This allows the column to develop a lesser shear from building drift than a column with fixity at both ends. An exception to the shear force per Section D2.6b is allowed for single-story columns with simple connections at both ends as shear from story drift will not develop in columns where flexure cannot occur at either end.

An additional exception is added to reduce the minimum required shear force at the column base due to column flexure. The forces determined from a nonlinear analysis in accordance with Section C3 may be used to determine shear in the column.

Systems in Sections E1, F1, G1, H1 and H4 are expected to have limited inelastic behavior. Consequently, in these systems, shear forces in columns that are not part of the SFRS due to nonuniform drifts between the first and second story of a structure are expected to be minimal. Therefore, the minimum shear force is not required for these systems.

Alternatively, shear forces in the columns can be determined by an analysis that considers a drift of 0.025 times the story height at either the first level or the second level,

but not both concurrently. This can be performed using a simple model of a cantilever column with a single backspan as illustrated in Figure C-D2.4. The shear developed at the column base due to a deflection of 0.025h can be determined. Of note, the shear forces caused by a given drift about the column weak axis are typically less than the strong axis.

There are several possible mechanisms for shear forces to be transferred from the column base into the supporting concrete foundation. Surface friction between the base plate and supporting grout and concrete is probably the initial load path, especially if the anchor rods have been pretensioned. Unless the shear force is accompanied by enough tension to completely overcome the dead loads on the base plate, this mechanism will probably resist some or all of the shear force. However, many building codes prescribe that friction cannot be considered when resisting code prescribed earthquake loads, and another design calculation method must be utilized. The other potential mechanisms are anchor rod bearing against the base plates, shear keys bearing on grout in the grout pocket, or bearing of the column embedded in a slab or grade beam. See Figure C-D2.5.

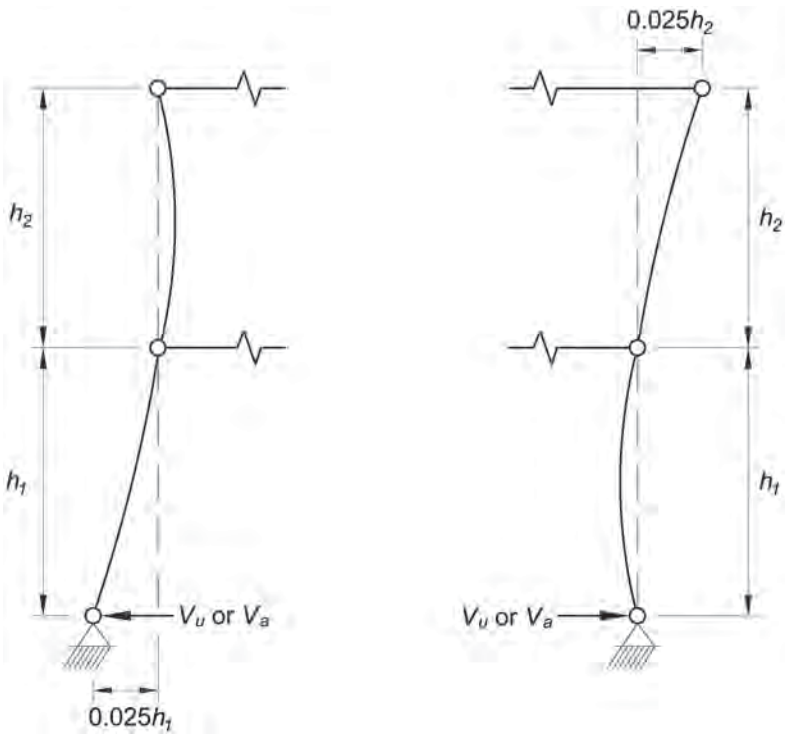


Fig. C-D2.4. Model to determine column drifts.

Anchor rod bearing is usually considered in design and is probably sufficient consideration for light shear loads. It represents the shear limit state if the base plate has overcome friction and has displaced relative to the anchor rods. The anchor rods are usually checked for combined shear and tension. Anchor rod bearing on the base plate may also be considered, but usually the base plate is so thick that this is not a problem. Note that oversize holes are typically used for anchor rods, and a weld washer may be required to transmit forces from the base plate to the anchor rods. Where shear is transferred through the anchor rods, anchor rods are subject to flexure.

A shear key should be considered for heavy shear loads, although welding and construction issues must be considered. If tension and/or overturning loads are present, anchor rods must also be provided to resist tension forces.

For foundations with large free edge distances, concrete blowout strength is controlled by concrete fracture; and the concrete capacity design (CCD) method prescribed in ACI 318 Chapter 17 provides a relatively accurate estimate of shear key concrete strength. For foundations with smaller edge distances, shear key concrete blowout strength is controlled by concrete tensile strength; and the 45° cone method prescribed in ACI 349 (ACI, 2006) and AISC Design Guide 1, *Base Plate and Anchor Rod Design* (AISC, 2010b) provides a reasonable estimate of shear key concrete strength. In recognition of limited physical testing of shear keys, it is recommended that the shear key concrete blowout strength be estimated by the lower of these two methods (Gomez et al., 2009).

Where columns are embedded, the bearing strength of the surrounding concrete can be utilized. Note that the concrete element must then be designed to resist this force and transfer it into other parts of the foundation or into the soil.

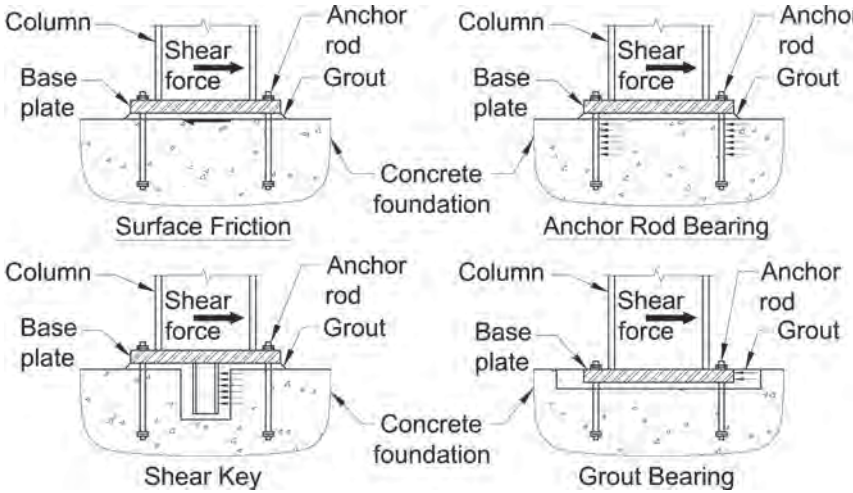


Fig. C-D2.5. Shear transfer mechanisms—column supported by foundation.

When the column base is embedded in the foundation, it can serve as a shear key to transfer shear forces. It is sometimes convenient to transfer shear forces to concrete grade beams through reinforcing steel welded to the column. Figure C-D2.6 shows two examples of shear transfer to a concrete grade beam. The reinforcing steel must be long enough to allow a splice with the grade beam reinforcing steel, allowing transfer of forces to additional foundations.

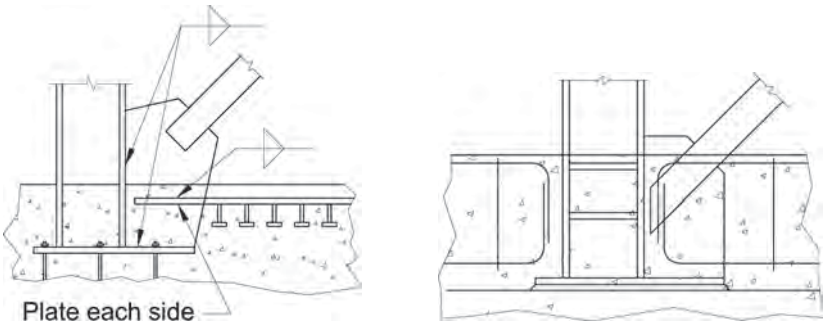
6c. Required Flexural Strength

Column bases for moment frames can be of several different types, as follows:

(1) A rigid base assembly may be provided which is strong enough to force yielding in the column. The designer should employ the same guidelines as given for the rigid fully restrained connections. Such connections may employ thick base plates, haunches, cover plates, or other strengthening as required to develop the column hinge. Where haunched-type connections are used, hinging occurs above the haunch, and appropriate consideration should be given to the stability of the column section at the hinge. See Figure C-D2.7 for examples of rigid base assemblies that can be designed to be capable of forcing column hinging. In some cases, yielding can occur in the concrete grade beams rather than in the column. In this case the concrete grade beams should be designed in conformance with ACI 318 Chapter 18.

(2) Large columns may be provided at the bottom level to limit the drift, and a "pinned base" may be utilized. The designer should ensure that the required shear capacity of the column, base plate and anchor rods can be maintained up to the maximum rotation that may occur. It should be recognized, however, that without taking special measures, column base connections will generally provide partial rotational fixity.

(3) According to the requirements of Section D2.6c(b)(2), the column base moment must be equal to or greater than the moment calculated using the overstrength seismic load. Since this moment is less than the flexural strength of the column,



(a) *(b)*

Fig. C-D2.6. Examples of shear transfer to a concrete grade beam.

there is a need to ensure that a ductile limit state will occur in either the connection or the foundation to avoid connection failure. A connection which provides "partial fixity" may be provided, such that the column behaves as a fixed column up to some moment, whereupon the column base yields prior to the column hinging. This can be achieved through flexural bending of the base plate similar to an end plate connection, bending of elements used as anchor chairs, ductile yielding of the foundation, uplift of the foundation or elongation of the anchor rods. For the latter, ACI 318 Chapter 17, provides guidance to ensure anchor rod elongation prior to concrete breakout.

(4) The column may continue below the assumed seismic base (e.g., into a basement, crawl space or grade beam) in such a way that column fixity is assured without the need for a rigid base plate connection. The designer should recognize that hinging will occur in the column, just above the seismic base or in the grade beam. If hinging is considered to occur in the grade beam, then the grade beam should be designed in conformance with ACI 318 Chapter 18. The horizontal shear to be resisted at the ends of the column below the seismic base should be calculated considering the expected strength, R_yF_y, of the framing. See Figure C-D2.8 for examples of a column base fixed within a grade beam.

Based on experimental observations, the ultimate strength of the column base will be reached when any one of the following yielding scenarios is activated (Gomez et al., 2010):

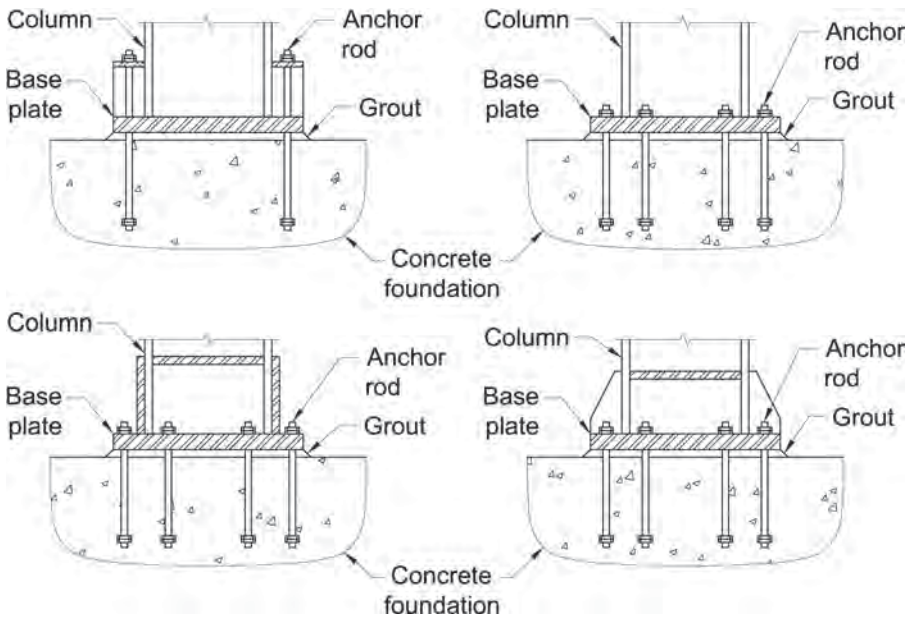


Fig. C-D2.7. Example of "rigid base" plate assembly for moment frames.

(1) Flexural yielding of both the tension side and compression side of the base plate

(2) Axial yielding of the anchor rods on the tension side

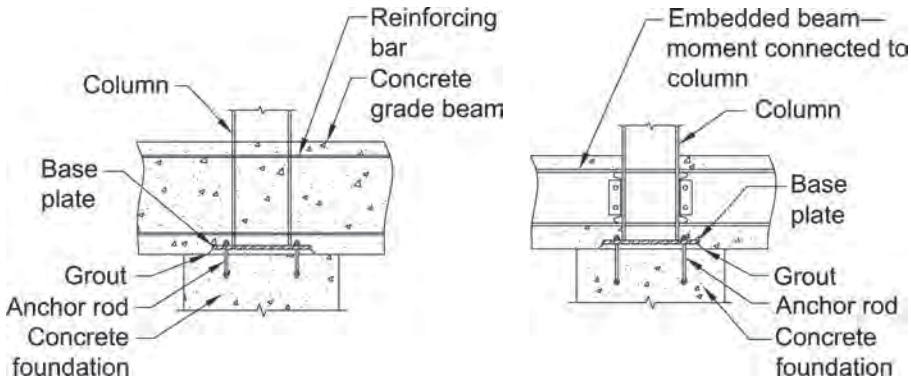
(3) Crushing of the concrete or grout

Historically, both triangular concrete stress blocks and rectangular concrete stress blocks have been used for the analysis of column base plates; the rectangular stress blocks give the best agreement with test results (Gomez et al., 2010).

7. Composite Connections

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.

Given the many alternative configurations of composite structures and connections, there are few standard details for connections in composite construction (Griffis, 1992; Goel, 1992a; Goel, 1993). However, tests are available for several connection details that are suitable for seismic design. References are given in this section and Commentary Chapters G and H. 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.



(a) *(b)*

Fig. C-D2.8. Examples of column base fixity in a grade beam.

General Requirements

The requirements for deformation capacity apply to both connections designed for gravity load only and connections that are part of the SFRS. 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, Figure C-D2.9 shows 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 SFRS. 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.

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 filled HSS 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 Section A3.2, calculations for the required strength of connections should, as a minimum, be made using the expected yield strength of the connected steel member or of the reinforcing bars in the connected concrete or composite member.

Nominal Strength of Connections

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

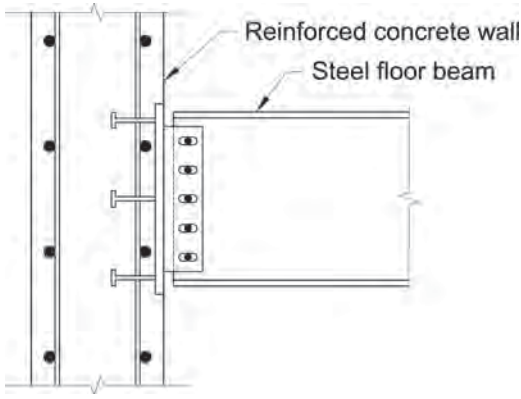


Fig. C-D2.9. Steel beam-to-reinforced concrete wall gravity load shear connection.

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 should 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% where large inelastic stress reversals are expected. This reduction is considered to be a conservative requirement that does not appear in ACI 318 but is applied herein due to the relative lack of experience with certain configurations of composite structures.

In many composite connections, steel components are encased by concrete that will inhibit or fully prevent local buckling. For seismic design where inelastic load 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 18). 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.

For fully encased connections between steel (or composite) beams and reinforced concrete (or composite) columns such as shown in Figure C-D2.10, 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-D2.11). This superposition of strengths for calculating the panel zone nominal shear strength is used in detailed design guidelines (Deierlein et al., 1989; ASCE, 1994; Parra-Montesinos and Wight, 2001) for composite connections that are supported by test data (Sheikh et al., 1989;

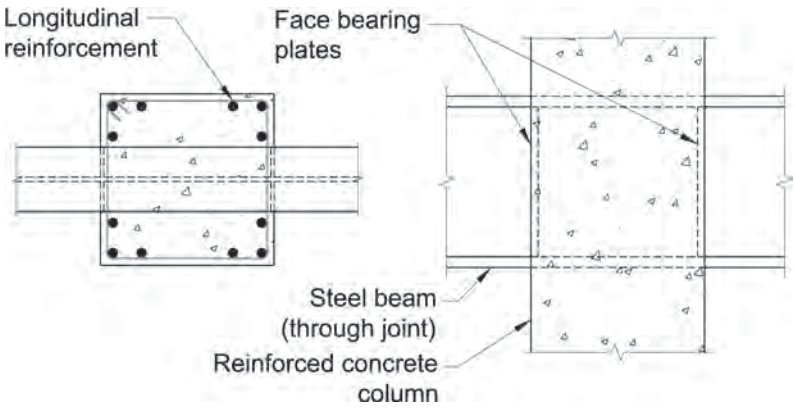


Fig. C-D2.10. Reinforced concrete column-to-steel beam moment connection.

Kanno and Deierlein, 1997; Nishiyama et al., 1990; Parra-Montesinos and Wight, 2001). Further information on the use and design of such connections is included in the commentary to Section G3.

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

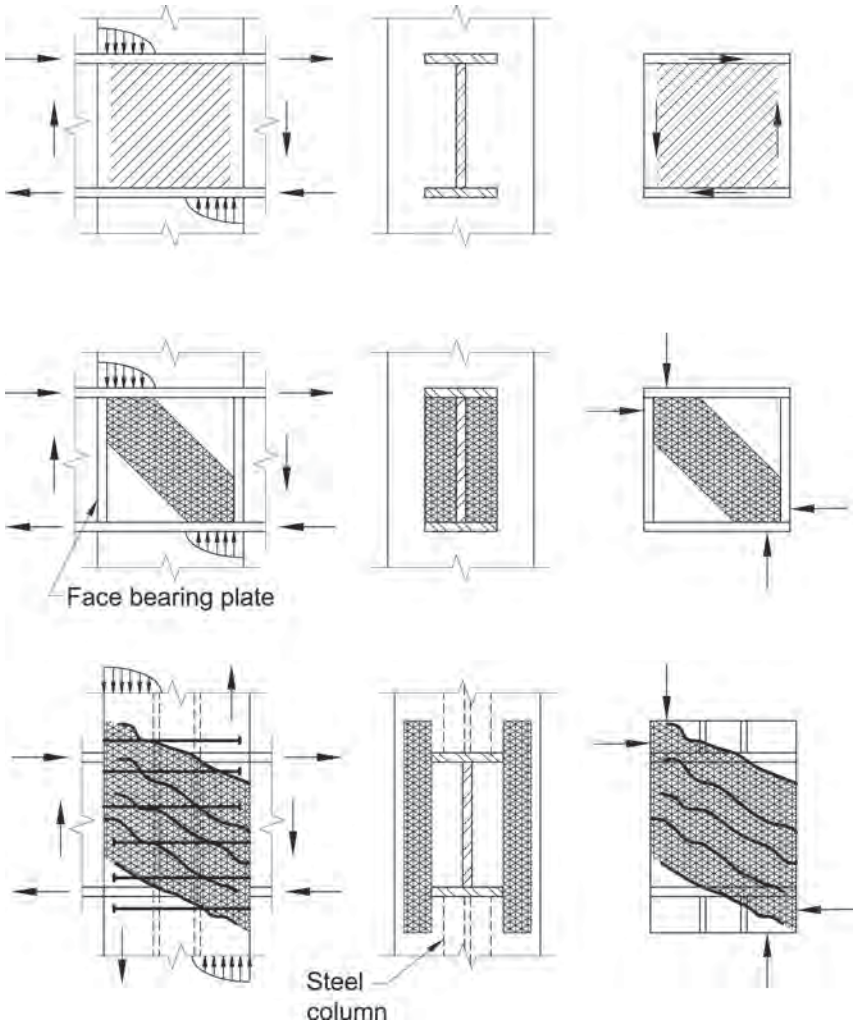


Fig. C-D2.11. Panel shear mechanisms in steel beam-to-reinforced concrete column connections (Deierlein et al., 1989).

tie models). Tie requirements for confinement usually are based on empirical models derived from test data and past performance of structures (ACI, 2002; Kitayama et al., 1987).

(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-D2.12), the strength of the concrete bearing against the column

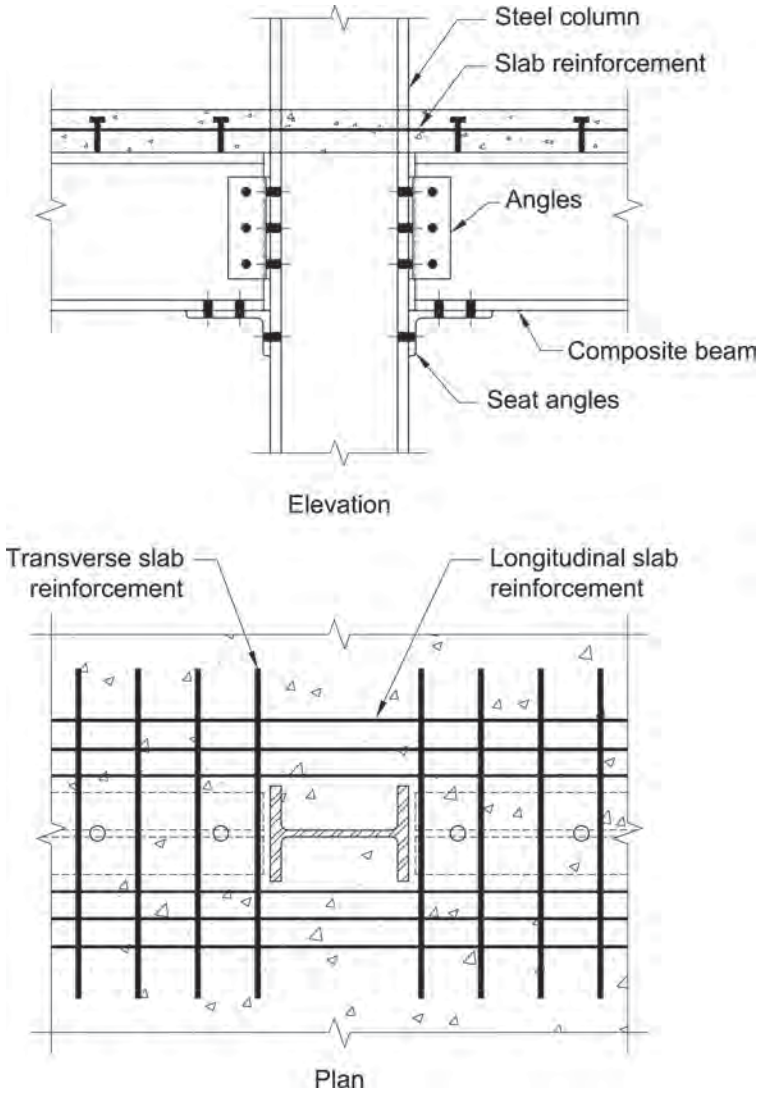


Fig. C-D2.12. Composite partially restrained connection.

flange should be checked (Green et al., 2004). 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. (300 mm) 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 Chapter 18 and in ACI 352R-02 (ACI, 2002), 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-D2.10 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 loads 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 352R-02.

At this time, there are not any provisions herein for determining panel zone shear strength; however, there is research that has been conducted on this subject. The following equations have been developed from research for calculating the panel zone shear strength of filled composite members:

$$V_n = V_c + V_{st} + V_{wn} \tag{C-D2-1}$$

where

$$V_c = \gamma A_{cp}\sqrt{f_c'}, \text{ kips (N)} \tag{C-D2-2}$$

γ = 28 for rectangular filled columns
= 24 for circular filled columns

A_{cp} = area of the concrete core engaged in the panel zone, in.^{2} (mm^2)

V_{st} = shear strength contribution of the filled composite column calculated using *Specification* Section I4.1, kips (N)

V_{wn} = shear strength contribution of the web of the steel beam in through-beam (uninterrupted) connections calculated using *Specification* Equation G2-1, kips (N)

The panel zone shear strength equations for filled composite columns are based on the research conducted by Elremaily (2000) and Koester (2000). The use of these equations has been illustrated by Fischer and Varma (2015).

8. Steel Anchors

Experiments of steel headed stud anchors subjected to shear or a combination of shear and tension consistently show that a reduction in strength occurs with cycling (McMullin and Astaneh-Asl, 1994; Civjan and Singh, 2003; Saari et al., 2004). Pallarés and Hajjar (2010a, 2010b) collected a wide range of test data of headed stud anchors subjected both to shear and combined shear and tension and documented that for composite members that are part of the SFRS in intermediate or special systems, a 25% reduction of the stud available strength given in the *Specification* is appropriate to allow for the effect of cyclic loads if the studs are expected to yield. Test data exists (Lee et al., 2005; Wang et al., 2011) to confirm the available strength of headed stud anchors up to 1 in. (25 mm) in diameter when subjected to monotonic loading. However, the available cyclic test data was almost exclusively for headed stud anchors with diameters up to ¾ in. (19 mm). As such, these provisions limit the diameter of headed stud anchors to ¾ in. (19 mm).

D3. DEFORMATION COMPATIBILITY OF NON-SFRS MEMBERS AND CONNECTIONS

Members that are not part of the SFRS and their connections may incur forces in addition to gravity loads as a result of story deflection of the SFRS during a seismic event. ASCE/SEI 7 Section 12.12.5 requires structural components that are not considered part of the SFRS to be able to resist the combined effects of gravity loads with any additional forces resulting from the design story drifts from seismic forces. The load effect due to the design story drift should be considered as an ultimate or factored load. Inelastic deformations of members and connections at these load levels are acceptable provided that instabilities do not result.

Nonuniform drifts of adjacent story levels may create significant bending moments in multistory columns. These bending moments will usually be greatest at story levels. Inelastic yielding of columns resulting from these bending moments can be accommodated when suitable lateral bracing is provided at story levels and when column shapes have adequate compactness (Newell and Uang, 2008). High shear forces at column splices resulting from these bending moments are addressed by the required shear strength requirements of Section D2.5c. The requirements for column splice location in Section D2.5a are intended to locate splices where bending moments are typically lower. Similarly, shear forces at column bases resulting from story drift are addressed by the requirements in Section D2.6b.

The P-Δ effect of the design story drift will also create additional axial forces in beams and girders due to column inclination in both single story and multistory columns. Connections of columns to beams or diaphragms should be designed to resist horizontal forces that result from the effects of the inclination of the columns. For

single-story columns, and multi-story columns where the inclination is constant, only the effect of the beam reactions at the story level requires a horizontal thrust to create equilibrium at that story level. However, for multistory columns where the column inclination varies between adjacent levels, the entire column axial force participates in creating a horizontal thrust for equilibrium. Figure C-D3.1 gives a comparison of the effect of column inclination on horizontal force at story level. Likewise, unequal drifts in multistory columns induce both flexure and shear in the column. Flexure will not be induced in columns with constant inclination and simple connections to beams.

Equivalent lateral force analysis methods have not been developed with an eye toward accurately estimating differences in story drift. Use of a modal response spectrum analysis to estimate differences in story drift is also problematic as this quantity is not tracked mode by mode in typical software. However, column shear can be tracked modally. Also, the horizontal thrust can be determined by detaching the column from the diaphragm and introducing a link element. Alternatively, thrust can be calculated from the change in column inclination, which can be estimated from the moment (and can be tracked mode by mode).

Properly designed simple connections are required at beam-to-column joints to avoid significant flexural forces. As per *Specification* Section J1, inelastic deformation of the connections is an acceptable means of achieving the required rotation. Standard shear connections per Part 10 of the AISC *Steel Construction Manual* (AISC, 2011)

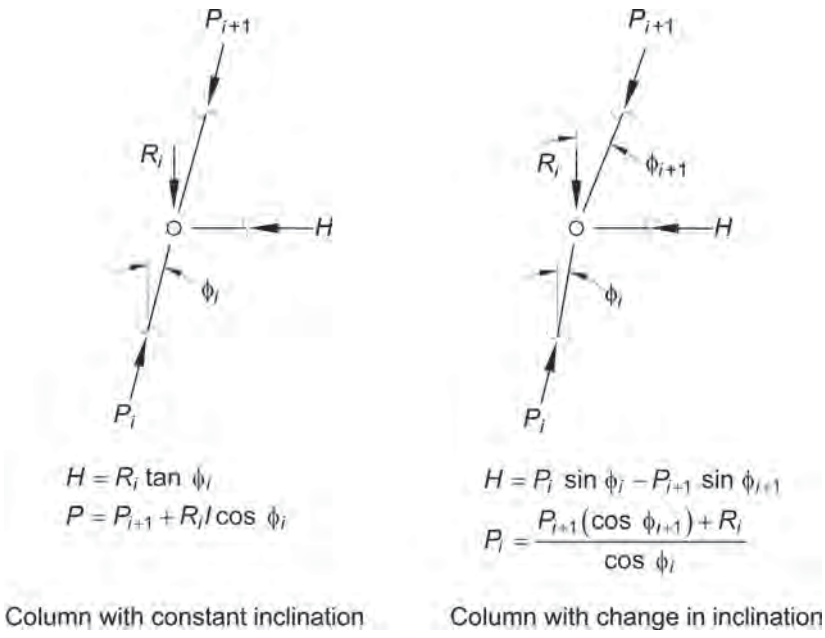


Fig. C-D3.1. Effect of column inclination on horizontal story force.

can be considered to allow adequate rotation at the joints without significant flexural moments. Double angles supporting gravity loads have been shown to attain maximum rotations of 0.05 to 0.09 rad and are suitable for combined gravity and axial forces as are WT connections which have demonstrated rotations of 0.05 to 0.07 rad (Astaneh-Asl, 2005a). Shear-plate connections (single plates), while inherently more rigid than double angles, have been shown to withstand gravity rotations ranging from 0.026 to 0.103 rad, and cyclic rotations of 0.09 rad (Astaneh-Asl, 2005b). Note that reducing the number of bolts in shear plates, and consequently the connection depth, increases the maximum possible rotation. Other connections at beam-to-column joints are acceptable if they are configured to provide adequate rotational ductility. Part 9 of the AISC *Steel Construction Manual* provides guidance on rotational ductility of end plate and WT connections that can be applied to many types of connections to ensure ductile behavior.

Beams and columns connected with moment connections that may experience inelastic rotation demands as a result of story drift should be detailed to maintain gravity support and provide any required resistance to seismic forces (such as axial collector forces) at the design story drift. Connections meeting the requirements of ordinary moment frames or conforming to the requirements of gusseted beam-to-column connections for SCBF, EBF or BRBF (for example, Section F2.6b) provide such resistance and deformation capacity.

D4. H-PILES

The provisions on seismic design of H-piles are based on the data collected on the actual behavior of H-piles during recent earthquakes, including the 1994 Northridge earthquake (Astaneh-Asl et al., 1994) and the results of full-scale cyclic pile tests (Astaneh-Asl and Ravat, 1997). In the test program, five full size H-Piles with reinforced concrete pile caps were subjected to realistic cyclic vertical and horizontal displacements expected in a major earthquake. Three specimens were vertical piles and two specimens were batter piles. The tests established that during cyclic loading for all three vertical pile specimens a very ductile and stable plastic hinge formed in the steel pile just below the reinforced concrete pile cap. When very large inelastic cycles were applied, local buckling of flanges within the plastic hinge area occurred. Eventually, low cycle fatigue fracture of flanges or overall buckling of the pile occurred. However, before the piles experienced fracture through locally buckled areas, vertical piles tolerated from 40 to 65 large inelastic cyclic vertical and horizontal displacements with rotation of the plastic hinge exceeding 0.06 rad for more than 20 cycles.

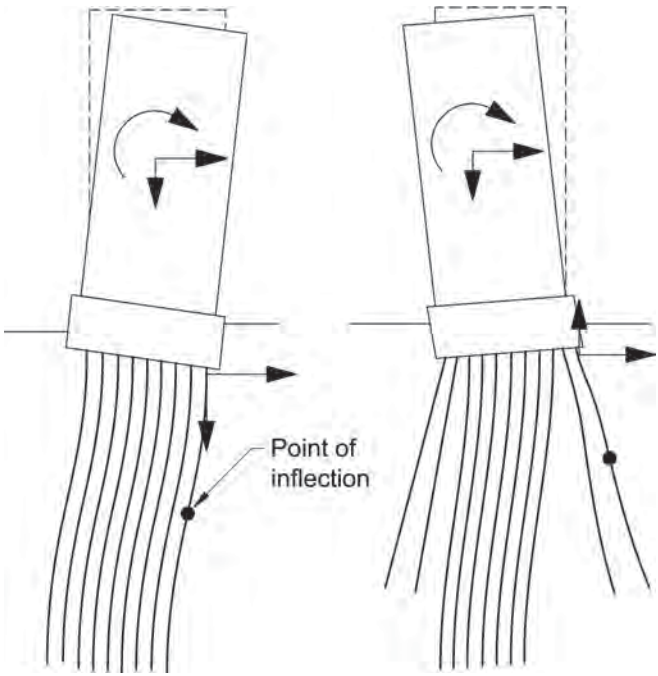
1. Design Requirements

Prior to an earthquake, piles, particularly vertical piles, are primarily subjected to gravity axial load. During an earthquake, piles are subjected to horizontal and vertical displacements as shown in Figure C-D4.1. The horizontal and vertical displacements of piles generate axial load (compression and possibly uplift tension), bending moment, and shear in the pile.

The lateral deflections can be particularly high in locations where upper soil layers are soft or where soils may be prone to liquefaction. A case study of performance of H-piles during the 1994 Northridge earthquake (Astaneh-Asl et al., 1994) investigated H-piles where the upper layers were either in soft soil or partially exposed. During tests of H-piles realistic cyclic horizontal and vertical displacements were applied to the pile specimens. Figure C-D4.2 shows test results in terms of axial load and bending moment for one of the specimens. Based on the performance of test specimens, it was concluded that H-piles should be designed following the provisions of the *Specification* regarding members subjected to combined loads. H-piles in soft soil conditions are expected to undergo significant lateral displacements and develop high bending forces and possibly plastic hinges near the pile cap. Consequently H-piles in soft soil conditions necessitate a compactness requirement that ensures ductile inelastic behavior. The flange compactness requirement is less stringent than that of wide-flange beams and is based on the width-to-thickness of the H-piles tested in the Astaneh study given their good performance.

2. Battered H-Piles

The vertical pile specimens demonstrated very large cyclic ductility as well as considerable energy dissipation capacity. A case study of performance of H-piles during



(a) Vertical Piles Only *(b) Vertical and Battered Piles*

Fig. C-D4.1. Deformations of piles and forces acting on an individual pile.

the 1994 Northridge earthquake (Astaneh-Asl et al., 1994) indicated excellent performance for pile groups with vertical piles only. However, the battered pile specimens did not show as much ductility as the vertical piles. The battered piles tolerated from 7 to 17 large inelastic cycles before failure. Based on relatively limited information on actual seismic behavior of battered piles, it is possible that during a major earthquake, battered piles in a pile group fail and are no longer able to support the gravity load after the earthquake. Because of this possibility, the use of battered piles to carry gravity loads is discouraged. Unless, through realistic cyclic tests, it is shown that battered piles will be capable of carrying their share of the gravity loads after a major earthquake, the vertical piles in seismic design categories D, E and F should be designed to support the gravity load alone, without participation of the batter piles.

3. Tension

Due to overturning moment, piles can be subjected to tension. Piles subjected to tension should have sufficient mechanical attachments within their embedded area to transfer the tension force in the pile to the pile cap or foundation.

4. Protected Zone

Since it is anticipated that during a major earthquake, a plastic hinge is expected to form in H-piles in soft soil conditions just under the pile cap or foundation, the use of mechanical attachment and welds over a length of pile below the pile cap equal to the depth of the pile cross section is prohibited. This region is therefore designated as a protected zone.

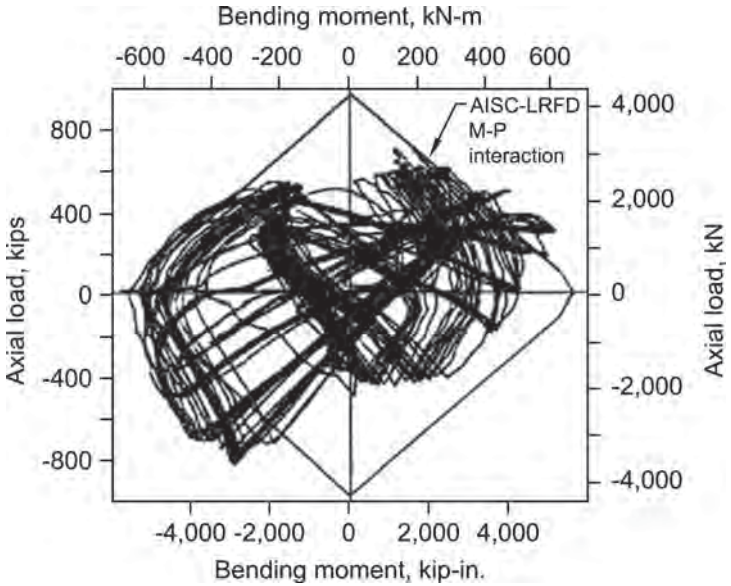


Fig. C-D4.2. Axial load-moment interaction for H-pile test.

CHAPTER E

MOMENT-FRAME SYSTEMS

E1. ORDINARY MOMENT FRAMES (OMF)

2. Basis of Design

Compared to intermediate moment frame (IMF) and special moment frame (SMF) systems, OMF are expected to provide only minimal levels of inelastic deformation capacity. To compensate for this lower level of ductility, OMF are designed to provide larger lateral strength than IMF and SMF, and thus, are designed using a lower R factor. Systems such as OMF with high strength and low ductility have seen much less research and testing than higher ductility systems. Consequently, the design requirements for OMF are based much more on judgment than on research. Due to the limited ductility of OMF and due to the limited understanding of the seismic performance of these systems, ASCE/SEI 7 (ASCE, 2016) places significant height and other limitations on their use.

Although the design basis for OMF is to provide for minimal inelastic deformation capacity, there is no quantitative definition of the required capacity as there is for IMF and SMF systems. Despite the lack of a quantitative inelastic deformation requirement, the overall intent of OMF design is to avoid nonductile behavior in its response to lateral load.

To provide for minimal inelastic deformation capacity, i.e., to avoid nonductile behavior, the general intent of the OMF design provisions is that connection failure should not be the first significant inelastic event in the response of the frame to earthquake loading. Connection failure, in general, is one of the less ductile failure modes exhibited by structural steel frames. Thus, as lateral load is increased on an OMF, the intent is that the limit of elastic response be controlled by limit states other than connection failure, such as reaching the limiting flexural or shear strength of a beam or a column, reaching the limiting shear strength of the panel zone, etc. For higher ductility systems such as IMF and SMF, inelasticity is intended to occur in specific frame elements. For example, in SMF, inelasticity is intended to occur primarily in the form of flexural yielding of the beams. This is not the case with OMF, where the initial inelastic response is permitted to occur in any frame element.

Thus, the basic design requirement for an OMF is to provide a frame with strong connections. That is, connections should be strong enough so that significant inelastic action in response to earthquake loading occurs in frame elements rather than connections. This applies to all connections in the frame, including beam-to-column connections, column splices, and column base connections. Requirements for OMF column splices and column base connections are covered in Section D2. Requirements for beam-to-column connections are covered in Section E1.6.

There is an exception where initial inelastic response of an OMF is permitted to occur in beam-to-column connections. This is for OMF provided with partially restrained (PR) moment connections. Requirements for PR moment connections are covered in Section E1.6c.

Design and detailing requirements for OMF are considerably less restrictive than for IMF and SMF. The OMF provisions are intended to cover a wide range of moment frame systems that are difficult or impossible to qualify as IMF or SMF. This includes, for example, metal building systems, knee-braced frames, moment frames where the beams and/or columns are trusses (but not STMF), moment frames where the beams and/or columns are HSS, etc.

OMF Knee-Brace Systems. Knee-brace systems use an axial brace from the beam to the column to form a moment connection. Resistance to lateral loads is by flexure of the beam and column. These systems can be designed as an OMF. The knee-brace system can be considered as analogous to a moment frame with haunch-type connections. The knee brace carries axial force only, while the beam-to-column connection carries both axial force and shear. A design approach for knee-braced systems is to design the beam-to-column connection, the braces, and the brace end connections for the forces required to develop $1.1R_yM_p/\alpha_s$ of the beam or column, or the maximum moment that can be delivered by the system, whichever is less. M_p is the plastic flexural strength of the beam or column at the point of intersection with the knee brace. The column and beams should be braced out of plane, either directly or indirectly at the knee brace locations, consistent with the requirements of *Specification* Appendix 6.

OMF Truss Systems. In some moment frame configurations, trusses are used for the beam elements in place of rolled shapes. These systems can be designed as a special truss moment frame (STMF) following the requirements of Section E4. Alternatively, these systems can also be designed as an OMF where OMF are allowed by ASCE/SEI 7 (ASCE, 2016). As an OMF, a design approach would be to design the truss and the truss-to-column connections for the maximum force that can be transferred by the system, consistent with the requirements of Section E1.6b(b). The maximum force that can be delivered to the truss and truss-to-column connections can be based on the flexural capacity of the columns, taken as $1.1R_yM_p/\alpha_s$ of the column, combined with vertical loads from the prescribed load combinations. Thus, the intent is to design a weak column system where inelasticity is expected to occur in the columns. The column should be braced out of plane, either directly or indirectly at the location of the top and bottom chord connection of the truss, consistent with the requirements of Appendix 6 of the *Specification*.

4. System Requirements

Unlike SMF, there is no beam-column moment ratio (i.e., strong column-weak beam) requirement for OMF. Consequently, OMF systems can be designed so that inelasticity will occur in the columns.

5. Members

There are no special restrictions or requirements on member width-to-thickness ratios or member stability bracing, beyond meeting the requirements of the *Specification.* Although not required, the judicious application of width-to-thickness limits and member stability bracing requirements as specified for moderately ductile members in Section D1 would be expected to improve the performance of OMF.

6. Connections

For all moment frame systems designed according to these Provisions, including SMF, IMF and OMF, the beam-to-column connections are viewed as critical elements affecting the seismic performance of the frame. For SMF and IMF systems, connection design must be based on qualification testing per Section K2 or a connection prequalified per Section K1 shall be used. For OMF, connections need not be prequalified nor qualified by testing. Rather, design of beam-to-column connections can be based on strength calculations or on prescriptive requirements. Design and detailing requirements for beam-to-column connections in OMF are provided in this section.

6b. FR Moment Connections

Three options are provided in this section for design of FR moment connections. Designs satisfying any one of these three options are considered acceptable. Note that for all options, the required shear strength of the panel zone may be calculated from the basic code prescribed loads, with the available shear strength calculated in accordance with *Specification* Section J10.6. This may result in a design where initial yielding of the frame occurs in the panel zones. This is viewed as acceptable behavior due to the high ductility exhibited by panel zones.

(a) The first option permits the connection to be designed for the flexural strength of the beam, taken as $1.1R_yM_p/\alpha_s$. The 1.1 factor in the equation accounts for limited strain hardening in the beam and other possible sources of overstrength. The required shear strength of the connection is calculated using the code-prescribed load combinations, where the shear force to the connection associated with the capacity-limited horizontal shear due to earthquake loading is calculated per Equation E1-1. The available strength of the connection is computed using the *Specification*. Note that satisfying these strength requirements may require reinforcing the connection using, for example, cover plates or haunches attached to the beam. The required flexural strength of the connection specified in this section, i.e., $1.1R_yM_p/\alpha_s$ of the beam, should also be used when checking if continuity plates are needed per Sections J10.1 through J10.3 in the *Specification*.

(b) The second option permits design of the connection for the maximum moment and shear that can be transferred to the connection by the system. Factors that can limit the forces transferred to the connection include column yielding, panel zone yielding, foundation uplift, or the overstrength seismic load. In the case of column yielding, the forces at the connection can be calculated assuming the column reaches a limiting moment of $1.1R_yM_p/\alpha_s$ of the column. In the case

of panel zone yielding, the forces at the connection can be computed assuming the shear force in the panel zone is $1.1R_y/\alpha_s$ times the nominal shear strength given by Equations J10-11 and J10-12 in the *Specification*. For frames with web-tapered members, as typically used in metal building systems, the flexural strength of the beam (rafter) or column will typically be first reached at some distance away from the connection. For such a case, the connection can be designed for the forces that will be generated when the flexural strength of a member is first reached anywhere along the length of the member. The flexural strength of the member may be controlled by local buckling or lateral-torsional buckling, and can be estimated using equations for the nominal flexural strength, M_n, in *Specification* Chapter F. However, lower-bound methods of determining M_n are not appropriate, and engineers should endeavor to establish a reasonable upper bound by considering items that contribute to the stability of the beam, even those that are typically ignored for design of the beam because they are difficult to quantify, not always present, etc. In particular, it is not appropriate to use $C_b = 1.0$. A realistic value of C_b should be used. Additionally, the stabilizing effects of the deck restraining the beam both laterally and torsionally should be included in determining this upper bound. M_p may always be used as the upper bound.

(c) The third option for beam-to-column connections is a prescriptive option for cases where a wide flange beam is connected to the flange of a wide flange column. The prescriptive connection specified in the section is similar to the welded unreinforced flange-bolted web (WUF-B) connection described in FEMA 350 (FEMA, 2000a). Some of the key features of this connection include the treatment of the complete-joint-penetration (CJP) beam flange-to-column welds as demand critical, treatment of backing bars and weld tabs using the same requirements as for SMF connections, and the use of special weld access hole geometry and quality requirements. Testing has shown that connections satisfying these requirements can develop moderate levels of ductility in the beam or panel zone prior to connection failure (Han et al., 2007).

Option (c) also permits the use of any connection in OMF that is permitted in IMF or SMF systems. Thus, any of the prequalified IMF or SMF connections in ANSI/AISC 358 can be used in OMF. However, when using ANSI/AISC 358 connections in an OMF, items specified in ANSI/AISC 358 that are not otherwise required in OMF systems are not required. For example, the WUF-W connection prequalified in ANSI/AISC 358 can be used for an OMF connection. However, items specified in ANSI/AISC 358 that would not be required when a WUF-W connection is used in an OMF include beam and column width-to-thickness limitations for IMF and SMF, beam stability bracing requirements for IMF or SMF, beam-column moment ratio requirements for SMF, column panel zone shear strength requirements for IMF or SMF, or requirements for a protected zone. None of these items are required for OMF, and therefore are not required when the WUF-W connection is used in an OMF. Similar comments apply to all connections prequalified in ANSI/AISC 358.

6c. PR Moment Connections

Section E1.6c gives strength requirements for PR connections, but does not provide complete prescriptive design requirements. PR connections are permitted to have a flexural strength that is substantially less than the connected beam or column. This will normally result in inelastic action occurring in the connection rather than in the beam or column during an earthquake. As described in Section E1.6c(b), the designer must consider the stiffness, strength and deformation capacity of PR moment connections on the seismic performance of the frame. This may require nonlinear time history analysis with accurate modeling of the PR connections to demonstrate satisfactory performance.

For design information on PR connections, refer to Leon (1990); Leon (1994); Leon and Ammerman (1990); Leon and Forcier (1992); Bjorhovde et al. (1990); Hsieh and Deierlein (1991); Leon et al. (1996); and FEMA 355D (FEMA, 2000e).

E2. INTERMEDIATE MOMENT FRAMES (IMF)

2. Basis of Design

IMF are intended to provide limited levels of inelastic rotation capacity and are based on tested designs. Due to the lesser rotational capacity of IMF as compared to SMF, ASCE/SEI 7 requires use of a lower seismic response modification coefficient, R, than that for SMF and places significant height and other limitations on its use.

While the design for SMF is intended to limit the majority of the inelastic deformation to the beams, the inelastic drift capability of IMF is permitted to be derived from inelastic deformations of beams, columns and/or panel zones.

The IMF connection is based on a tested design with a qualifying story drift angle of 0.02 rad based on the loading protocol specified in Section K2. It is assumed that this limited connection rotation will be achieved by use of larger frame members than would be required in an SMF, because of the lower R and/or higher C_d/R values used in design.

Commentary Section E3 offers additional discussion relevant to IMF.

4. System Requirements

4a. Stability Bracing of Beams

See Commentary Section D1.2a on stability bracing of moderately ductile members and Commentary Section E3.4b for additional commentary.

5. Members

5a. Basic Requirements

This section refers to Section D1, which provides requirements for connection of webs to flanges as for built-up members and requirements for width-to-thickness

ratios for the flanges and webs of the members. Because the rotational demands on IMF beams and columns are expected to be lower than for SMF, the width-to-thickness limitations for IMF are less severe than for SMF. See Commentary Section E3.5a for further discussion.

5b. Beam Flanges

The requirements in this section are identical to those in Section E3.5b. See Commentary Section E3.5b for further discussion.

5c. Protected Zones

For commentary on protected zones, see Commentary Section D1.3.

6. Connections

6a. Demand Critical Welds

The requirements in this section are identical to those in Section E3.6a. See Commentary Section E3.6a for further discussion.

6b. Beam-to-Column Connection Requirements

The minimum story drift angle required for qualification of IMF connections is 0.02 rad while that for SMF connections is 0.04 rad. This level of story drift angle has been established for this type of frame based on engineering judgment applied to available tests and analytical studies, primarily those included in FEMA (2000d) and FEMA (2000f).

ANSI/AISC 358 (AISC, 2016b) describes nine different connections that have been prequalified for use in both IMF and SMF systems. The prequalified connections include the reduced beam section (RBS), the bolted unstiffened extended end plate (BUEEP), the bolted stiffened extended end plate (BSEEP), the bolted flange plate (BFP), the welded unreinforced flange-welded web (WUF-W), the Kaiser bolted bracket (KBB), the ConXtech ConXL, the SidePlate, and the Simpson Strong-Tie Strong Frame Moment Connection. In a few cases, the limitations on use of the connections are less strict for IMF than for SMF, but generally, the connections are the same.

6c. Conformance Demonstration

The requirements for conformance demonstration for IMF connections are the same as for SMF connections, except that the required story drift angle is smaller. Refer to Commentary Section E3.6c for further discussion.

6d. Required Shear Strength

The requirements for shear strength of the connection are the same for IMF as for SMF. See Commentary Section E3.6d for further discussion.

6e. Panel Zone

The panel zone for IMF is required to be designed according to *Specification* Section J10.6, with no further requirements in the Provisions. As noted in Commentary Section E2.2, panel zone yielding is permitted as part of the inelastic action contributing to the drift capacity of the IMF and the requirements of the *Specification* are considered adequate to achieve the expected performance.

6f. Continuity Plates

The requirements in this section are identical to those in Section E3.6f. See Commentary Section E3.6f for further discussion.

6g. Column Splices

The requirements in this section are identical to those in Section E3.6g. See Commentary Section E3.6g for further discussion.

E3. SPECIAL MOMENT FRAMES (SMF)

2. Basis of Design

SMF are generally expected to experience significant inelastic deformations during large seismic events. It is expected that most of the inelastic deformation will take place as rotation in beam "hinges," with limited inelastic deformation in the panel zone of the column. The beam-to-column connections for these frames are required to be qualified based on tests that demonstrate that the connection can sustain a story drift angle of at least 0.04 rad based on the loading protocol specified in Section K2. Other provisions are intended to limit or prevent excessive panel zone distortion, column hinging, and local buckling that may lead to inadequate frame performance in spite of good connection performance.

Beam-to-column connections in SMF systems are permitted to be fully restrained or partially restrained. ANSI/AISC 358 prequalification considers the performance of the connection and frame. In order to permit the use of partially restrained connections in SMF systems, system performance equivalent to SMF systems meeting all of the requirements of Section E3 is required to be demonstrated by analysis. The analysis should evaluate the effect of connection restraint in the elastic and inelastic range on system performance and should demonstrate equivalent performance to systems employing qualifying fully restrained connections. This may be accomplished using FEMA P-795 (FEMA, 2011), which considers the similarity of the hysteretic response of a "substitute" connection—in this case, a partially restrained connection—and a "benchmark" connection, which could be any prequalified connection. ANSI/AISC 358 has prequalified one partially restrained connection, the Simpson Strong-Tie Strong Moment Frame connection, for use in SMF. Alternatively, equivalent performance may also be substantiated through analysis conforming to ASCE/SEI 7 Sections 12.2.1.1 and 12.2.1.2.

Since SMF and IMF connection configurations and design procedures are based on the results of qualifying tests, the parameters of connections in the prototype structure must be consistent with the tested configurations. Chapter K and ANSI/AISC 358 provide further detail on this requirement.

3. Analysis

The strong-column/weak-beam (SC/WB) concept, as defined for planar frames in Section E3.4a, is a capacity-design approach intended to provide for frame columns strong enough to distribute frame (primarily beam) yielding over multiple stories, rather than concentrating inelastic action in column hinging at a single story (weak story). The requirement outlined in Section E3.4a is an approximate and simplified method, in use for several generations of these Provisions, that is deemed to provide the desired performance for planar frames. It should be recognized that other analyses could be used to demonstrate that the desired performance could be achieved, for example, an analysis considering the performance on a story, rather than individual column, basis.

Recognizing that in systems such as SMF, significant yielding of the structure is expected under the design displacements, and recognizing that design displacements can occur in any direction relative to the orthogonal axes of the structure, the possible effects of yielding of the structure in both directions simultaneously must be considered in columns that participate in SMFs in more than one direction.

ASCE/SEI 7 requires that analyses include the effects of 100% of the design motions in one direction in conjunction with 30% of those in the orthogonal direction. As even the 30% design motion is likely capable of yielding the structure, and considering that the 100% motion may occur in any direction relative to the structure's axes, it is clear that simultaneous yielding of orthogonal systems is likely and should be considered in the design.

The extent to which simultaneous yielding of orthogonal systems is of concern is a matter of configuration and design. Consider the following examples:

(1) An efficiently-designed symmetrical two-way moment frame with shared columns that conforms with Section E3.4a in each direction independently is subjected to design motions at or near 45° to the structure's axes. For this case, a story mechanism could occur, due to hinging of all columns in a story, because of the weakening effects of the unaccounted for biaxial effects on the columns. In this case, the designer should consider application of the strong-column/weak-beam analysis in both orthogonal directions simultaneously.

(2) A system consisting of multi-bay planar moment frames in each orthogonal direction, intersecting at corner columns only. For this case, demonstration of the desired performance could be shown by an analysis that considers the relative strength of the columns to the beams on a story, rather than individual, basis. Additionally, the bending strength of the corner columns would need to be considered as the column strength was reduced by the orthogonal yielding effects. As a more simple and conservative alternate, the strength of the corner

columns could be ignored in calculating the story strength. In either case the corner column would need to be checked for strength considering the effects of axial force and bi-axial bending as required by the *Specification*.

Other analysis methods could also be considered to confirm the desired performance as described in the following sections.

Column-Tree Method. One approach to get a reasonable estimate of required strength of columns for ensuring essentially elastic behavior is to consider the equilibrium of the entire column (sometimes called the "column tree") in its expected extreme deformed condition (Goel and Chao, 2008). For this purpose, the column from bottom to top can be treated as a vertical cantilever with all expected forces acting on it to satisfy equilibrium. The forces will include moments and shears from the yielded beams framing into the column at all floor levels along with gravity loads supported by the column. By assuming an appropriate vertical distribution of lateral inertia forces, and expected moment at the base of the column, the magnitude of the lateral forces can be calculated by using the moment equilibrium equation for the "column tree." For columns that are part of frames in a single plane (flexural loading about one axis), it is appropriate to take the moment applied by the yielded beams as the probable moment, M_{pr}. Lower values may be justifiable recognizing it is unlikely that beams at levels within a multi-story building will reach this value (Goel and Chao, 2008).

For columns that are part of intersecting frames, the preceding calculation needs to be carried out in two orthogonal planes along the two principal axes of the column. It is highly unlikely that maximum expected moments and corresponding shears in the beams would occur simultaneously along the entire height of the column. Using the nominal plastic moment capacity at beam ends and corresponding shear appears to be reasonable to represent the intersecting frames behaving inelastically simultaneously. The bending moments, shears, and axial force in each story in both orthogonal planes can be calculated by statics. The design of the bi-axially loaded beam-columns can be carried out by using the *Specification.* In applying P-Δ effects, a drift resulting in the yielding of beams in each intersecting frame should be considered. 1% drift is often a reasonable approximation to achieve this.

Interaction Method. In most building configurations, an SFRS can be idealized as a system of planar moment frames, with internal forces being resisted in the plane of the frames. Equation E3-2 utilized in the verification of SC/WB is an approximation of the full plastic P-M interaction for uniaxial bending. This equation represents the moment capacity of a column reduced due to the effect of an axial force. In the case where a column forms part of two or more intersecting moment frames, it may be necessary to check the SC/WB criteria about both axes of the column. In this situation, Equation E3-2 does not explicitly address bi-axial bending and account for the reduction in moment capacity of the column about the axis under consideration due to the moment demand in the column about the orthogonal axis. Equation E3-2 can be modified to include the effect of bi-axial bending by similarly assuming a linear P-M_x-M_y interaction, commonly referred to as a "yield surface"; see Equation C-E3-1.

In Equation C-E3-1, the subscripts x and y represent in-plane and out-of-plane section properties of the column, respectively, and do not designate the strong and weak axes of the column as done elsewhere. In design when it is necessary to verify SC/WB about both column axes, the orthogonal section properties of the column will change accordingly.

$$M_{pcx}^{*} = Z_x \left[F_{yc} - \left(\frac{\alpha_s P_c}{A_g} + \frac{M_y}{Z_y} \right) \right] \qquad \text{(C-E3-1)}$$

where

A_g = gross area of column, in.^{2} (mm^2)
F_{yc} = specified minimum yield stress of column, ksi (MPa)
M_{pcx}^{*} = plastic flexural strength of the column in the plane of the frame under consideration, kip-in. (N-mm)
M_y = required out-of-plane flexural strength of the column taking into account all potential yielding beams that may contribute to the applied moment, kip-in. (N-mm)
P_c = P_{uc} or P_{ac} as defined in Section E3.4a, kips (N). In this case, P_c should be determined in Chapter D by addressing the axial force inputted from all frames connected to the column.
Z_x = plastic section modulus of the column in the plane of the frame under consideration, in.^{3} (mm^3)
Z_y = plastic section modulus of the column out of plane of the frame under consideration, in.^{3} (mm^3)
α_s = 1.0 for LFRD and 1.5 for ASD

In the simplest case, M_y can be estimated as $\sum M_{pb}^{*}/2$, where M_{pb}^{*} is the plastic flexural strength of a beam in the out-of-plane frame at the joint under consideration, kip-in. (N-mm).

The linear yield surface given by Equation C-E3-1 is illustrated in Figure C-E3.1. Only one quadrant is shown for brevity.

Equation E3-2 and its bi-axial extension (Equation C-E3-1) may provide a conservative estimate of the plastic flexural capacity for specific sections. For example, based on classical plastic design theory, the strong-axis plastic flexural strength (taken as the x-axis) of a wide-flange section can be taken as Equation C-E3-2.

$$M_{pcx} = 1.18 Z_x \left(F_y - \frac{P_c}{A_g} \right) \le Z_x F_y \qquad \text{(C-E3-2)}$$

Similarly, the weak-axis plastic flexural strength (taken as the y-axis) of a wide-flange section can be taken as Equation C-E3-3.

$$M_{pcy} = 1.19 Z_y \left[F_y - \frac{\left(P_c / A_g \right)^2}{F_y} \right] \le Z_y F_y \qquad \text{(C-E3-3)}$$

Equations C-E3-2 and C-E3-3 are for cases when no moment about the axis orthogonal to the axis under consideration is present. Several yield surfaces that account for bi-axial bending are discussed in the SSRC *Guide to Stability Design Criteria* (Ziemian, 2010). For example, a linear equation applicable for a wide-flange section was proposed by Pillai (1974):

$$\frac{P_c}{P_y} + 0.85\frac{M_x}{M_{px}} + 0.6\frac{M_y}{M_{py}} \le 1 \qquad \text{(C-E3-4)}$$

Equation C-E3-4 can be reconfigured to provide the plastic flexural strength about the strong-axis (taken as the *x*-axis) while including the flexural demand about the weak-axis (taken as the *y*-axis). The strong-axis plastic moment strength for a wide-flange shape can be taken as Equation C-E3-5.

$$M_{pcx} = 1.18Z_x\left[F_y - \left(\frac{P_c}{A_g} + 0.6\frac{M_y}{Z_y}\right)\right] \le Z_x F_y \qquad \text{(C-E3-5)}$$

Similarly, the weak-axis plastic flexural strength for a wide-flange shape can be taken as Equation C-E3-6. This equation is provided for illustration only since beam-to-column connections are not yet prequalified for framing into the weak-axis of a wide-flange column.

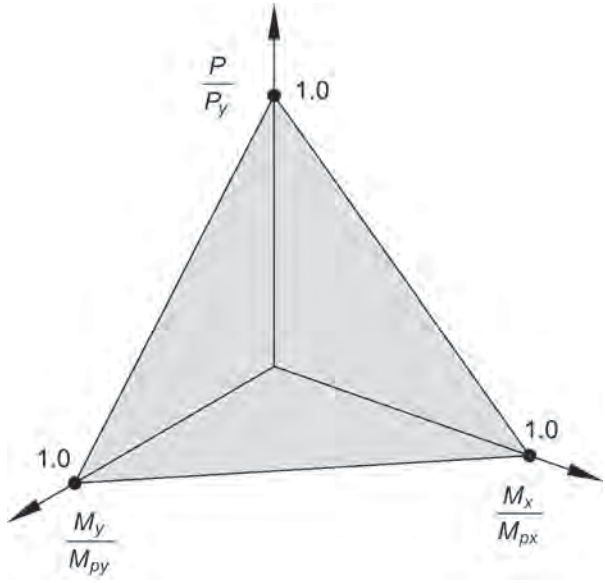


Fig. C-E3.1. Linear yield surface for bi-axial bending.

$$M_{pcy} = 1.67 Z_y \left[F_y - \left(\frac{P_c}{A_g} + 0.85 \frac{M_x}{Z_x} \right) \right] \leq Z_y F_y \qquad \text{(C-E3-6)}$$

More accurate estimates for a wide-flange section may be obtained using a nonlinear interaction equation, Equation C-E3-7, based on Tebedge and Chen (1974):

$$\left(\frac{M_x}{M_{px}} \right)^{\alpha} + \left(\frac{M_y}{M_{py}} \right)^{\alpha} \leq 1 \qquad \text{(C-E3-7)}$$

where M_{px} and M_{py} can be determined from Equations C-E3-2 and C-E3-3, respectively. The exponent α for a wide-flange section is given by Equation C-E3-8:

$$\alpha = \begin{cases} 1.0 & \text{for } b_f/d < 0.5 \\ 1.6 - \dfrac{P/P_y}{2\ln\left(P/P_y\right)} & \text{for } 0.5 \leq b_f/d < 1.0 \end{cases} \qquad \text{(C-E3-8)}$$

It is common in the case of a cruciform-type column built up from orthogonal flanged sections that each axis is treated independently of the other, neglecting the perpendicular section properties, and Equation E3-2 is applicable for each axis. This decoupled approach is appropriate where the only attachment between the orthogonal sections occurs at the neutral axis of each section such that flexural actions in one section do not significantly influence the state of stress in the orthogonal section. Where built-up sections are substantially attached at locations other than the neutral axis of each section, for example at the toes of flanges in cruciform-type columns built up from orthogonal flanged sections, the bi-axial bending of the built-up column shape should be considered.

4. System Requirements

4a. Moment Ratio

As noted, the strong-column weak-beam (SC/WB) concept is often mistakenly assumed to be formulated to prevent any column flange yielding in a frame, and that if such yielding occurs, the column will fail. Tests have shown that yielding of columns in moment frame subassemblages does not necessarily reduce the lateral strength at the expected seismic displacement levels.

The SC/WB concept is 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 meeting SC/WB requirements 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 in the system. Weak column frames, particularly those with weak or soft stories, are likely to exhibit an undesirable response at those stories with the highest column demand-to-capacity ratios.

Compliance with the SC/WB concept and Equation E3-1 gives no assurance that individual columns will not yield, even when all connection locations in the frame comply. Nonlinear response history analyses have shown that, as the frame deforms inelastically, points of inflection shift and the distribution of moments varies from the idealized condition. Nonetheless, yielding of the beams rather than the columns will predominate and the desired inelastic performance will, in general, be achieved in frames with members sized to meet the requirement in Equation E3-1.

Early formulations of the SC/WB relationship idealized the beam/column intersection as a point at the intersection of the member centerlines. Post-Northridge beam-to-column moment connections are generally configured to shift the plastic hinge location into the beam away from the column face and a more general formulation was needed. ANSI/AISC 358 provides procedures to calculate the location of plastic hinges for the connections included therein. For other configurations, the locations can be determined from the applicable qualifying tests. Recognition of expected beam strength (see Commentary Section A3.2) is also incorporated into Equation E3-1.

Three exceptions to Equation E3-1 are given. In the first exception, columns with low axial loads used in one-story buildings or in the top story of a multi-story building need not meet Equation E3-1 because concerns for inelastic soft or weak stories are not significant in such cases. Additionally, exception is made for columns with low axial loads, under certain conditions, in order to provide design flexibility where the requirement in Equation E3-1 would be impractical, such as at large transfer girders. Finally, Section E3.4a provides an exception for columns in levels that are significantly stronger than in the level above because column yielding at the stronger level would be unlikely.

In applying Equation E3-1, recognition should be given to the location of column splices above the girder-to-column connection being checked. When the column splice is located at 4.0 ft (1.2 m) or more above the top of the girder, it has been customary to base the calculation on the column size that occurs at the joint. If the column splice occurs closer to the top of the beam, or when the column above the splice is much smaller than that at the joint, consideration should be given to whether the column at the joint is capable of providing the strength assumed using the customary approach.

4b. Stability Bracing of Beams

See Commentary Section D1.2b on stability bracing of highly ductile members.

In addition to bracing along the beam length, the provisions of Section D1.2c call for the placement of lateral bracing near the location of expected plastic hinges. Such guidance dates to the original development of plastic design procedures in the early 1960s. In moment frame structures, many connection details attempt to move the plastic hinge a short distance away from the beam-to-column connection. Testing carried out as part of the SAC program (FEMA, 2000a) indicated that the bracing provided by typical composite floor slabs is adequate to avoid excessive strength deterioration up to the required story drift angle of 0.04 rad. Therefore, the FEMA

recommendations do not require the placement of supplemental lateral bracing at plastic hinge locations adjacent to column connections for beams with composite floor construction. These provisions allow the placement of lateral braces to be consistent with the tested connections that are used to justify the design. If a reduced beam section connection detail is used, the reduced flange width may be considered in calculation of the bracing force. The requirements of Section E3.5c should be considered when placing bracing connections.

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

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

1. Braced Connections

Beam-to-column connections are usually braced 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 bracing of the column flanges is required only at the level of the top flanges of the beams. If it cannot be shown that the column remains elastic, lateral bracing is required at both the top and bottom beam flanges because of the potential for flexural yielding, and consequent lateral-torsional buckling of the column.

The required strength for lateral bracing at the beam-to-column connection is 2% of the nominal strength of the beam flange. In addition, the element(s) providing lateral bracing should provide adequate stiffness to inhibit lateral movement of the column flanges (Bansal, 1971). In some cases, a bracing member will be required for such lateral bracing (direct stability bracing). Alternatively, calculations may show that adequate lateral bracing can be provided by the column web and continuity plates or by the flanges of perpendicular beams (indirect stability bracing).

The 1997 Provisions (AISC, 1997b) required column lateral bracing when the ratio in Equation E3-1 was less than 1.25. The intent of this provision was to require bracing to prevent lateral-torsional buckling for cases where it cannot be assured that the column will not hinge. Studies utilizing inelastic analyses (Gupta and Krawinkler, 1999; Bondy, 1996) have shown that, in severe earthquakes, plastic hinging can occur in the columns even when this ratio is significantly larger than 1.25. (See also discussion under Commentary Section E3.4a). The revised limit of 2.0 was selected as a reasonable cutoff because column plastic hinging for values greater than 2.0 only occurs in the case of extremely large story drifts. The intent of the revisions to this section is to encourage appropriate bracing of column flanges rather than to force the use of much heavier columns, although other benefits may accrue by use of heavier columns, including possible elimination of continuity and doubler plates that may offset the additional material cost.

2. Unbraced Connections

Unbraced connections occur in special cases, such as in two-story frames, at mechanical floors, or in atriums and similar architectural spaces (multi-tier conditions). 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 limit the likelihood of column buckling.

5. Members

5a. Basic Requirements

Reliable inelastic deformation capacity for highly ductile members requires that width-to-thickness ratios of projecting elements be limited to cross sections resistant to local buckling well into the inelastic range. Although the width-to-thickness ratios for compact elements in *Specification* Table B4.1 are sufficient to prevent local buckling before the onset of yielding, available test data suggest that these limits are not adequate for the required inelastic rotations in SMF. The limits given in Table D1.1 are deemed adequate for the large ductility demands to which these members may be subjected (Sawyer, 1961; Lay, 1965; Kemp, 1986; Bansal, 1971).

5b. Beam Flanges

Abrupt changes in beam flange area in locations of high strain, as occurs in plastic hinge regions of SMF, can lead to fracture due to stress concentrations. For connections such as the reduced beam section (RBS), the gradual flange area reduction, when properly configured and fabricated, can be beneficial to the beam and connection performance. Such conditions are permitted when properly substantiated by testing.

5c. Protected Zones

For commentary on protected zones see Commentary Section D1.3.

6. Connections

6a. Demand Critical Welds

For general commentary on demand critical welds see Commentary Section A3.4.

The requirement to use demand critical welds for complete-joint-penetration (CJP) groove welded joints in beam-to-column connections of SMF was first included in the 2002 Provisions (AISC, 2002). The requirement for notch-tough welds with Charpy V-notch toughness of 20 ft-lb at −20°F (−28.9°C) was introduced in the 1999 Supplement No. 1 to the 1997 Provisions. FEMA 350 and 353 (FEMA, 2000b) recommended that supplemental requirements beyond the basic toughness noted above should be applied to CJP welds in these connections. Welds for which these special requirements apply are referred to as demand critical welds.

The requirement to use demand critical welds for groove welded column splices and for welds at column base plates was new to these Provisions in 2010. The change was

made because, although it is likely that, in general, strain demands at near-mid-height column splice locations are less severe than those at beam-to-column joints, Shen et al. (2010) showed that bending at these locations can be large enough to cause flange yielding. This fact, coupled with the severe consequence of failure, was the justification for this requirement.

For the case of column-to-base plate connections at which plastic hinging is expected in the column, the condition is very similar to the condition at a beam-to-column connection. Where columns extend into a basement or are otherwise restrained in such a way that the column hinging will occur at a level significantly above the base plate, this requirement is judged to be overly conservative, and an exception is provided.

6b. Beam-to-Column Connections

Section E3.6b gives the performance and design requirements for the connections, with a special provision that outlines requirements for the use of partially-restrained connections when justified by analysis; see Commentary Section E3.2. Section E3.6c provides the requirements for verifying that the selected connections will meet the performance requirements. These requirements have been derived from the research of the SAC Joint Venture as summarized in FEMA 350.

FEMA 350 recommends two criteria for the qualifying drift angle (QDA) for SMF. The "strength degradation" drift angle, as defined in FEMA 350, means the angle where "either failure of the connection occurs, or the strength of the connection degrades to less than the nominal plastic capacity, whichever is less." The "ultimate" drift angle capacity is defined as the angle "at which connection damage is so severe that continued ability to remain stable under gravity loading is uncertain." Testing to this level can be hazardous to laboratory equipment and staff, which is part of the reason that it is seldom done. The strength degradation QDA is set at 0.04 rad and the ultimate QDA is set at 0.06 rad. These values formed the basis for extensive probabilistic evaluations of the performance capability of various structural systems (FEMA, 2000f) demonstrating with high statistical confidence that frames with these types of connections can meet the intended performance goals. For the sake of simplicity, and because many connections have not been tested to the ultimate QDA, the Provisions adopt the single criterion of the strength degradation QDA. In addition, the ultimate QDA is more appropriately used for the design of high performance structures.

Although connection qualification primarily focuses on the level of plastic rotation achieved, the tendency for connections to experience strength degradation with increased deformation is also of concern. Strength degradation can increase rotation demands from P-Δ effects and the likelihood of frame instability. In the absence of additional information, it is recommended that this degradation should not reduce flexural strength, measured at a drift angle of 0.04 rad, to less than 80% of the nominal flexural strength, M_p, calculated using the specified minimum yield stress, F_y. Figure C-E3.2 illustrates this behavior. Note that 0.03 rad plastic rotation is equivalent to 0.04 rad drift angle for frames with an elastic drift of 0.01 rad.

ANSI/AISC 358 describes ten different connections that have been prequalified for use in both IMF and SMF systems. The prequalified connections include the reduced beam section (RBS), the bolted unstiffened extended end plate (BUEEP), the bolted stiffened extended end plate (BSEEP), the bolted flange plate (BFP), the welded unreinforced flange-welded web (WUF-W), the Kaiser bolted bracket (KBB), the ConXtech ConXL connection, the SidePlate connection, the Simpson Strong-Tie Strong Frame connection, and the double-tee connection. In a few cases, the limitations on use of the connections are less strict for IMF than for SMF, but generally, the connections are the same.

The following explains the use of the ANSI/AISC 358 Simpson Strong-Tie Strong Frame moment connection, but is appropriate to other partially restrained (PR), or partial-strength connections that may be added in the future, or may be proposed for specific projects.

The limitation of $0.8M_p$ was originally adopted based on judgment, before the tools to perform sophisticated nonlinear dynamic analysis were readily available and before the building code, or ASCE/SEI 7, had adopted quantitative performance criteria. The general intent of the building code was that under "severe" but undefined earthquakes, buildings should not collapse. Typical hysteretic curves for highly ductile elements like moment frames (assuming they actually behaved in a ductile manner) were perceived to have the general shape shown in Figure C-E3.3, in which the hysteretic backbone would include an "elastic range," a "plastic-strain hardening range," and a "plastic strength-degrading range."

Based on linear dynamic analysis (typically of idealized single degree of freedom systems), researchers had determined that response of structures with lateral systems that have been pushed into the "strength-degrading" range can be unbounded and lead

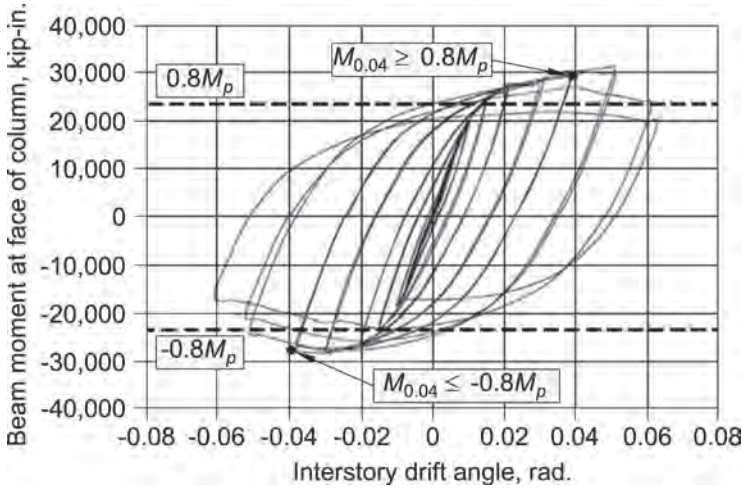


Fig. C-E3.2. Acceptable strength degradation, per Section E3.6b.

to collapse or very large lateral displacement. The 80% limitation was implemented to provide some assurance that structures would not be pushed "too far" into the strength degrading range, though the definition of "too far" was not quantified.

Recent tests by Simpson Strong Tie have demonstrated yield links in PR connections that were able to develop only about 50% of the beam's theoretical M_p. Regardless, at 0.04 rad, the connections clearly were not yet reaching the strength-degrading regime of response that the 80% M_p was intended to guard against. Because every connection technology may have quite different hysteretic characteristics, it is not practicable to be able to directly broaden the 80% M_p definition to address all technologies that may be appropriate, and which may come forward. Consequently, the requirement has been broadened to allow for the demonstration of equivalent performance through substantiating analysis as an alternate to meeting the 80% M_p threshold.

In the time since the 80% M_p was adopted as a standard, the industry's ability to perform nonlinear analysis and also the building code's definition of acceptable performance has evolved substantially. The ASCE/SEI 7-10 standard (ASCE, 2010) defined acceptable performance in terms of a limiting permissible conditional probability of collapse, given the occurrence of MCE shaking. These definitions are carried forward in ASCE/SEI 7-16 (ASCE, 2016). Two documents developed by the Applied Technology Council (ATC) on behalf of the Federal Emergency Management Agency (FEMA) define procedures for assuring that structures meet these performance (noncollapse) criteria; one of these is FEMA P-695 (FEMA, 2009b). FEMA P-695 uses an extension of the probabilistic framework developed by the FEMA/SAC project to qualify post-Northridge moment frames, by computing the probability of collapse of frames of given configuration and hysteretic characteristics. The companion document, FEMA P-795 (FEMA, 2011), provides a means of judging

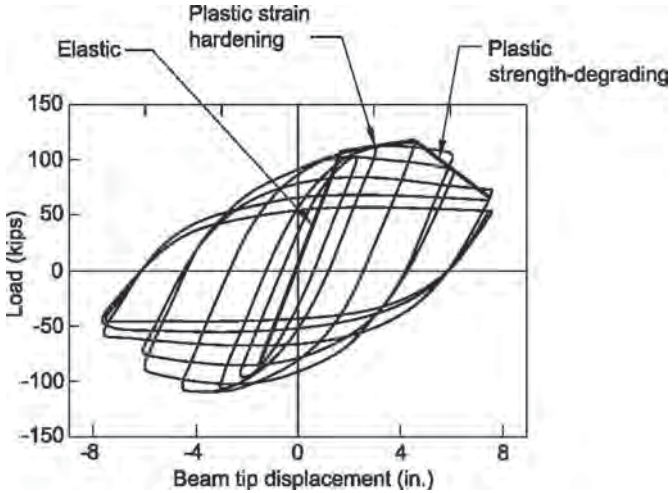


Fig. C-E3.3. Load and tip displacement data for a PR-connected cantilevered beam.

whether the substitution of a component, that is, a connection, into a system that has been demonstrated by FEMA P-695 to have adequate collapse resistance, will affect that resistance. ASCE/SEI 7 Section 12.2.1 adopts both methodologies as a means of demonstrating acceptable performance either for new structural systems (Section 12.2.1.1), or for substitute components in existing systems (Section 12.2.1.2).

With regard to rotation data for the Simpson Strong-Tie Strong Frame connection, ANSI/AISC 358 contains a detailed design procedure for this connection that includes determination of the rotational stiffness of the connection. The procedure requires that this flexibility be considered in determining frame adequacy (drift). The CPRP performed review of available hysteretic test data that substantiates that the connection stiffness representation contained in the design procedure is a reasonable approximation of that obtained in testing.

6c. Conformance Demonstration

This section provides requirements for demonstrating conformance with the requirements of Section E3.6b. This provision specifically permits the use of prequalified connections meeting the requirements of ANSI/AISC 358 to facilitate and standardize connection design. Connections approved by other prequalification panels may be acceptable but are subject to the approval of the authority having jurisdiction. Use of connections qualified by prior tests or project-specific tests may also be used, although the engineer of record is responsible for substantiating the connection performance. Published testing, such as that conducted as part of the SAC project and reported in FEMA 350 and 355 or project-specific testing, may be used to satisfy this provision.

6d. Required Shear Strength

The seismic component of the required shear strength of the beam-to-column connection is defined as the shear that results from formation of the probable maximum moment at the plastic hinge locations, which can be determined as in Equation E3-6. This shear must be combined with other shear forces, such as gravity forces, using the load combinations of the applicable building code.

6e. Panel Zone

1. Required Shear Strength

Cyclic testing has demonstrated that significant ductility can be obtained through shear yielding in column panel zones through many cycles of inelastic loading (Popov et al., 1996; Slutter, 1981; Becker, 1971; Fielding and Huang, 1971; Krawinkler, 1978; Lee et al. 2005a and 2005b; Shin and Engelhardt, 2013). Consequently, it is not generally necessary to provide a panel zone that will remain elastic under earthquake loading. Initial significant yielding of the panel zone will occur when the shear force in the panel zone reaches the values given by Equations J10-9 and J10-10 of the *Specification.* However, both experimental and computational studies have shown that panel zones can resist substantially higher shear forces due to strain hardening and due to contributions of the

column flanges in resisting panel zone shear. Consequently, the ultimate shear strength of the panel zone can be more than 50% greater than the shear at first yield, particularly for columns with thick flanges. This additional shear strength is considered in Equations J10-11 and J10-12 of the *Specification*, which provide an estimate of the shear resistance of the panel zone after moderate levels of cyclic inelastic deformation has occurred. These equations are based on the work by Krawinkler (1978).

Despite the ductility demonstrated by properly proportioned panel zones in previous studies, there are concerns that excessive inelastic panel zone distortions can adversely affect the performance of beam-to-column connections (Krawinkler, 1978; Englekirk, 1999; El-Tawil et al., 1999). Krawinkler noted that large shear distortions of the panel zone result in the formation of localized "kinks" at the corners of the panel zone that can lead to the occurrence of fracture in the vicinity of the beam flange-to-column flange groove welds. Many tests, however, have shown that cyclic joint rotations well in excess of ± 0.04 rad can be achieved prior to the occurrence of fracture (Krawinkler, 1978; Engelhardt et al., 2000; Lee et al., 2005b; Shin and Engelhardt, 2013). In addition to concerns about how shear distortion may affect joint performance, there are also uncertainties on how overall frame performance will be affected when panel zones are substantially weaker than the beams.

To summarize, past research has shown that shear yielding in the panel zone can provide high levels of stable cyclic inelastic deformation, and can be an excellent source of ductility in steel moment-resisting frames. However, past research has also suggested that caution is needed in panel zone design, as excessive panel zone yielding may have adverse effects on joint performance and on overall frame performance. Based on these observations, these Provisions have taken the approach that beam flexural yielding should still be the primary source of inelastic deformation in SMF, but that limited yielding of panel zones is acceptable.

The required strength of the panel zone is defined as the shear force in the panel zone when the fully yielded and strain hardened flexural strength of the attached beams has been developed. For connections where the beam flanges are welded directly to column flanges, such as the prequalified RBS and WUF-W connections in ANSI/AISC 358, the LRFD required shear strength of the panel zone, R_u, can be estimated as follows:

$$R_u = \frac{\sum M_f}{d - t_f} - V_{col} \qquad \text{(C-E3-9)}$$

In this equation, $\sum M_f$ is the sum of the beam moments at the face of the column when the beams have achieved their probable maximum moment at the plastic hinge, M_{pr}, as defined in ANSI/AISC 358. V_{col} is the shear force in the portion of the column outside of the panel zone that occurs when the beams have achieved their probable maximum moment.

The available strength of the panel zone is computed using *Specification* Section J10.6. As specified in these Provisions, the available strength is computed using $\phi = 1.00$ (LRFD) or $\Omega = 1.50$ (ASD), reflecting the view that limited panel zone yielding is acceptable.

Specification Section J10.6 provides two options for computing panel zone available strength. According to the *Specification*, the first option, given by Equations J10-9 and J10-10, is used "when the effect of inelastic panel-zone deformation on frame stability is not accounted for in the analysis." The second option, given by Equations J10-11 and J10-12, is used "when the effect of inelastic panel-zone deformation on frame stability is accounted for in the analysis." As discussed, Equations J10-9 and J10-10 correspond to first significant yield of the panel zone, and using these equations will result in panel zones that remain nominally elastic during earthquake loading. In contrast, Equations J10-11 and J10-12 provide an estimate of the shear resistance after the panel zone has developed moderate inelastic deformation. Design using these equations will result in panel zones that may experience limited inelastic deformation under earthquake loading. In general, if code-specified drift limits are satisfied using analyses based on centerline dimensions of the beams and columns and include P-Δ effects, this can be considered as meeting the requirements to permit use of Equation J10-11 or J10-12. For further discussion on this issue, refer to Hamburger et al. (2009).

These Provisions also permit panel zone design to be based on tested connections. Considerable caution is needed with this approach if it leads to a panel zone that is significantly weaker than would otherwise be obtained using these Provisions. As described previously, weaker panel zones can increase the propensity for fracture at the beam-to-column connection and can also potentially adversely affect overall frame performance. These potential adverse effects should be carefully evaluated when considering the use of weaker panel zones based on tested connections.

2. Panel-Zone Thickness

Section E3.6e.3 requires a minimum doubler thickness of ¼ in. (6 mm) to prevent use of very thin doubler plates that may result in fabrication and welding difficulties or which may be too weak and/or flexible to adequately brace continuity plates. In addition, Equation E3-7 is required to minimize shear buckling of the panel zone during inelastic deformations. Thus, when the column web and web doubler plate(s) each meet the requirements of Equation E3-7, 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-E3.4 and the total panel zone thickness can be used in Equation E3-7.

When plug welds are required, Section E3.6e.2 requires a minimum of four plug welds. As a minimum, the spacing should divide the plate into rectangular panels in such a way that all panels meet the requirements of Equation E3-7. Additionally, since a single plug weld would seem to create a boundary condition that

is much different than a continuously restrained edge, it would be advisable to place the plug welds in pairs or lines, dividing the plate into approximately equal-sized rectangles. Plug welds, when used, should, as a minimum, meet the requirements of *Specification* Section J2.3.

An alternative detail is shown in Figure C-E3.5, where web doubler plates are placed symmetrically in pairs spaced away from the column web. In this configuration, both the web doubler plates and the column web are required to each independently meet Equation E3-7 in order to be considered as effective.

3. Panel Zone Doubler Plates

Requirements for attachment of doubler plates to columns have been updated for the 2016 edition of these Provisions based on recent research (Shirsat, 2011; Donkada, 2012; Gupta, 2013) as well as a reevaluation of past research (Mays, 2000; Lee et al., 2005a, 2005b). There are several different conditions using web doubler plates depending on the need for continuity plates and on the particular design conditions. Doublers may be placed against the column web or spaced away from the web, and they may be used with or without continuity plates.

Figure C-E3.6 shows doubler plates in contact with the web of the column. The research studies noted previously have shown that force is transferred to the doubler primarily through the welds connecting the vertical edges of the doubler

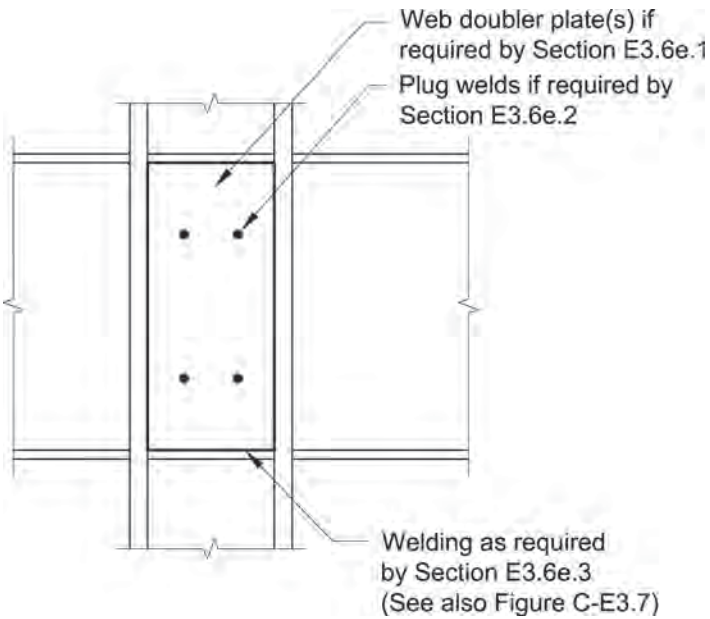
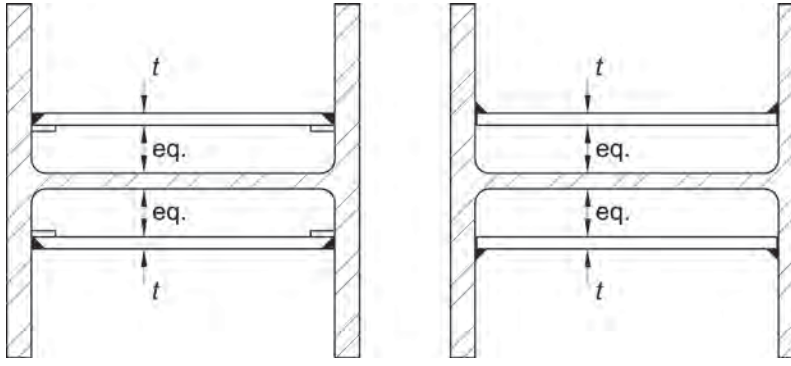


Fig. C-E3.4. Connecting web doubler plates with plug welds.

to the column flanges. Two options are available for this weld: a groove weld as shown in Figure C-E3.6(a) or a fillet weld as shown in Figure C-E3.6(b). When a groove weld is used, past versions of these Provisions required CJP groove welds. This was problematic, as there is no prequalified CJP groove weld joint detail in AWS D1.1/D1.1M or AWS D1.8/D1.8M for this type of joint. To address this problem, a prequalified doubler plate-to-column flange joint detail has been added to AWS D1.8/D1.8M clause 4.3. Further, these Provisions now designate this weld as a PJP groove weld that extends from the surface of the doubler to the column flange [as shown in Figure C-E3.6(a)] and in accordance with the detail in AWS D1.8/D1.8M. Based on a review of all available research, a judgment was made that routine ultrasonic testing of this weld is not justified. Consequently, the weld is now designated as PJP to reflect this view.

When a groove weld is used as shown in Figure C-E3.6(a), an additional concern is welding to the *k*-area of the column web. Welding into the flange/web fillet region, as shown in Figure C-E3.6(a), does not constitute welding in the *k*-area, although it clearly is very close to the *k*-area. To minimize the chances of welding to the *k*-area, it may be helpful to allow the doubler edge to land slightly within the flange/web fillet of the column. The Provisions permit a 1⁄16-in. (2 mm) gap between the doubler and the column web [Figure C-E3.6(a)] and allow the doubler to still be treated as being in contact with the web when landing within the flange/web fillet. In some cases, welding into the *k*-area, i.e., welding on the flat portion of the column web may be unavoidable, for example, because of variations in the actual as-rolled *k* dimension of the column.

Figure C-E3.6(b) shows the option of using a fillet weld to connect the vertical edge of the doubler to the column flange. Research (Shirsat, 2011; Donkada, 2012; Gupta, 2013) has shown that the state of stress at the edge of the doubler is dominated by vertical shear, but that significant horizontal normal stresses are also developed near the top and bottom of the doubler in the region of the beam

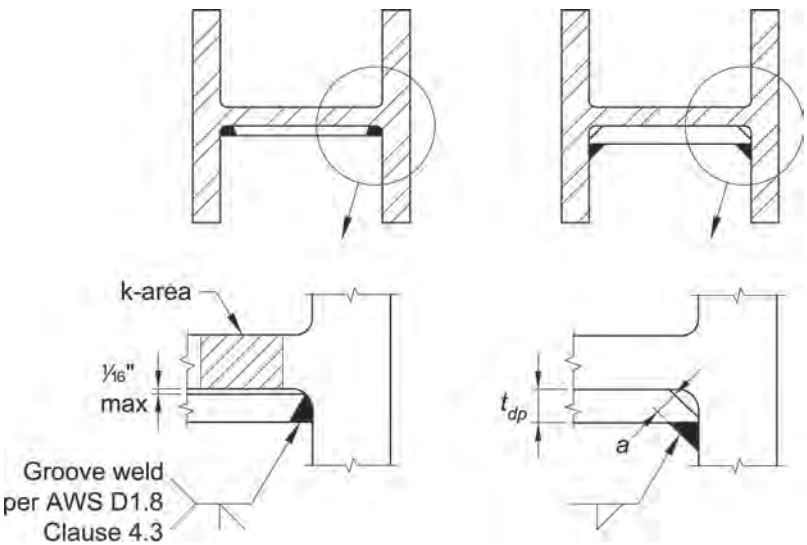


(a) CJP groove welded *(b) Fillet welded*

Fig. C-E3.5. Doubler plates spaced away from the web.

flanges. Consequently, the fillet weld along the vertical edge of the doubler is subject to both vertical shear forces and normal forces perpendicular to the axis of the weld. Although the weld sees both shear and tension, the Provisions state that the required strength of the fillet weld is equal to the available shear yielding strength of the full doubler plate thickness, where the available shear yielding strength is computed using *Specification* Equation J4-3. Sizing the fillet weld for shear will result in adequate strength for the weld loaded in tension, since the available strength of fillet welds loaded perpendicular to the weld's longitudinal axis is 50% higher than the available strength of a fillet welds loaded in shear along its longitudinal axis. For a doubler plate with a specified minimum yield stress of 50 ksi (345 MPa) and a weld filler metal with $F_{EXX} = 70$ ksi (485 MPa), a fillet weld with a leg size of 1.35 times the doubler plate thickness will develop the available shear yielding strength of the doubler plate. This same fillet weld size will also be adequate to develop the available tension yielding strength of the doubler plate. Thus, by sizing the fillet weld to develop the available shear strength of the doubler, the weld inherently has sufficient capacity to develop the available strength in pure tension or in a combined tension/shear stress state.

Using a fillet weld to connect the vertical edge of a doubler to the column flange when the doubler is in contact with the column web will normally require a bevel at the edge of the doubler to clear the column flange/web fillet as shown in Figure C-E3.6(b). When such a bevel is used, the shear strength of the doubler may be controlled by the section defined by the minimum distance from the



(a) Doubler plate groove welded to column flanges

(b) Doubler plate fillet welded to column flanges

Fig. C-E3.6. Doubler plate in contact with column web.

toe of the fillet weld to the edge of the doubler along the bevel. This minimum distance is shown by the dimension a in Figure C-E3.6(b). When the dimension, a, is less than the full doubler plate thickness, t_{dp}, then the shear yielding strength of the full doubler plate thickness cannot be developed. Consequently, the size of the fillet weld and the geometry of the bevel should be proportioned so that $a \geq t_{dp}$. This may require increasing the size of the fillet weld beyond that needed to satisfy weld strength requirements. Note, however, that large fillet welds placed on relatively thin doubler plates can produce considerable welding-induced distortion in the doubler. As an alternative, the thickness of the doubler plate, t_{dp}, can be increased so that shear yielding along the section defined by a provides the required panel zone shear strength.

When a single, thick doubler plate in contact with the column web is welded to the column flanges, considerable welding-induced distortion may occur in the column flanges. These welding distortion problems can be somewhat alleviated by splitting the doubler and placing doublers of similar thickness on each side of the web. For example, a 1-in.- (25 mm) thick doubler plate is needed to provide adequate panel zone shear strength. This can be accommodated by using a single 1-in.- (25 mm) thick plate on one side of the column web, or by using ½-in.- (12 mm) thick doubler plates on both sides of the column web. The decision to split a doubler can be made in conjunction with the fabricator, or it can be left to the discretion of the fabricator.

As an alternative to placing doubler plates in contact with the web, it is also permissible to use doubler plates spaced away from the web, as shown in Figure C-E3.5. Spaced doubler plates must be provided in symmetric pairs, and can be connected to the column flanges using CJP groove welds [Figure C-E3.5(a)], fillet welds [Figure C-E3.5(b)], or built-up PJP groove welds. If CJP groove welds are used, removal of backing bars is not required.

When doubler plates are used without continuity plates, they are required to extend a minimum of 6 in. (150 mm) above and below the deepest beam framing into the column (Figure C-E3.7). This extension permits a more uniform transfer of stress to the doubler and to the doubler-to-column flange weld in the region near the beam flanges. It also places the termination of the doubler-to-column flange weld away from the highly stressed region near the beam flanges. When doubler plates are extended above and below the joint as shown in Figure C-E3.7, research (Shirsat, 2011; Donkada, 2012; Gupta, 2013) has shown that fillet welds are not required along the top and bottom edges of the doubler plate. The only exception is when either the doubler plate or column web thickness does not satisfy Equation E3-7. In this case, minimum size fillet welds are required along the top and bottom edges of the doubler plate to help maintain stability of the panel zone, in addition to the plug welds required in Section E3.6e.2. When fillet welds are provided along the top and bottom edges of the doubler plate, these welds should not extend into the k-area of the column.

When doublers are used with continuity plates, they may be located between the continuity plates, or they may be extended above and below the continuity plates. Figure C-E3.8(a) shows an example of an extended doubler plate used with continuity plates. This case requires that the continuity plate be welded to the doubler plate. Recent research examining this case (Donkada, 2012; Gupta, 2013) has shown that welding the continuity plate to the doubler plate does not substantially change the shear force in the doubler plate. That is, the forces and state of stress in the doubler plate are very similar with or without the continuity plate. However, all requirements of the *Specification* must be satisfied. *Specification* Section J10.8 assumes a model in which the stiffener transfers the difference in force between the required strength (the flange force) and available strength of the unstiffened column. The doubler plate, by itself, must have sufficient shear strength to resist the difference between the flange force and the available strength of the unstiffened column computed according to *Specification* Section J10 for the lesser of the limit states of flange local bending, web local yielding, and web local crippling. The required shear strength computed according to the difference in these forces need not be added with the shear force in the doubler plate due to the panel zone shear force. For some SMF beam-to-column joint configurations, *Specification* Section J10 may indicate that no continuity plates are required, but Equation E3-8 and E3-9 will still require continuity plates. For these cases, no special consideration is needed in the design of the doubler plate. Recent research has also shown that no welds are needed along the top and bottom edges of an extended doubler plate when continuity plates are present. In cases where the doubler-plate thickness does not satisfy Equation E3-7, the continuity plate serves to help restrain buckling

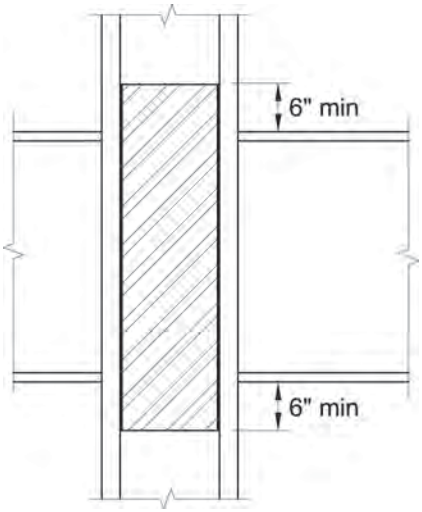
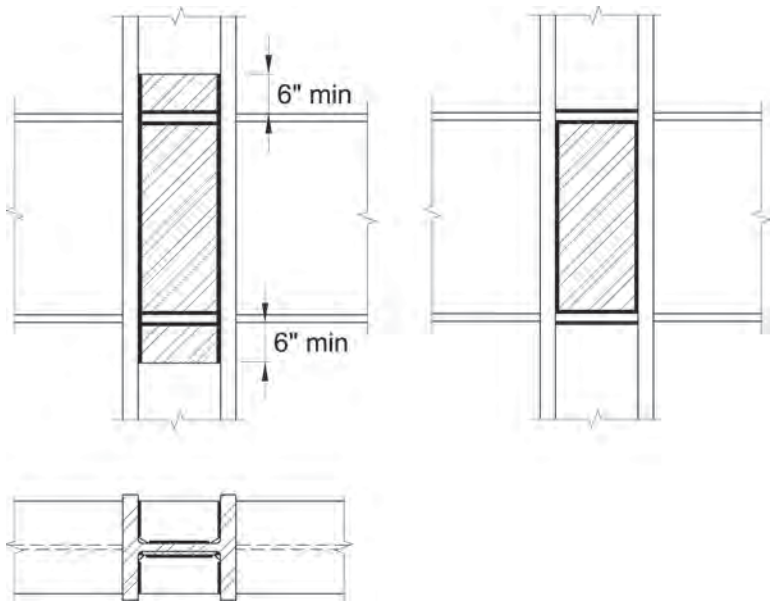


Fig. C-E3.7. Doubler plates used without continuity plates.

of the doubler plate, and consequently, welds at the top and bottom edges of the doubler plate are not needed.

Figure C-E3.8(b) shows an example of a doubler plate placed between continuity plates. For this case, welding the doubler to the continuity plate is required. This weld should extend over the full width of the continuity plate between *k*-areas of the column, and should be designed to develop at least 75% of the shear yielding strength of the doubler over its contact length with the continuity plate. The doubler-to-continuity plate weld helps transfer force to the doubler and reduces stress concentrations near the ends of the doubler-to-column flange welds. For a doubler plate with a specified minimum yield stress of 50 ksi (345 MPa) and a weld filler metal with $F_{EXX} = 70$ ksi (485 MPa), the strength requirement for the doubler-to-continuity plate weld can be satisfied by specifying a PJP weld with an effective throat equal to the doubler plate thickness. Other options for welding the doubler to the continuity plate are provided in AISC Design Guide 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999). Detailing this weld requires consideration of how the continuity plate-to-column weld will be combined with the doubler-to-continuity plate weld. Detailing and sequencing of these combined welds can be made in conjunction with the fabricator.



(a) Doubler plate extended beyond continuity plates

(b) Doubler plate placed between continuity plates

Fig. C-E3.8. Doubler plate used with continuity plates.

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.

6f. Continuity Plates

Beam-flange continuity plates serve several purposes in moment connections. They help to distribute beam-flange forces to the column web, they stiffen the column web to prevent local crippling under the concentrated beam flange forces, and they minimize stress concentrations that can occur in the joint between the beam flange and the column due to nonuniform stiffness of the column flange.

1. Conditions Requiring Continuity Plates

In the 2010 Provisions, two equations (E3-8 and E3-9) were provided which determined conditions under which continuity plates were not required opposite wide-flange beams in wide-flange, built-up I-shape, or cruciform columns. In the current Provisions, former Equation E3-8 is deleted, in favor of an analysis using *Specification* Section J10. Equations in the User Note are provided for calculation of the required strength at the column face for the local limit states in the column that are required to be checked using *Specification* Section J10.

Equations E3-8 and E3-9 are the same as in 2010 and are intended to provide a lower bound on the stiffness of the column flange based on its thickness in relation to the width of the beam flange. Column flanges not meeting the limits given in these equations will deflect more under the beam flange load which may lead to undesirable stress patterns at the beam-to-column flange weld. Justification for the use of Equation E3-8 is based on studies discussed in FEMA 355D (FEMA, 2000e). Subsequent research by Lee et al. (2005a) confirmed the adequacy of designs based on these equations.

The design equations for continuity plates have been developed based on consideration of the behavior of columns in lower stories of buildings, where the column extends a considerable distance above the top flange of the connected beam. These equations do not apply in the top story of a building, where the column terminates at approximately the level of the top flange of the beam. In such cases, beam-flange continuity plates or column cap plates, having a thickness not less than that of the connected beam flange, should be provided. Figure C-E3.9 presents a detail for such a connection, where the beam flange is welded directly to the cap plate and the cap plate is welded to the column so as to deliver the beam-flange forces to the column web.

Alternatively, if the column projects sufficiently above the beam top flange, the preceding methods can be considered valid. Although comprehensive research to establish the necessary distance that the column must extend above the beam for this purpose has not been performed, it may be judged to be sufficient if the column is extended above the top beam flange a distance not less than $d_c/2$ or

$b_f/2$, whichever is less, where d_c is the depth of the column and b_f is the width of the column flange.

The 2010 Provisions included equations to calculate the requirement for continuity plates in boxed wide-flange columns. The basis for these equations has not been established; therefore, the equations have been removed from the Provisions. It is recommended that designers perform appropriate analyses, consult research, and/or conduct tests to determine the need for continuity plates for box columns. Analyses to demonstrate that continuity plates are not needed should demonstrate that the nonlinear stress and strain patterns in the beam-to-column flange welds are consistent with those of tested connections.

2. Continuity Plate Requirements

Requirements to determine the thickness of continuity plates are based on studies by FEMA 355D (FEMA, 2000e) and Lee et al. (2005a). Continuity plates with these minimum thicknesses have been shown to have adequate stiffness and strength to enable a relatively uniform distribution of strain across the flange of the connecting girder.

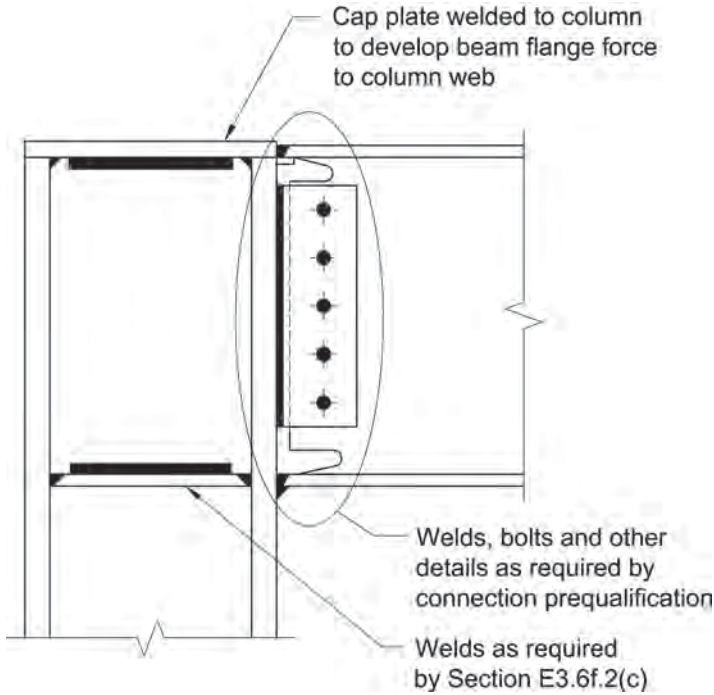


Fig. C-E3.9. Cap plate detail at column top.

The 2010 Provisions required a minimum continuity plate thickness for two-sided connections equal to the full thickness of the thicker beam flange, largely based on the use of full-thickness continuity plates in successfully tested connections. Although the references noted indicate that continuity plates of thickness equal to one-half of the thicker beam flange thickness can provide adequate performance for these connections, a more conservative value of three-quarters of the thicker beam flange is used to address the range of demands that may be seen in two-sided connections as compared to one-sided connections.

3. Continuity Plate Welding

The connection of continuity plates to column webs is designed to be capable of transmitting the maximum shear forces that can be delivered to the connection. This may be limited by the beam-flange force, the shear strength of the continuity plate itself, the welded joint between continuity plate and column flange, or the strength of the column panel zone.

The Provisions require that continuity plates be attached to column flanges with CJP groove welds in order that the strength of the beam flange can be properly developed into the continuity plate. Research by Lee et al. (2005a, 2005b) demonstrated that properly sized fillet welded connections also performed adequately for this purpose, although this is not yet permitted by the Provisions. For single-sided connections in which a moment-connected beam attaches to only one of the column flanges, it is theoretically not necessary to attach the continuity plate to the column flange that does not have a beam attached because there is no quantifiable force to transfer from the column flange to the continuity plate. In such cases, acceptable performance is expected if the continuity plate is attached to the column with a pair of minimum-size fillet welds.

6g. Column Splices

In the 1997 Provisions, there were no special requirements for column splices in SMF systems other than those currently given in Section D2.5. The requirement in Section D2.5a was intended to address column bending at the splice by requiring splices to be at least 4 ft (1.2 m) or one-half the column clear height from the beam-to-column connection. This requirement was based on general recognition that in elastic analyses of moment frames the columns are typically bent in double curvature with an inflection point somewhere near the middle of the column height and, therefore, little bending of the column was expected at the splice.

Nonlinear analyses performed during the FEMA/SAC project following the Northridge earthquake, and subsequently (Shen et al., 2010; Galasso et al., 2015) clearly demonstrated that bending moments in the mid-height of columns can be substantial and that, in fact, the columns may be bent in single curvature under some conditions. Given this fact, and recognition of the potential for severe damage or even collapse due to failure of column splices, the need for special provisions for splices of moment frame columns was apparent.

The provisions of Section E3.6g are intended to ensure that a stress of 55 ksi (380 MPa) (i.e., R_yF_y for A992/A992M steel) is developed in the flange of the smaller column, either through use of CJP groove welds or another connection that provides similar strength, and that the shear strength of the splice is sufficient to resist the shear developed when M_{pc} occurs at each end of the spliced column.

The exception permitting the use of partial-joint-penetration (PJP) welds in column splices is based on recent testing (Shaw et al., 2015). This testing, along with fracture mechanics simulation (Stillmaker et al., 2015) has demonstrated that if detailed appropriately, splices constructed with PJP groove welds provide strength similar to splices with CJP groove welds, and are able to develop a stress of 55 ksi (380 MPa) in the smaller column. Since the 1997 Provisions, PJP welds have not been permitted in splices (in SMF and IMF) because the unfused weld root in the PJP weld was considered to be a potential initiator of fracture. However, this recent research shows that fracture toughness demands at the weld root are lower than the toughness capacity implied by minimum Charpy V-notch toughness requirements, if the requirements of Section E3.6g are satisfied. The scientific basis for these requirements is as follows:

(1) The fracture toughness demand is directly related to the length of the unfused weld root relative to the flange thickness. Requiring the effective throat thickness to be at least 85% of the thinner flange limits the length of the unfused weld root relative to flange thickness.

(2) The potential fracture plane is at the location of the weld root. Requiring the thicker flange to be 5% thicker than the thinner flange, along with the requirement for the transition reinforcement, limits the fracture toughness demand at the weld root by preserving a sufficient net section in the fracture plane. Similar considerations motivate the detailing requirements for the web.

(3) The requirement for smooth, tapered transitions is based on ensuring similarity to the specimens tested by Shaw et al. (2015), and the general undesirability of sharp flaws and stress risers in welded connections.

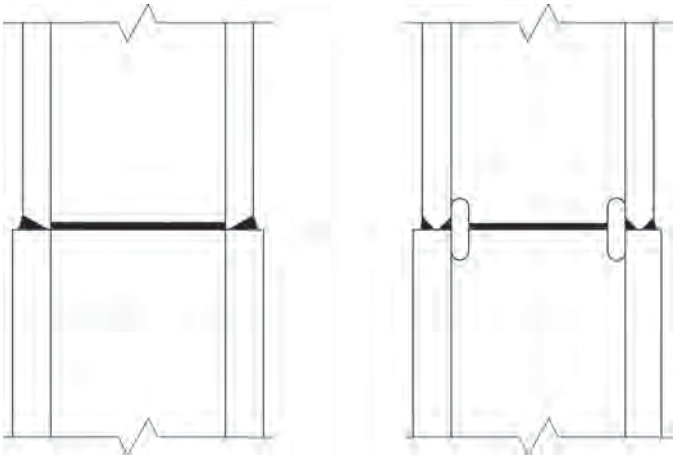
Figure C-E3.10 illustrates details that are compliant with the Provisions. Figure C-E3.10(a) shows a PJP splice detail with a single weld deposited from the outside of the flange. This may be feasible for thinner flanges [thickness less than 2½ in. (63 mm)] and does not require an access hole in the column web. Figure C-E3.10(b) shows a PJP splice detail with a double-sided flange weld, which may be required for thicker flanges.

E4. SPECIAL TRUSS MOMENT FRAMES (STMF)

1. Scope

Truss-girder moment frames have often been designed with little or no regard for truss 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-E4.1 results in excessively large story drifts in building frames subjected to earthquake ground motions with peak accelerations on the order of 0.4g to 0.5g.



(a) Single-bevel flange weld *(b) Double-bevel flange weld with access hole*

Fig. C-E3.10. Splice details with partial-penetration-groove welds.

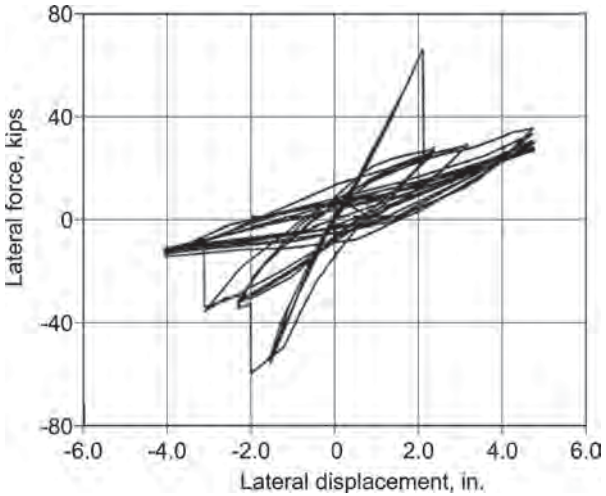


Fig. C-E4.1. Strength degradation in undetailed truss girder.

Research 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-E4.2, 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. STMF have been validated by extensive testing of full-scale subassemblages with story-high columns and full-span special truss girders. As illustrated in Figure C-E4.3, STMF are ductile with stable hysteretic behavior. The stable hysteretic behavior continues for a large number of cycles, up to 3% story drifts.

2. Basis of Design

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.

3. Analysis

3a. Special Segment

The design procedure of STMF is built upon the concept that the special segment of truss girders will yield in shear under the prescribed earthquake load combinations,

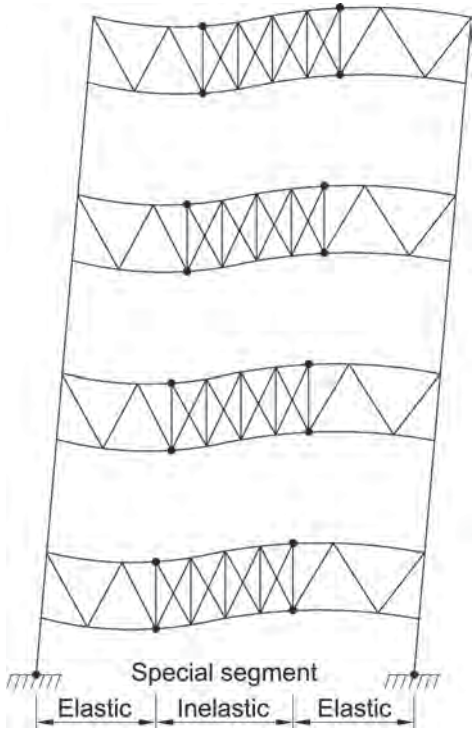


Fig. C-E4.2. Intended yield mechanism of STMF with diagonal web members in special segment.

while all other frame members and connections remain essentially elastic. Thus, for the purpose of determining the required shear strength of special segments the truss girders can be treated as analogous beams in moment frames (Rai et al., 1998). The chord and diagonal members of the special segments are then designed to provide the required shear strength as specified in Section E4.5.

3b. Nonspecial Segment

All frame members and connections of STMF outside the special segments must have adequate strength to resist the combination of factored gravity loads and maximum expected shear strength of the special segments by accounting for reasonable strain-hardening and material overstrength. For this purpose, one of several analysis approaches can be used. One approach is to consider the equilibrium of properly selected elastic portions (sub-structures) of the frame and perform elastic analysis. Alternatively, a nonlinear static pushover analysis of a model of the entire frame can be carried out up to the maximum design drift. The intended yielding members of the special segments, including chord and diagonal members and column bases, are modeled to behave inelastically, while all others are modeled (or "forced") to behave elastically.

4. System Requirements

4a. Special Segment

It is desirable to locate the STMF special segment near midspan 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% of the truss span length provides a reasonable limit on the ductility demand, while the upper limit of 50% of the truss span length represents more of a practical limit.

The required strength of interconnection for X-diagonals is intended to account for buckling over half the full diagonal length (El-Tayem and Goel, 1986; Goel and Itani,

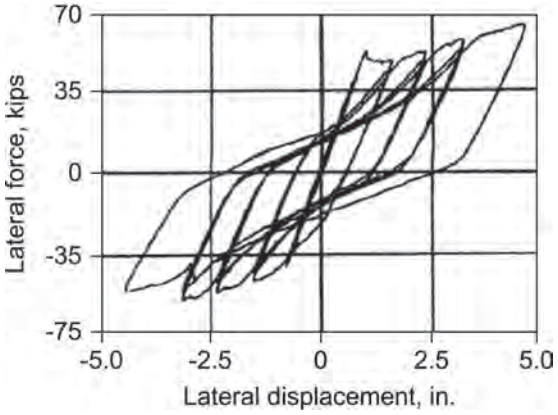


Fig. C-E4.3. Hysteretic behavior of STMF.

1994b). It is recommended that half the full diagonal length be used in calculating the available compressive strength of the interconnected X-diagonal members in the special segment.

Because it is intended that the yield mechanism in the special segment form over its full length, no major structural loads should be applied within the length of the special segment. In special segments with open Vierendeel panels, in other words, when no diagonal web members are used, significant structural loads should be avoided. Accordingly, a restrictive upper limit is placed on the axial load in diagonal web members due to gravity loads applied directly within the special segment.

4b. Stability Bracing of Trusses

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 requirements for truss chord members have been slightly revised to make them consistent with what was used successfully in the original testing program.

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

Columns should be laterally braced at the points of connection with the truss members in order to provide adequate stability during expected cyclic deformations of the frames. A lateral bracing requirement has been added which is partly based on what was used successfully in the original testing program.

5. Members

5b. 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 minimum shear strength in the special segment through flexural yielding of the chord members and to limit the axial load to a maximum value. Plastic analysis can be used to determine the required shear strength of the truss special segments under the earthquake load combination.

5c. Expected Vertical Shear Strength of Special Segment

STMF are required to be designed to maintain essentially 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 outside the special segments are to be designed for calculated loads by applying the combination of gravity loads and equivalent lateral loads 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 E4-5, 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% (Basha and Goel, 1994). Tests (Jain et al., 1978) on axially loaded members have shown that $0.3P_{nc}$ is representative

of the average nominal post-buckling strength under cyclic loading. Based on a more recent study by Chao and Goel (2008) the first two terms of Equation E4-5 were revised in the 2010 Provisions to give a more accurate estimate of contribution from the chord members.

Equation E4-5 was formulated without considering the contribution from any intermediate vertical members within the special segment other than those at the ends of the special segment. In cases where those intermediate vertical members possess significant flexural strength, their contribution should also be included in calculating the value of V_{ne}. Recent full-scale STMF experimental testing indicated that intermediate vertical members can significantly increase V_{ne}. A modified equation which considers the contribution of intermediate vertical members has been proposed by Chao et al. (2015).

5d. Width-to-Thickness Limitations

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-to-thickness ratios that are less than $0.18\sqrt{E/F_y}$ 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.

5e. Built-Up Chord Members

Built-up chord members in the special segment can be subjected to rather large rotational demands at the plastic hinges requiring close stitch spacing in order to prevent lateral-torsional buckling of the individual elements. Based on the findings from a recent experimental study (Parra-Montesinos et al., 2006), a stitch spacing requirement for chord members in the special segment has been added.

5f. Protected Zones

When special segments yield under shear, flexural plastic hinges will form at the ends of the chord members. Therefore, those regions are designated as protected zones. Also, included in the protected zones are vertical and diagonal members of the special segments, because those members are also expected to experience significant yielding. Recent component testing performed by Chao et al. (2015) indicates that the plastic rotation capacity of the chord members can be considerably compromised when the vertical members or stiffeners are welded to the chord members at the end of the special segment. Full-scale STMF testing shows that the plastic hinges can freely extend when the end connection of vertical members at the ends of the special segment is not welded to the chord members.

6. Connections

6a. Demand Critical Welds

Refer to the commentary on Section E3.6a.

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

The diagonal members of the special segments are expected to experience large cyclic deformations in axial tension and post-buckling compression. Their end connections must possess adequate strength to resist the expected tension yield strength.

6c. Column Splices

The requirements in this section are identical to those in Section E3.6g. See Commentary Section E3.6g for further discussion.

E5. ORDINARY CANTILEVER COLUMN SYSTEMS (OCCS)

2. Basis of Design

ASCE/SEI 7 (ASCE, 2016) includes two types of cantilever column systems: ordinary and special. The ordinary cantilever column system (OCCS) is intended to provide a minimal level of inelastic rotation capability at the base of the column. This system is permitted in seismic design categories B and C only, and to heights not exceeding 35 ft. A low seismic response modification coefficient, R, of 1.25 is assigned due to the system's limited inelastic capacity and lack of redundancy. The OCCS has no requirements beyond those in the *Specification* except as noted in Section E5.4a.

4. System Requirements

4a. Columns

ASCE/SEI 7 limits the required axial load on columns in these systems under the load combinations including the overstrength seismic load to 15% of the available strength. This limitation is included in these provisions. Columns in OCCS would be prone to P-Δ collapse if high axial loads were permitted.

E6. SPECIAL CANTILEVER COLUMN SYSTEMS (SCCS)

2. Basis of Design

ASCE/SEI 7 (ASCE, 2016) includes two types of cantilever column systems, ordinary and special. The special cantilever column systems (SCCS) is intended to provide a limited level of inelastic rotation capability at the base of the column. This system is permitted in seismic design categories B through F, but is limited to heights not exceeding 35 ft. A relatively low seismic response modification coefficient, R, of 2.5 is assigned due to the system's limited inelastic capacity and lack of redundancy.

4. System Requirements

4a. Columns

ASCE/SEI 7 limits the required axial load on columns in these systems under the load combinations including the overstrength seismic load to 15% of the available strength. This limitation is included in these provisions. Columns in SCCS would be

prone to P-Δ collapse if high axial loads were permitted because even modest rotations at the base of the columns can translate into significant drift at the top where the majority of the gravity load is generally applied.

4b. Stability Bracing of Columns

Stability bracing of columns at the spacing required for moderately ductile members is required. Although the columns themselves must satisfy requirements for highly ductile members, the wider spacing of braces permitted is considered to be adequate because of the relatively low inelastic demand expected and the practical difficulty in achieving bracing in many of these structures. For structures where there is no reasonable way to meet bracing requirements, need for bracing may be precluded by selecting appropriately proportioned members.

5. Members

5a. Basic Requirements

The column members are required to satisfy the width-to-thickness and other provisions for highly ductile members. The intention is to preclude local buckling at the hinging location (bottom of the column), which in this type of structure, with little redundancy, could lead rapidly to collapse.

5b. Column Flanges

Abrupt changes in beam flange area in locations of high strain, as occurs in plastic hinge regions at the base of SCCS columns, can lead to fracture due to stress concentrations.

5c. Protected Zones

For commentary on protected zones see Commentary Section D1.3.

6. Connections

6a. Demand Critical Welds

For general commentary on demand critical welds, see Commentary Section A3.4. For additional commentary appropriate to column splices and column-to-base plate connections, see Commentary Section E3.6a.

6b. Column Bases

It is apparent that a column base in the SCCS must be capable of developing the moment capacity of the column, including overstrength and strain hardening. Detailed requirements are provided in Section D2.6 and commentary is provided in the corresponding commentary section.

CHAPTER F

BRACED-FRAME AND SHEAR-WALL SYSTEMS

F1. ORDINARY CONCENTRICALLY BRACED FRAMES (OCBF)

1. Scope

Ordinary concentrically braced frames (OCBF) have minimal design requirements compared to other braced-frame systems. The Provisions assume that the applicable building code significantly restricts the permitted use of OCBF and specifies a low R factor so that ductility demands will be low. Specifically, it is assumed that the restrictions given in ASCE/SEI 7 (ASCE, 2016) govern the use of the structural system.

The scope includes OCBF above an isolation system. The provisions in Section F1.7 are intended for use in the design of OCBF for which forces have been determined using R_I equal to 1.0. R_I is defined in ASCE/SEI 7 as the "numerical coefficient related to the type of seismic force-resisting system above the isolation system." Such OCBF are expected to remain essentially elastic during design level earthquakes and, therefore, provisions that are intended to accommodate a higher level of inelastic response, such as Section F1.4a, are not required for their design.

2. Basis of Design

OCBF are not expected to be subject to large inelastic demands due to the relatively low R factor assigned to the system in ASCE/SEI 7.

3. Analysis

Due to the expected limited inelastic demands on OCBF, an elastic analysis is considered sufficient when supplemented with use of the overstrength seismic load as required by these Provisions.

4. System Requirements

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

V- and inverted-V-type bracing can induce a high unbalanced force in the intersecting beam. Unlike the special concentrically braced frame (SCBF) provisions, which require that the beams at the intersections of such braces be designed for the expected strength of the braces to prevent a plastic hinge mechanism in the beam, the corresponding OCBF provisions permit the beam design on the basis of the maximum force that can be developed by the system. This relief for OCBF acknowledges that, unlike SCBF, the beam forces in an OCBF frame at the time of an imminent system failure mode could be less critical than those due to the expected strength of the connecting braces. See Commentary Section F2.6c.1 for techniques that may be used to determine the maximum force developed by the system.

4b. K-Braced Frames

K-bracing can have very poor post-elastic performance. After brace buckling, the action of the brace in tension induces large flexural forces on the column, possibly leading to buckling. No adequate design procedures addressing the high-consequence stability issues are available.

4c. Multi-Tiered Braced Frames

A detailed description of the characteristics of multi-tiered braced frames is provided in the commentary for special concentrically braced frames. Due to the reduced level of ductility required for a multi-tiered ordinary concentrically braced frame (MT-OCBF) as compared to a multi-tiered SCBF (MT-SCBF) ($R = 3.25$ versus $R = 6$), a simpler set of design requirements is provided for the MT-OCBF. In this approach, the basis of the design is an elastic analysis of the frame with an R of 3.25. This seismic design force level is used for the braces only. The connections, struts and columns are designed for seismic forces increased by a factor of 3 to make these elements more robust. This corresponds to 1.5 times the overstrength seismic loads, i.e., to an R value equal to $3.25/3 = 1.08$, which is approximately equivalent to force levels associated with elastic response. Such higher required strength for the connections, columns and struts aims at ensuring that these elements can resist the maximum forces imparted by the braces. Failure of connections or struts may induce large unbalanced horizontal loads on the columns. This, in turn, may endanger the frame integrity in view of the fact that intermediate tier levels are not connected to other lateral load-resisting elements of the structure. For the columns, the amplified design loads is an indirect, simpler means of providing the columns with sufficient strength to resist in-plane flexural demands resulting from nonuniform brace forces and deformations in adjacent tiers. The benefits of designing the struts and strut connections to torsionally brace the columns of the multi-tiered braced frame were demonstrated by research (Stoakes and Fahnestock, 2013) and are incorporated into these provisions also.

For the special case of tension-only bracing proportioned such that the controlling slenderness ratio of each brace is 200 or more, it is recognized that the columns, struts and connections are not prone to problems associated with compression buckling of the brace since these braces have little overstrength from compression or flexural strength. Horizontal unbalanced brace loads due to brace buckling are also small. As a result, the design requirements for the brace connections, columns and struts revert to the basic requirements for an OCBF frame. However, because the frame is not connected at every tier level to the other lateral load-resisting elements in the building (i.e., no diaphragm is present at the intermediate tier levels to help distribute loads to other lateral load-resisting systems), there is a potential for progressive yielding in multi-tier frames that results in flexural demand on the columns in the plane of the frame. As a result, the column is checked for in-plane bending due to the calculated difference in shear strength between tier levels. As a minimum, this force level is prescribed to be 5% of the larger shear capacity of the tier above and below that tier level. This minimum force level is intended to also capture potential differences in brace strength due to material yield strength variability. These potential in-plane force and

bending demands can be shared with additional columns by appropriately connecting these additional columns to the braced frame at each tier level. It is noted that this same requirement is not applied to MT-OCBF frames with tension-compression bracing (controlling slenderness ratio of each brace less than 200) since these columns are already penalized by the use of the higher effective load amplification factor of 3 for these frames.

5. Members

5a. Basic Requirements

Only moderate ductility is expected of OCBF. Accordingly, in the 2010 Provisions, the member ductility requirement for braces was modified to require moderately ductile members.

5b. Slenderness

In V- and inverted V-braced frames, braces with large slenderness ratios are not permitted. This restriction is intended to limit the unbalanced forces that develop in framing members after brace buckling; see Commentary Section F2.4c.

5c. Beams

In past versions of the Provisions it was assumed that beams and their connections were treated as collectors, and thus beams were required to be designed for the overstrength seismic load in accordance with ASCE/SEI 7. This requirement has been specifically added to the 2016 Provisions to provide greater clarity.

6. Connections

6a. Brace Connections

Bracing connections are designed for forces corresponding to the overstrength seismic load with exceptions that allow for the force to be limited to the expected brace strength. The intent is to ensure that brace yielding or buckling occurs prior to failure of a connection limit state. Net section rupture of the member is to be included with connection limit states. Allowing the required strength of a brace connection to correspond to the overstrength seismic load is considered appropriate for systems designed for limited ductility.

The Provisions permit that bolt slip be designed for a lower force level than is required for other limit states when oversized holes are used in accordance with Section D2.2(c) Exception (1). This reflects the fact that bolt slip does not constitute connection failure and that the associated energy dissipation can serve to reduce seismic response. Other limit states, such as bolt shear and bolt bearing/tearout, are required to be designed for the overstrength seismic load subject to the exceptions discussed previously.

7. Ordinary Concentrically Braced Frames above Seismic Isolation Systems

Above isolation, system and member ductility demands are greatly reduced compared to nonisolated OCBF. Accordingly, beams are not required to resist forces corresponding to unbalanced brace nonlinear behavior. However, most engineers recognize that, since the intent of the code is now to preclude collapse in the maximum credible earthquake, should an earthquake occur that is larger than those considered in the design, some ductility of the system is desirable for the survivability of the structure, and certain basic requirements remain: amplified loads for the design of beams, columns, and connections, and the elimination of the nonductile K-bracing configuration.

The requirements in this section are similar to Section F1.5, except that the L_c/r limitation is applied to all braces. Tension-only bracing is not considered to be appropriate for use above isolation systems under the conditions permitted.

The requirements of Section F1.4a are considered to be excessive for OCBF above the isolation system because the forces on the system are limited and buckling of braces is not anticipated. The only requirement that remains applicable from Section F1.4a is for the beams to be continuous between columns.

F2. SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)

1. Scope

Special concentrically braced frames (SCBF) are a type of concentrically braced frame; that is, braced frames in which the centerlines of members that meet at a joint intersect at a point, thus forming a vertical truss system that resists lateral loads. A few common types of concentrically braced frames are shown in Figure C-F2.1, including diagonally braced, X-braced, and V-braced (or inverted V-braced). Use of tension-only bracing in any configuration is not permitted for SCBF. Because of their geometry, concentrically braced frames provide complete truss action with members

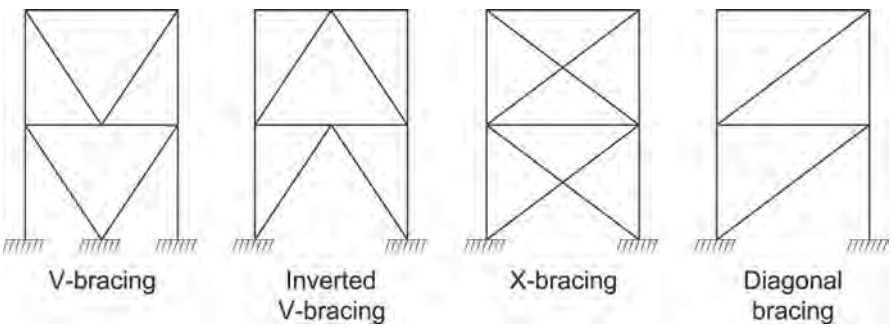


Fig. C-F2.1. Examples of concentric bracing configurations.

subjected primarily to axial loads 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.

2. Basis of Design

SCBF are distinguished from OCBF (and from braced frames that are part of steel systems not specifically detailed for seismic resistance, e.g., designed with $R = 3$) by enhanced requirements for ductility. Accordingly, provisions were developed so that the SCBF would exhibit stable and ductile behavior in the event of a major earthquake.

During a severe earthquake, bracing members in a concentrically braced frame are subjected to large deformations in cyclic tension and compression. In the compression direction flexural buckling causes the formation of flexural plastic hinges in the brace or gusset plates as it deforms laterally. These plastic hinges are similar to those in beams and columns in moment frames. Braces in a typical concentrically braced frame can be expected to yield and buckle at rather moderate story drifts of about 0.3% to 0.5%. 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 must be properly detailed.

Damage during past earthquakes and that observed in laboratory tests of concentrically braced frames with little consideration of ductile member design and detailing has generally resulted from the limited ductility and corresponding brittle failures, which are usually manifested in the rupture of connection elements or bracing members. The lack of compactness in braces results in severe local buckling, which imposes a high concentration of flexural strains at the location of buckling and ultimately provides a low level of ductility. Large story drifts that result from early brace ruptures 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 exhibited by such systems. Extensive analytical and experimental work by Goel has shown that improved design parameters, such as limiting width-to-thickness (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 (Goel, 1992b; Goel, 1992c). 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 verified 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 the 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 in Wallace and Krawinkler (1985) and Tang and Goel (1989).

Since the stringent design and detailing requirements for SCBF are expected to produce more reliable performance when subjected to cyclic deformation demands imposed by severe earthquakes, model building codes have reduced the design load level below that required for OCBF.

3. Analysis

While SCBF are typically designed on the basis of an elastic analysis, their expected behavior includes significant nonlinearity due to brace buckling and yielding, which is anticipated in the maximum credible earthquake. Braced-frame system ductility can only be achieved if beams and column buckling can be precluded. Thus there is a need to supplement the elastic analysis in order to have an adequate design.

The required strength of braces is typically determined based on the analysis required by ASCE/SEI 7. The analysis required by this section is used in determining the required strength of braced-frame beams and columns, as well as of brace connections, as it is necessary to design these elements to resist forces corresponding to brace yielding.

Prior to the 2010 Provisions, the expected nonlinear behavior of SCBF was addressed through a series of design rules that defined required strengths of elements superseding those derived using elastic elements. These included:

(1) Forces for beams in V- and inverted V-braced frames

(2) Forces for the design of brace connections

(3) Forces for column design

These design rules were intended to approximate forces corresponding to inelastic response without requiring an inelastic analysis.

While these requirements addressed the most important shortcomings of elastic analysis, several other cases have been identified, including:

(1) Beams not intersected by braces in the two-story X-braced configuration (such as the beam at the third floor in Figure C-F2.2(a).

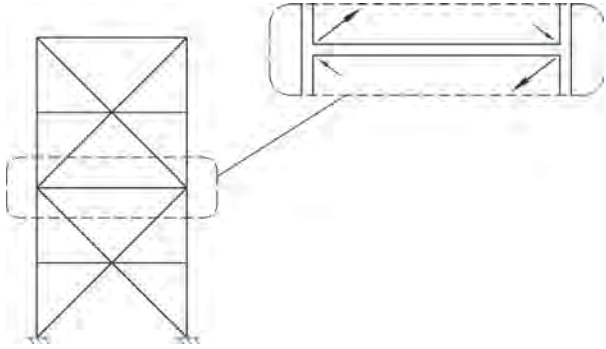
(2) Interior columns in multi-bay braced frames. See Figure C-F2.2(b).

Rather than creating new (and increasingly complicated) design rules to address these omissions in previous Provisions, it was decided to simply mandate explicit consideration of the inelastic behavior by requiring a plastic-mechanism analysis, the simplest form of inelastic analysis. It is naturally desirable that engineers performing analyses of ductile systems give some thought to the manner in which they will behave.

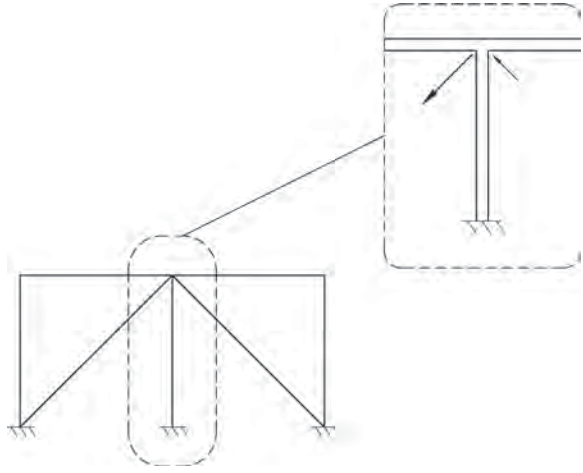
Because the compression behavior of braces differs substantially from the tension behavior, two separate analyses are required:

(1) An analysis in which all braces have reached their maximum forces

(2) An analysis in which tension braces are at their maximum strength level and compression braces have lost a significant percentage of their strength after buckling



(a) Post-elastic flow of forces through braced-frame beam



(b) Post-elastic flow of forces through interior braced-frame column

Fig. C-F2.2. Examples of post-elastic flow of forces in braced-frame systems.

The first-mode of deformation is considered when determining whether a brace is in compression or in tension. That is, the columns are considered to be inclined in one direction rather than in reverse curvature (see Figure C-F2.3). Consideration must also be given to the behavior when the columns are inclined in the opposite direction.

Consistent with previous editions of these Provisions, when maximum axial forces are calculated for columns, the engineer is permitted to neglect the flexural forces that result from the design story drifts. This permits straightforward determination of seismic forces.

The analysis requirements utilize the expected strengths of braces in tension and compression. Tests have shown that typical bracing members demonstrate a minimum residual post-buckling compressive strength of about 30% of the initial compressive strength (Hassan and Goel, 1991).

The provisions require design of columns to resist forces corresponding to the development of the full plastic mechanism (that is, yielding and buckling of all braces), unless a nonlinear analysis in accordance with Section C3 demonstrates that a lower force can be used with sufficient reliability. Previous editions allowed the use of the overstrength seismic load in lieu of the full capacity of the connecting braces, based on the expectation of reduced likelihood of simultaneous yielding at multiple floors. Unfortunately, research indicates that the reduction is less dramatic than anticipated and may not be significant for certain building configurations (Richards, 2009).

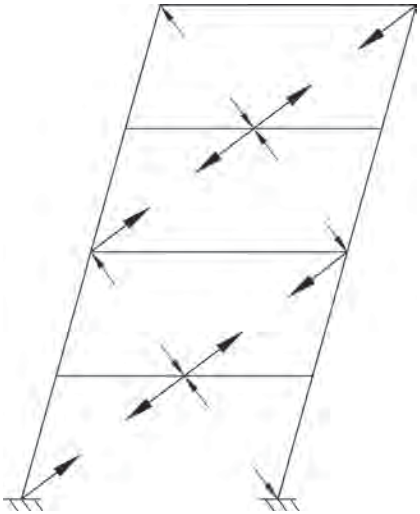


Fig. C-F2.3. Anticipated braced-frame mechanism.

4. System Requirements

4a. Lateral Force Distribution

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. It is envisioned that such an exception would apply to a small number of braces in the structure. It is generally preferable to have braces sized in proportion to their required strength. Where braces have vastly different overstrengths the inelastic demands may be concentrated (and amplified) in a small number of braces.

4b. V- and Inverted V-Braced Frames

V-braced and inverted V-braced (chevron) frames exhibit a special problem that sets them apart from other configurations. The expected behavior of SCBF is that upon continued lateral displacement as the brace in compression buckles, its force drops while that in the brace in tension continues to increase up to the point of yielding. In order for this to occur in these frames, an unbalanced vertical force must be resisted by the intersected beam, as well as its connections and supporting members.

The adverse effect of this unbalanced load 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 (Khatib et al., 1988), or by the use of zipper columns.

A two-story X-braced system and a zipper column system are illustrated in Figure C-F2.4. Two-story X- and zipper-braced frames can be designed with post-elastic behavior consistent with the expected behavior of V-braced SCBF. These configurations can also capture the increase in post-elastic axial loads on beams at other levels. It is possible to design two-story X-braced and zipper frames with post-elastic behavior that is superior to the expected behavior of V-braced SCBF by proportioning elements to discourage single-story mechanisms (Khatib et al., 1988). For more information on these configurations, see Khatib et al. (1988), Yang et al. (2008), and Tremblay and Tirca (2003).

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.

Adequate lateral bracing at the brace-to-beam intersection is necessary in order to prevent adverse effects of possible lateral-torsional buckling of the beam. The stability of this connection is influenced by the flexural and axial forces in the beam, as well as by any torsion imposed by brace buckling or the post-buckling residual

out-of-straightness of a brace. The bracing requirements in the *Specification* were judged to be insufficient to ensure the torsional stability of this connection. Therefore a requirement based on the moment due to the flexural strength of the beam is imposed.

4c. K-Braced Frames

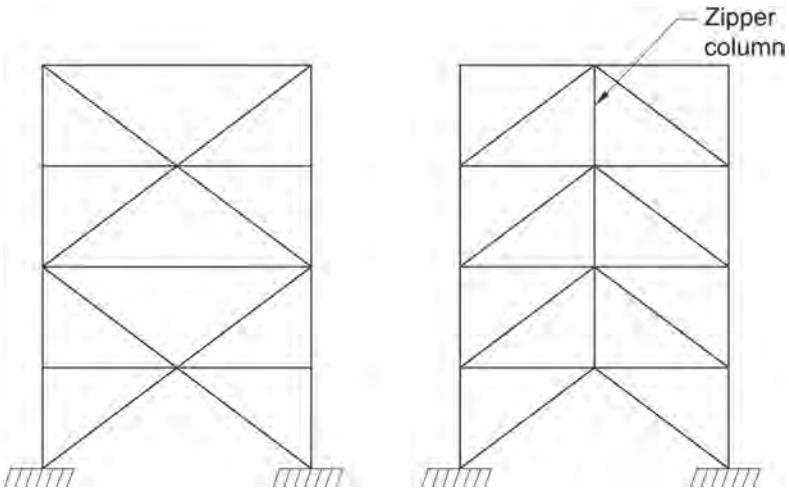
K-bracing is generally not considered desirable in concentrically braced frames and is prohibited entirely for SCBF because it is considered undesirable to have columns that are subjected to unbalanced lateral forces from the braces, as these forces may contribute to column failures.

4d. Tension-Only Frames

SCBF provisions have not been developed for use with braces that only act in tension. Thus tension-only braced frames are not allowed for SCBF. (Tension-only bracing is allowed for OCBF).

4e. Multi-Tiered Braced Frames

Multi-tiered braced frames (MTBF) are braced frames with two or more tiers of bracing, or bracing panels between horizontal diaphragm levels or locations of out-of-plane support. MTBF are common in tall single-story building structures when it is not practical to use single bracing members spanning from roof to foundation levels. As shown in Figure C-F2.5, they can be built using various bracing configurations and have more than one bay. In industrial applications, braced frames used to



(a) Two story X-braced frame *(b) "Zipper column" with inverted V-bracing*

Fig. C-F2.4. Types of braced frames.

longitudinally brace crane runways or trussed legs supporting equipment, such as conveyors, form MTBF. They are also used in multi-story buildings with tall story heights such as stadia or concert halls. MTBF columns are typically I-shaped members oriented such that out-of-plane buckling is about strong axis and in-plane weak axis buckling occurs over a reduced length. Along braced lines, gravity columns can be horizontally tied at every strut level to benefit from the shorter in-plane buckling length, as is often seen along exterior walls.

Contrary to conventional braced frames in multi-story applications, there are no floor diaphragms to laterally brace the columns out of the plane of the frame at every tier

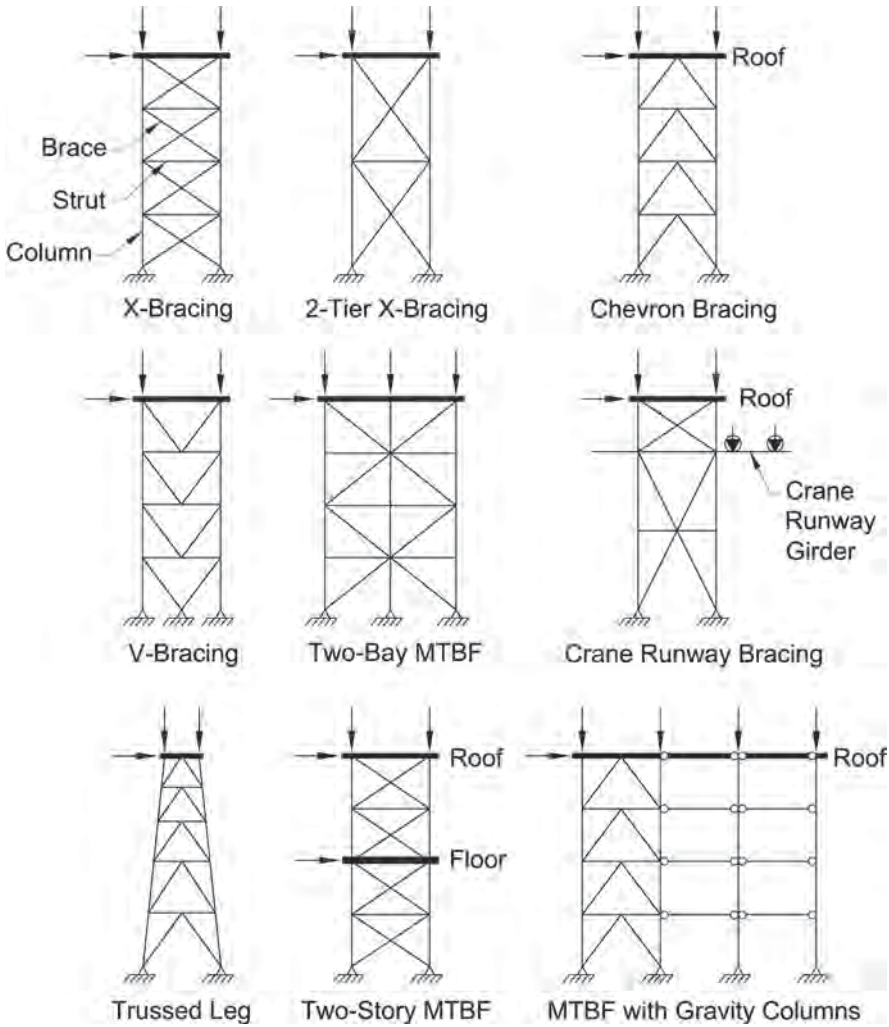


Fig. C-F2.5. Typical MTBF configurations.

level where braces intersect with the columns. Forces inducing out-of-plane deformations of the columns during a seismic event may affect their out-of-plane stability and must be considered in design. These include out-of-plane forces resulting from imperfections at the location of points of intersection of members carrying axial loads or from out-of-plane buckling of the braces. Such effects can affect more slender columns not subjected to other lateral loads, which is the case for columns of interior braced frames not subjected to transverse wind loading. Struts in V- or inverted V-bracing are typically laterally unbraced and must be proportioned to maintain their out-of-plane stability when subjected to twisting arising from brace buckling. The requirement of Section F2.5a that struts satisfy the requirements for moderately ductile members may make V-type or inverted V-type configurations impractical.

Inelastic response of MTBF also results in additional in-plane demands that may endanger the frame stability. In particular, unbalanced horizontal loads develop at brace-to-column intersecting points after buckling of the compression braces, which could result in significant in-plane bending moments in the columns. Brace yielding and buckling in MTBF tend to develop progressively along the frame height, which can lead to nonuniform drifts in the bracing panels and, thereby, additional in-plane flexural demands on the columns. Unbalanced horizontal brace forces can be effectively resisted by introducing horizontal struts at tier levels; however, bending moments from nonuniform brace yielding must be resisted by the columns. Axial compression combined with in-plane and out-of-plane bending can lead to column flexural-torsional buckling due to initial imperfections and inelasticity effects. Columns must also have minimum in-plane flexural stiffness to prevent excessive drifts that could lead to premature brace fracture. Contrary to other bracing systems, column bending demands must therefore be explicitly considered in design to achieve satisfactory seismic performance and new requirements have been introduced in the Provisions to assess and properly address this demand and other aspects specific to MTBF.

In each braced frame, the story shear in every tier must be resisted by braces acting in tension and compression to ensure that the frame will exhibit a symmetrical inelastic response dominated by braces acting in tension in each direction. Horizontal struts are required at all tier levels to resist the unbalanced horizontal loads induced at brace-to-column connection points after brace buckling. In absence of a strut, the unbalanced horizontal force would impose significant in-plane flexural demand on the column that could lead to column buckling, as is the case in K-braced frames (see Figure C-F2.6 for illustrations of this behavior). After brace buckling, the struts ensure that the lateral loads can be transferred over the entire story height mainly through truss action involving tension-acting braces and struts in compression. Maximum compression in struts is therefore determined from analysis, as discussed in Section F2.3 case (b) when braces in tension are assumed to resist forces corresponding to their expected strength and braces in compression are assumed to resist their expected post-buckling strength.

Upon buckling and subsequent straightening when reloaded in tension, bracing members impose bending moments on their connections and other members framing into

the connections. When the braces are detailed to buckle out-of-plane, out-of-plane and torsional moments are imposed on the columns. These two moments are respectively the vertical and horizontal components of a moment equal to the expected flexural resistance of the brace (see out-of-plane brace buckling in Figure C-F2.7). If brace connections are detailed to accommodate ductile inelastic rotations, this moment can be limited to $1.1R_y$ times the connection nominal flexural resistance. It is noted that braces buckling out-of-plane do not induce out-of-plane transverse forces at brace-to-column connections, and the moments at work points can be taken as the moments corresponding to the flexural resistance of the braces or brace connections, depending

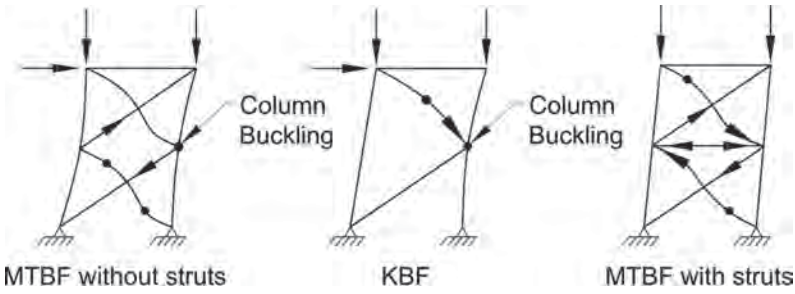


Fig. C-F2.6. Role of strut members in MT-BRBF.

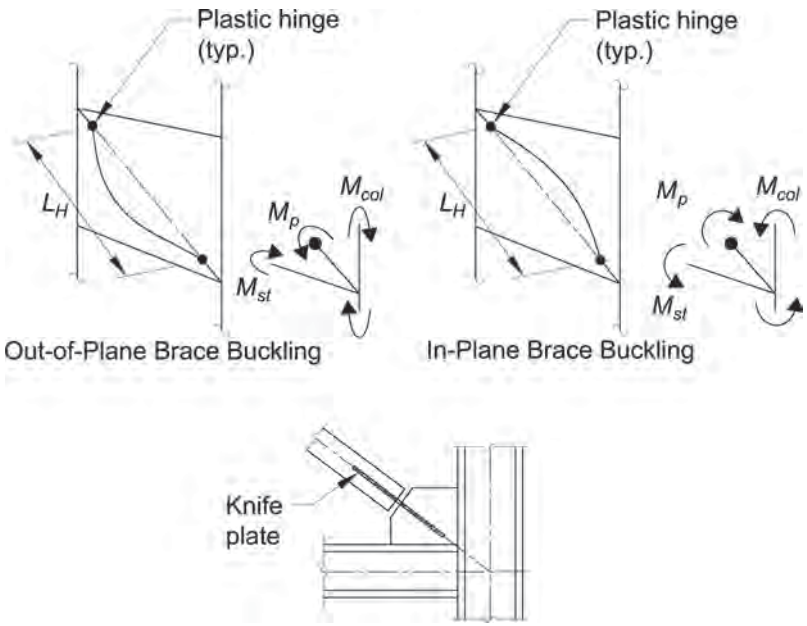


Fig. C-F2.7. Forces induced by buckling of the braces.

which one governs. Out-of-plane moments must be resisted by the columns whereas torsional moments would typically be resisted by the struts bending in the horizontal plane (struts are used to restrain columns against torsion—see Figure C-F2.7).

When braces and their connections are detailed for in-plane buckling, in-plane moments are imposed on the columns and struts as a result of brace buckling (see in-plane brace buckling in Figure C-F2.7). These moments can be resisted by the columns or the struts, or a combination thereof, depending on the connection details and relative member stiffness. Connections of braces buckling in-plane are generally detailed such that plastic hinging forms in the braces next to the connections. In this case, the moment demand can be high and impact the columns as it corresponds to the brace expected flexural strength. That demand can be significantly reduced by adopting a knife plate connection detail in which inelastic rotation occurs through plate bending, or by providing an unstiffened gusset connection to the web of wide-flange columns such that the flexibility of the column web accommodates the rotations associated with brace buckling. As for out-of-plane brace buckling, moments at column centerlines can be taken equal to those developing in the braces or brace connections.

In V- and inverted V- (chevron) bracing, the struts also act as beams resisting the unbalanced vertical loads arising from the braces after brace buckling. In the absence of floor diaphragms at tier levels, lateral stability of the beams can be achieved by providing beams with sufficient strength and stiffness against twisting, as recommended for V- and inverted V- bracing. As stated previously, providing beams with sufficient strength and stiffness that also meet the requirements for moderately ductile members may not be practical for certain configurations. In the case of braces buckling out-of-plane, additional torsion is induced that must be considered in design.

Bracing panels in multi-tiered braced frames act in series between the foundation and the roof levels, or between stories in multi-story applications. Recent research (Imanpour et al., 2013) has shown that brace buckling and yielding typically develops progressively along the frame height which results in nonuniform tier drifts inducing in-plane bending moments in the columns. This behavior is illustrated in Figure C-F2.8 for a uniform 4-tiered chevron braced frame. As shown, bending is more pronounced in a tier where the brace tension yielding has developed, causing relatively larger drifts and degradation of the compression brace strength in the post-buckling range, while brace tension yielding has not been triggered yet in an adjacent tier. During an earthquake, this scenario occurs in sequence, starting from the weakest tier and propagating in the frame until brace tension yielding has developed in all tiers. The combination of axial compression and bending in the columns may cause in-plane flexural instability of the columns before a complete plastic mechanism is reached where all braces have yielded in tension and attained their post-buckling strength in compression. This behavior is more pronounced in frames with different tier heights or with variability in strength between tiers. Similar response is, however, observed in frames with identical tiers due to unavoidable variability in member strength properties, imperfections and boundary conditions between tiers.

Section F2.3 now includes a third analysis case to assess the flexural demand imposed on MT-SCBF columns as brace inelastic response progresses along the frame height. For simple frames, column moments and axial loads can be determined by manual calculations, as is done for Section F2.3 analysis cases (a) and (b). For more complex MT-SCBF configurations, nonlinear static (pushover) analysis can be used to capture the expected sequence of brace yielding and resulting member forces. In both cases, the analysis is performed until a full brace buckling and yielding mechanism has been reached, corresponding to analysis case (b). Alternatively, column forces can be determined from nonlinear response history analysis. The latter would be more appropriate for taller frames with a large number of tiers as brace yielding may only develop over a fraction of the frame height, resulting in reduced flexural demand.

Manual calculation is illustrated herein. If nonlinear analysis (static or dynamic) is used, it must be performed in accordance with Chapter C. Guidance on modelling and analysis can be found in Imanpour et al. (2016a, 2016b). The model must account for brace yielding and buckling responses. In static nonlinear analysis, the rate of brace compressive strength degradation must be accentuated to reproduce the conditions expected under cyclic seismic demand (Imanpour and Tremblay, 2014). In nonlinear analysis of uniform frames, brace strengths in one tier must be intentionally reduced by a small amount (5% may be appropriate) to reproduce the initiation and subsequent progression of brace buckling and yielding expected in actual frames. Scenarios where brace yielding initiates in the bottom or top tier generally lead to more critical conditions for the columns, as described below.

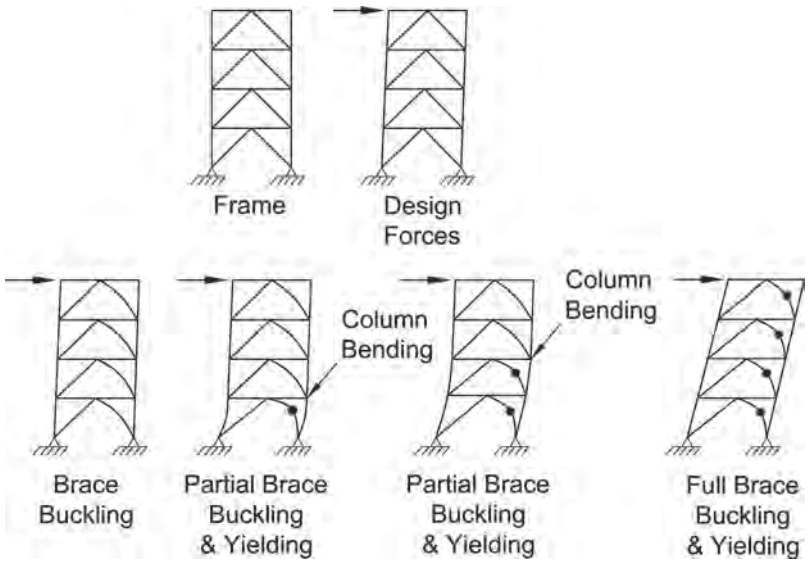


Fig. C-F2.8. Progression of brace buckling and yielding in MT-SCBF.

In well-proportioned frames subjected to increasing lateral loads, all compression braces buckle nearly simultaneously, followed by brace yielding occurring in the tension brace that has the highest stress ratio as the load is increased further and brace force redistribution occurs after brace buckling. In Figure C-F2.9, brace yielding initiates in Tier 1 (the lowest tier). As the brace stretches, drift increases in this tier which causes bending of the columns. The strength of the compression brace reduces in Tier 1 and the total story shear carried by the brace reduces. Horizontal equilibrium is maintained by shears developing in the columns as they bend. Column flexure reaches a maximum when the tension brace in Tier 2 reaches its expected yield strength, T_{exp}, while the compression brace strength in Tier 1 has reduced to its expected post-buckling strength, C'_{exp}. In Tier 2, the compression brace still carries a load close to its expected buckling strength, C_{exp}, and a conservative estimate of the unbalanced brace story shear, ΔV_{br}, is:

$$\Delta V_{br} = (T_{exp} + C_{exp})_2 \cos\theta_2 - (T_{exp} + C'_{exp})_1 \cos\theta_1 \qquad \text{(C-F2-1)}$$

The brace force scenarios in Tiers 1 and 2, respectively, correspond to those described in Section F2.3 analysis cases (b) and (a). A numerical example for a 2-story inverted V-bracing configuration is shown in Figure C-F2.10. The diagram shows the frame resisting the difference between brace story shear strengths in Tiers 1 and 2 (400 kips – 300 kips = 100 kips) when brace yielding initiates in the second level. In this case, the total frame shear is less than the capacity of the braces in the strongest tier because the column shear is in the opposite direction. As shown, the unbalanced brace story shear is resisted equally by the two columns and moments can be readily obtained from statics. Axial loads induced by the braces can also be easily determined, including the effect of vertical unbalanced brace load at the roof level.

In multi-bay braced frames, unbalanced story shears are resisted by all columns. Gravity columns along braced lines are often tied to MT-SCBF by means of horizontal

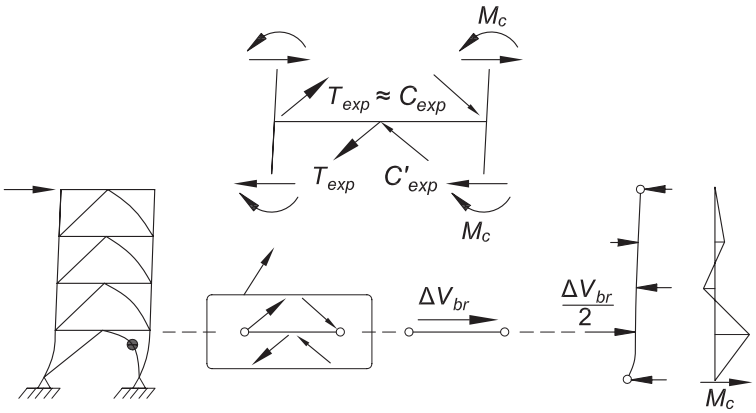


Fig. C-F2.9. Unbalanced brace story shear strengths in MT-SCBF.

strut members at tier levels such that their in-plane buckling length is reduced. In this case, a portion of the unbalanced story shear is resisted by the gravity columns, reducing the demand on the braced frame columns. The flexural demand is distributed between braced frame and gravity columns as a function of their relative flexural stiffness properties (Imanpour et al., 2015). Connecting struts must then be designed to carry the axial loads arising from this distribution and the gravity columns must resist the axial compression plus their share of the flexural demand.

In frames with three or more tiers, the progression of brace yielding and buckling along the height results in a series of scenarios inducing various bending moment demands. This behavior is illustrated in Figure C-F2.11 for a uniform frame for the case where brace yielding initiates in the bottom tier. In the figure, Cases 1 and 2 correspond to Section F2.3 analysis cases (a) and (b), respectively. Moments can be estimated by neglecting column continuity at the top end of the tier in which brace tension yielding is triggered (case 1). In this simplified model, the column behaves as a simply supported element resisting its share of the unbalanced brace story shear at the level between tiers where analysis cases 1 and 2 apply. In this particular case, the unbalanced brace story shear is zero between two consecutive tiers where case 2 exists. In frames with nonuniform brace strengths, additional forces would need to be considered at these levels.

In design, not all scenarios need to be considered as only one or a few cases will induce critical combinations of axial load and in-plane moment for the columns. For uniform frames, maximum in-plane moments and axial loads may occur in the lowest tier when brace yielding is triggered in that tier after propagation of inelastic

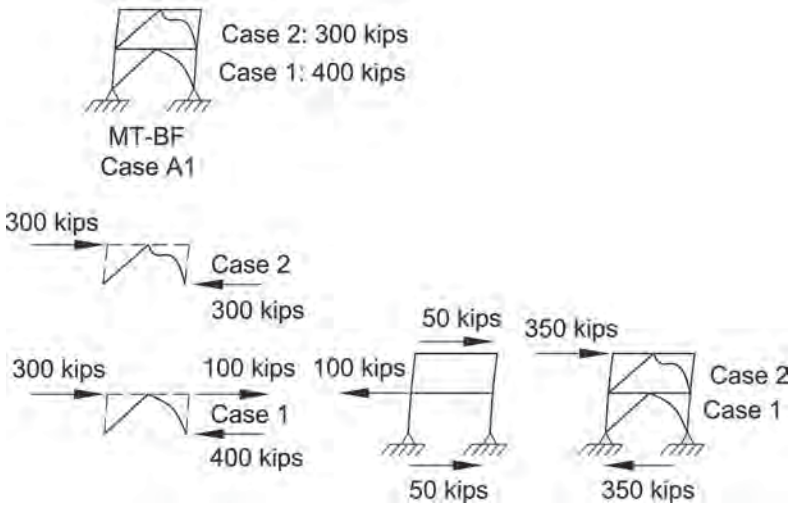


Fig. C-F2.10. In-plane flexural demand for the columns of a two-story inverted V-bracing configuration (brace yielding in level 2).

response from the top (Figure C-F2.12). Note that out-of-plane moments arising from brace buckling or imperfections must also be considered when verifying the columns, which may affect the critical scenario.

Frames with nonuniform geometries with different brace sizes may result in more complex response, as shown in Figure C-F2.13. Propagation of brace yielding will

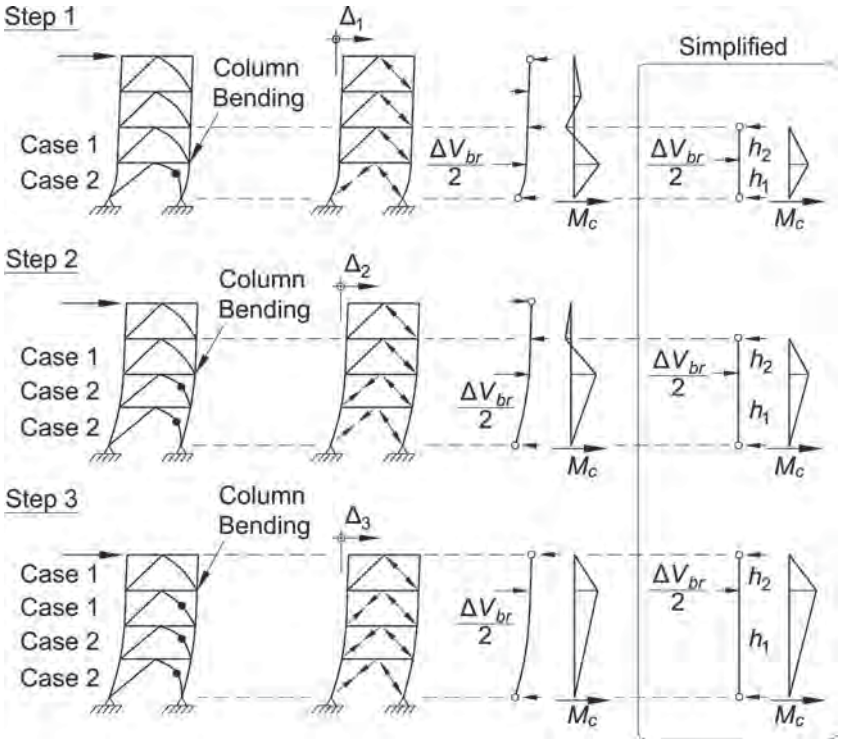


Fig. C-F2.11. Column in-plane flexural demand for a uniform MT-SCBF.

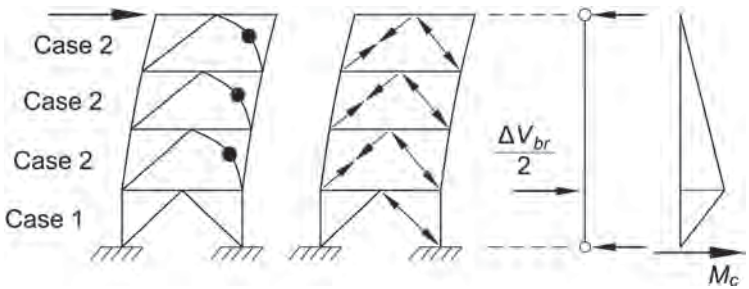


Fig. C-F2.12. Progression of brace yielding from the frame top.

depend on the relative brace story shear resistance and nonlinear analysis appropriate for this type of frame. Alternatively, column flexural demands can be determined using a suite of linear static analyses with a structural model in which the buckled and yielded braces are removed and replaced by horizontal forces corresponding to the horizontal components of their expected strengths. In each analysis, the horizontal load applied at the top of the frame is adjusted such that the tension brace in the tier where the conditions of analysis case a apply. The procedure is illustrated in Figure C-F2.13. Brace yielding initiates in Tier 2 and subsequently develops in Tiers 3 and 1. In the figure, horizontal forces V'_{exp} correspond to brace story shears determined with the brace expected post-buckling compressive strengths C'_{exp}. Column axial loads are determined by summing the vertical components of the brace strengths.

In-plane bending moments in columns heavily depends on the difference between brace compressive strengths, C_{exp} and C'_{exp}, at different tiers. Nonlinear response analysis (Imanpour et al., 2016a, 2016b) have shown that less severe conditions typically exist under actual ground motions, the compression brace forces in the yielded tier being generally higher than C'_{exp} whereas the compression brace in the tier where brace yielding is triggered has lost part of its compressive strength, which results in smaller values of ΔV_{br} compared to the value predicted by Equation

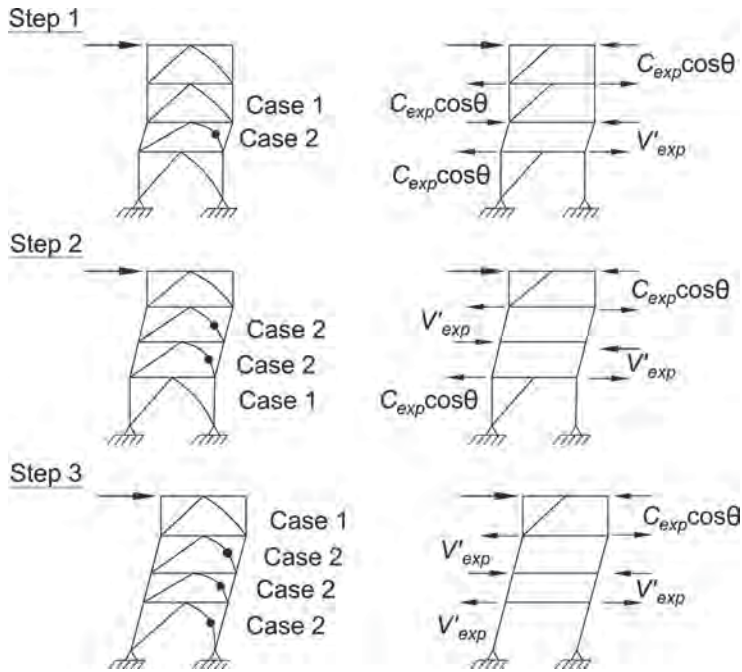


Fig. C-F2.13 Column in-plane flexural demand from linear static analysis for a nonuniform MT-SCBF.

C-F2-1. The conservatism of the approach is deemed to compensate for variability in brace strengths due to uncertainties in material yield strength and brace boundary conditions. Calculations should thus be performed using values of C_{exp} and C'_{exp} as specified in the Provisions. When brace buckling response is explicitly modelled in nonlinear dynamic analysis, material variability should also be considered by varying the brace yield strength in tiers where maximum bending moments are obtained. Greater demand is expected when brace sizes or brace inclinations vary along the frame height. Greater demand is also observed when the brace sizes are kept the same even when tier heights are varied. Attention must be paid when configuring the frame geometry and brace sizes to minimize the demand.

Numerical simulations indicate that in some cases, this in-plane column yielding reduces the out-of-plane flexural buckling strength of the column (Stoakes and Fahnestock, 2013). This reduction is most pronounced when the in-plane column yielding occurs near mid-height of the column, which is the situation in two-tiered frames. However, the deleterious effects of in-plane column yielding on out-of-plane flexural buckling can be mitigated by providing torsional bracing which satisfies the minimum stiffness and strength requirements developed by Helwig and Yura (1999), at every tier level. Torsional bracing of columns can be provided by mobilizing the out-of-plane flexural stiffness of tier-level struts. I-shaped struts oriented such that their webs are in the horizontal plane represent an effective means of providing torsional stiffness and strength through strong-axis bending. Struts must also resist in-plane torsional moments imposed by brace out-of-plane buckling. Strut-to-column connections must be detailed to develop the required strength and stiffness.

Axial forces acting in braces and struts may induce out-of-plane horizontal loading to the columns due to imperfections in the connecting points resulting from column out-of-plane out-of-straightness. Effects of these forces are amplified due to second-order and inelasticity effects resulting from the presence of axial compression load in the columns. Imperfection effects are present under any load combination that includes lateral loads, including seismic loads. They can be evaluated through the direct analysis method with explicit consideration of geometrical imperfections, as described in *Specification* Chapter C. Alternatively, horizontal notional loads are given in the Provisions that can be applied to account for geometrical imperfection and inelasticity effects. When applying these loads, second-order effects must still be considered using either the direct second-order analysis method or the approximate second-order analysis method where moments are amplified by the B_1 factor, as described in *Specification* Appendix 8. In addition, a maximum value is specified in the Provisions for the amplification factor B_1 to prevent from using columns exhibiting limited out-of-plane stiffness.

Column shear distortion is the sum of the overall frame drift and the distortion due to column bending. It is limited to 2%, which is considered reasonable for buckling braces.

5. Members

5a. Basic Requirements

Traditionally, braces have shown little or no ductility after overall (member) buckling, which produces a plastic hinge at the brace midpoint. At this plastic hinge, local buckling can cause large strains, leading to fracture at low drifts. It has been found that braces with compact elements are capable of achieving significantly more ductility by forestalling local buckling (Goel, 1992b; Hassan and Goel, 1991; Tang and Goel, 1989). Widthto-thickness ratios of compression elements in bracing members have been set 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 fracture due to local buckling is especially prevalent in rectangular HSS with widthto-thickness ratios larger than the prescribed limits (Hassan and Goel, 1991; Tang and Goel, 1989). Even for square HSS braces designed to meet the seismic width-to-thickness ratios of these Provisions, local buckling leading to fracture may represent a limitation on the performance (Yang and Mahin, 2005).

The same limitations apply to columns in SCBF, as their flexural strength and rotation capacity has been shown to be a significant contributor to the stability of SCBF (Tremblay, 2001, 2003). It has also been demonstrated that SCBF can be subject to significant story drift (Sabelli et al., 2003), requiring columns to undergo inelastic rotation.

Enhanced ductility and fracture life of rectangular HSS bracing members can be achieved in a variety of ways. The HSS walls can be stiffened by using longitudinal stiffeners, such as rib plates or small angle sections in a hat configuration (Liu and Goel, 1987). Use of plain concrete infill has been found to be quite effective in reducing the severity of local buckling in the post-buckling range of the member (Liu and Goel, 1988; Lee and Goel, 1987). Based on their test results, Goel and Lee (1992) formulated an empirical equation to determine the effective width-to-thickness ratio of concrete-filled rectangular HSS bracing members. The effective width-to-thickness ratio can be calculated by multiplying the actual width-to-thickness ratio by a factor, $[(0.0082KL/r) + 0.264]$, for KL/r between 35 and 90, where KL/r is the effective slenderness ratio of the member. The purpose of concrete infill as described herein is to inhibit the detrimental effects of local buckling of the HSS walls. Use of concrete to achieve composite action of braces is covered in Section H2.5b.

As an alternative to using a single large HSS, consideration may be given to using double smaller HSS sections stitched together and connected at the ends to a single gusset plate (or cross shape if needed) in much the same way as double angle or channel sections are used in a back-to-back configuration (Lee and Goel, 1990). Such double HSS sections offer a number of advantages, including: reduced fit up problems, smaller width-to-thickness ratio for the same overall width of the section, promotion of in-plane buckling in most cases eliminating the problem of out-of-plane bending of gusset plates, greater energy dissipation as three plastic hinges form in the

member, and greater strength because of the effective length factor, K, being close to 0.5 as opposed to K=1.0 when out-of-plane buckling occurs in a single HSS and single gusset plate member.

5b. Diagonal Braces

The required strength of bracing members with respect to the limit state of tensile rupture on the net section is the expected brace strength. It should be noted that some, if not all, steel materials commonly used for braces have expected yield strengths significantly higher than their specified minimum yield strengths; some have expected yield strengths almost as high as their expected tensile strength. For such cases, no significant reduction of the brace section is permissible and connections may require local reinforcement of the brace section. This is the case for knife-plate connections between gusset plates and ASTM A53 or A500 braces (e.g., pipe, square, rectangular or round HSS braces), where the over-slot of the brace required for erection leaves a reduced section. If this section is left unreinforced, net section rupture will be the governing limit state and brace ductility may be significantly reduced (Korol, 1996; Cheng et al., 1998). Reinforcement may be provided in the form of steel plates welded to the tube, increasing the effective area at the reduced brace section (Yang and Mahin, 2005). Braces with two continuous welds to the gusset wrapped around its edge (instead of the more typical detail with four welds stopping short of the gusset edge) performed adequately in the tests by Cheng. However, this practice may be difficult to implement in field conditions; it also creates a potential stress riser that may lead to crack initiation.

Where there is no reduction in the section, or where the section is reinforced so that the effective net area is at least as great as the brace gross area, this requirement does not apply. The purpose of the requirement is to prevent tensile rupture on the net section prior to significant ductility; having no reduction in the section is deemed sufficient to ensure this behavior. Reinforcement, if present, should be connected to the brace in a manner that is consistent with the assumed state of stress in the design. It is recommended that the connection of the reinforcement to the brace be designed for the strength of the reinforcement on either side of the reduced section.

The slenderness (L_c/r) limit is 200 for braces in SCBF. Research has shown that frames with slender braces designed for compression strength behave well due to the overstrength inherent in their tension capacity. Tremblay (2000), Tang and Goel (1989) and Goel and Lee (1992) have found that the post-buckling cyclic fracture life of bracing members generally increases with an increase in slenderness ratio. An upper limit is provided to preclude dynamic effects associated with extremely slender braces.

Closer spacing of stitches and higher stitch strength requirements are specified for builtup bracing members in SCBF (Aslani and Goel, 1991; Xu and Goel, 1990) than those required for typical built-up members. This is especially critical for double-angle and double-channel braces that impose large shear forces on the stitches upon buckling. These are intended to restrict individual element bending between the stitch

points and consequent premature fracture of bracing members. Typical spacing following the requirements of the *Specification* is permitted when buckling does not cause shear in the stitches. Bolted stitches are not permitted within the middle 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 a plastic hinge in the postbuckling range. Studies also showed that placement of double angles in a toe-to-toe configuration reduces bending strains and local buckling (Aslani and Goel, 1991).

5c. Protected Zones

Welded or shot-in attachments in areas of inelastic strain may lead to fracture. Such areas in SCBF include gusset plates and expected plastic-hinge regions in the brace.

Figures C-F2.14 and C-F2.15 show the protected zone of an inverted V- and an X-braced frame, respectively. Note that for the X-braced frame, the half-length of the brace is used and a plastic hinge is anticipated at any of the brace quarter points.

6. Connections

6a. Demand Critical Welds

Groove welds at column splices are designated as demand critical for several reasons. First, although the consequences of a brittle failure at a column splice are not clearly understood, it is believed that such a failure may endanger the safety of the frame. Second, the actual forces that will occur at a column splice during an earthquake are very difficult to predict. The locations of points of inflection in the columns during an earthquake are constantly moving, are ground motion dependent, and cannot be reliably predicted from analysis. Thus, even though analysis of the frame under code specified load combinations (with the overstrength seismic load) may show that no tension will occur at a weld, such an analysis cannot be considered reliable for the prediction of these demands. Because of the critical nature of column splices and the

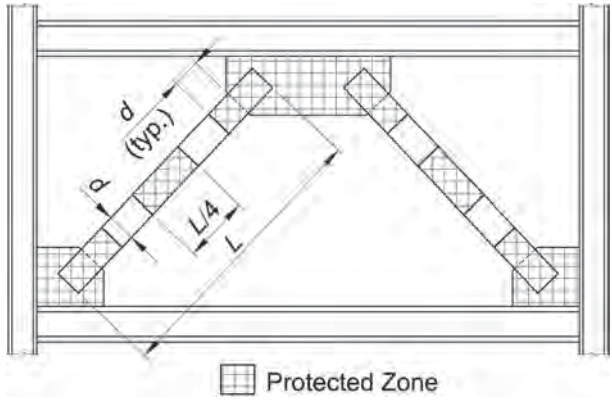


Fig. C-F2.14. Protected zone of inverted V-braced frame.

inability to accurately predict the forces that will occur at these locations, it is the intent of the Provisions that column splices be one of the strongest elements of the frame and be designed in a conservative manner. Accordingly, in order to provide a high degree of protection against brittle failure at column splice groove welds, the use of demand critical welds is specified. PJP groove welds are included in this requirement, because the unfused portion on the weld makes PJP welds particularly prone to brittle failure.

6b. Beam-to-Column Connections

Braced frames are likely to be subject to significant inelastic drift. Thus their connections will undergo significant rotation. Connections with gusset plates can be vulnerable to rupture if they are not designed to accommodate this rotation. Recent testing (Uriz and Mahin, 2004) has indicated that designs that do not properly account for the stiffness and distribution of forces in braced frame connections may be subject to undesirable performance.

The provision allows the engineer to select from three options. The first is a simple connection (for which the required rotation is defined as 0.025 rad). The connections presented in *Manual* Part 10 (AISC, 2011) are capable of accommodating rotations of 0.03 rad and therefore meet the requirement for a simple connection. However, it is important to recognize that in many configurations, the gusset and beam behave rigidly

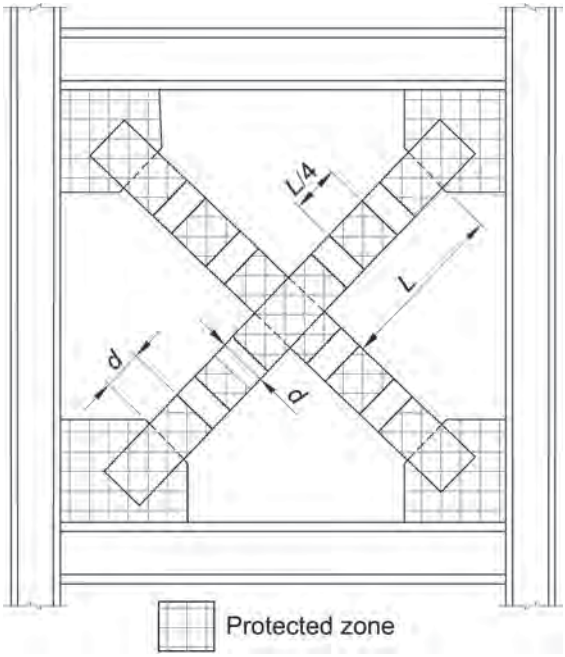


Fig. C-F2.15. Protected zone of X-braced frame.

relative to one another such that the beam-to-column connection and the gusset-to-column connection should be treated similarly with respect to deformation demands to achieve rotational ductility. An example of this would be a configuration tested at the University of Illinois (Stoakes and Fahnestock, 2010) that effectively allowed rotation between the beam and column, which is illustrated in Figure C-F2.16. In this case it is important the gusset-to-column connection have deformation characteristics similar to the beam-to-column connection, which is achieved by use of similar double angle connections. (Note that the connection illustrated does not indicate the typical SCBF hinge zone discussed in the commentary for Section F2.6c.) A similar configuration using bolted-bolted double angles to connect the gusset plate to the main members and the beam to the column was tested by McManus et al. (2013) and is shown in Figure C-F2.17. The testing performed by McManus et al. also suggested that unstiffened connections of the beam and gusset to the column web allow for rotation of the beam and gusset relative to the column through flexing of the column web, thereby reducing undesirable "pinching" forces in the gusset, beam and column. The result is a reduced susceptibility to damage in structural members resulting from large frame drifts.

Fahnestock et al. (2006) also tested a connection with rotation capacity outside the gusset plate; this connection is discussed in the commentary for Section F4.6c. A similar concept was proposed by Thornton and Muir (2008) and is shown in Figure C-F2.18. These configurations also reduce "pinching" forces by allowing the rotation to occur outside the beam-to-column and gusset-to-column connection.

The second option is a fully restrained moment connection for which the maximum moment can be determined from the expected strength of the connecting beam or column.

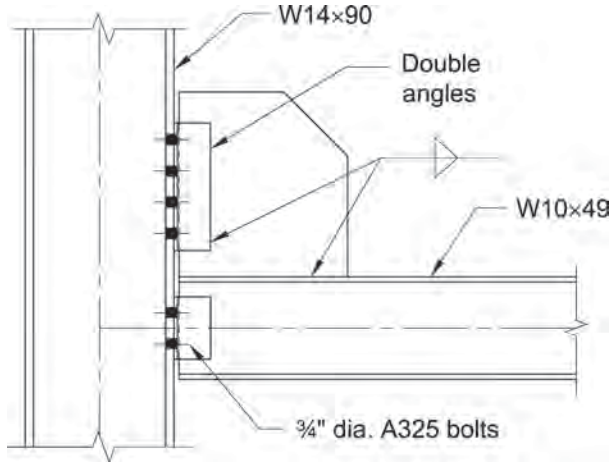
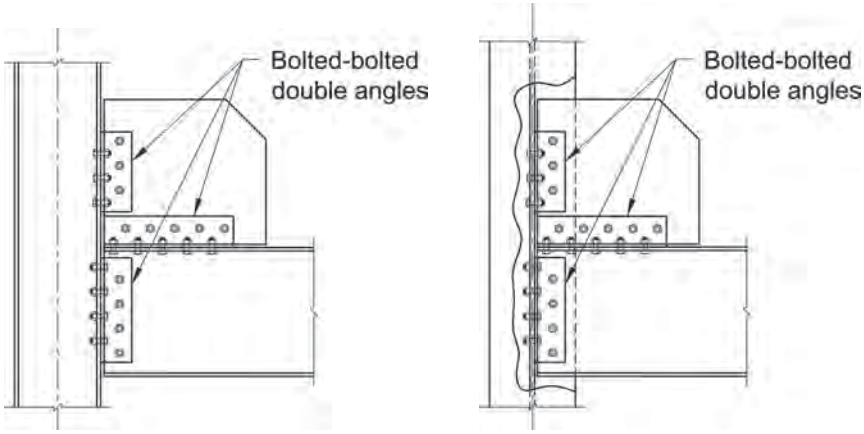


Fig. C-F2.16. Beam-to-column connection that allows rotation (Stoakes and Fahnestock, 2010).



(a) All-bolted unstiffened gusset connection to column flange

(b) All-bolted gusset connection to column web

Fig. C-F2.17. All-bolted beam-to-column connection that allows rotation (McManus et al., 2013).

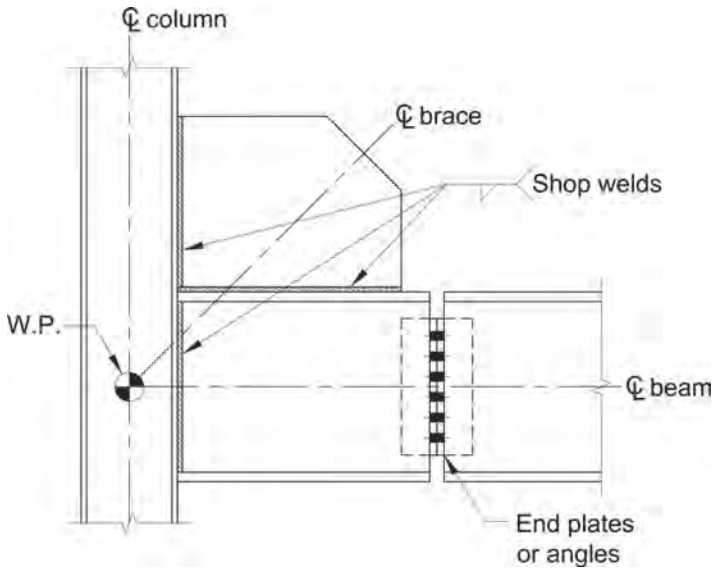


Fig. C-F2.18. Beam-to-column connection that allows rotation (Thornton and Muir, 2008).

The third option has been added in this edition of the Provisions, and is also a fully restrained moment connection. Rather than give a required strength of the connection, this option refers to the prescriptive requirements for one of the OMF connection alternatives.

6c. Brace Connections

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-Asl 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 loads 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.

Certain references suggest limiting the free edge length of gusset plates, including SCBF brace-to-beam connection design examples in the *Seismic Design Manual*, (AISC, 2006), and other references (Astaneh-Asl et al., 2006; ICC, 2006). However, the committee has reviewed the testing cited and has concluded that such edge stiffeners do not offer any advantages in gusset plate behavior. There is therefore no limitation on edge dimensions in these provisions.

1. Required Tensile Strength

Braces in SCBF are required to have gross section tensile yielding as their governing limit state so that they will yield in a ductile manner. Local connection failure modes such as block shear rupture must be precluded. Therefore, the calculations for these failure modes must use the maximum load that the brace can develop.

The minimum of two criteria, the expected axial tensile strength of the bracing member and the maximum force that could be developed by the overall system, determines the required strength of both the bracing connection and the forces delivered to the beam-to-column connection. This second limit is included in the Provisions for structures where elements other than the tension bracing limit the system strength. Depending on the specific situation(s), there are a number of ways one can determine the maximum force transferred to the connection. They include:

(1) Perform a pushover analysis to determine the forces acting on the connections when the maximum frame capacity, leading to an imminent collapse mechanism, is reached.

(2) Determine how much force can be resisted before causing uplift of a spread footing (note that the foundation design forces are not required to resist more than the code base shear level). This type of relief is not typically applicable to a deep foundation since the determination of when uplift will occur is not easy to determine accurately.

(3) Perform a suite of inelastic time history analyses in accordance with Section C3 and envelop the connection demands.

Calculating the maximum connection force by one of these three methods is not a common practice on design projects. In some cases, such an approach could result in smaller connection demands. But, from a conceptual basis, since the character of the ground motions is not known to any great extent, it is unrealistic to expect that such forces can be accurately calculated. All three approaches rely on an assumed distribution of lateral forces that may not match reality (the third approach is probably the best estimate, but also the most calculation intensive). In most cases, providing the connection with a capacity large enough to yield the member is needed because of the large inelastic demands placed on a structure by a major earthquake.

Bolt slip has been removed as a limit state which must be precluded. The consequences of exceeding this limit state in the maximum credible earthquake are not considered severe if bearing failure and block shear rupture are precluded.

2. Required Compressive Strength

Bracing connections should be designed to withstand the maximum force that the brace can deliver in compression. A factor of 1.1 was applied to the expected brace strength in previous editions in consideration of the use of conservative column curve equations in determining this force. This factor has been removed in the 2016 Provisions because the (1/0.877) factor used to determine the expected brace strength in Section F2.3 adequately bounds the maximum anticipated force the brace can deliver.

3. Accommodation of Brace Buckling

Braces in SCBF are expected to undergo cyclic buckling under severe ground motions, forming plastic hinges at their center and at each end. To prevent fracture resulting from brace rotations, bracing connections must either have sufficient strength to confine inelastic rotation to the bracing member or sufficient ductility to accommodate brace end rotations.

For brace buckling in the plane of the gusset plates, the end connections should be designed to resist the expected compressive strength and the expected flexural strength of the brace as it transitions from pure compression towards a condition dominated by flexure (Astaneh-Asl 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 designed to satisfy Section F2.6c.3(b), 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 minimal restraint plastic rotations. This requires the end of the brace to be held back away from the beam and column so that the gusset can effectively form a plastic hinge as the brace

buckles. Such gussets tend to have larger unbraced lengths and in some cases the required thickness may be governed by the need to preclude the occurrence of plate buckling prior to member buckling.

Astaneh-Asl et al. (1986) recommended providing a linear hinge zone with a length of two times the plate thickness. 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.

This condition is illustrated in Figure C-F2.19 and provides hysteretic behavior as illustrated in Figure C-F2.21. The distance of $2t$ shown in Figure C-F2.19 should be considered the minimum offset distance. In practice, it may be advisable to specify a slightly larger distance (for example, $2t + 1$ in.) on construction documents to provide for erection tolerances. More information on seismic design of gusset plates can be obtained from Astaneh-Asl (1998).

More recently, Roeder recommended an elliptical hinge zone that provides similar rotation capacity and a shorter unbraced length, allowing for thinner gusset plates. Such thinner gusset plates contribute to the overall inelastic drift capacity of the frame (Roeder et al., 2011). An application of this method is shown in the *Seismic Design Manual* (AISC, 2012).

Tsai et al. (2013) provide design recommendations for gussets configured to allow in-plane rotation. Such connections can be used with braces designed to buckle in the plane of the frames. Braces so designed would have in-plane deformations that would need to be accommodated, rather than out-of-plane ones. Figure C-F2.20 shows a gusset designed to allow in-plane rotation.

Alternatively, connections with stiffness in two directions, such as cross gusset plates, can be designed and detailed to satisfy Section F2.6c.3(a). 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).

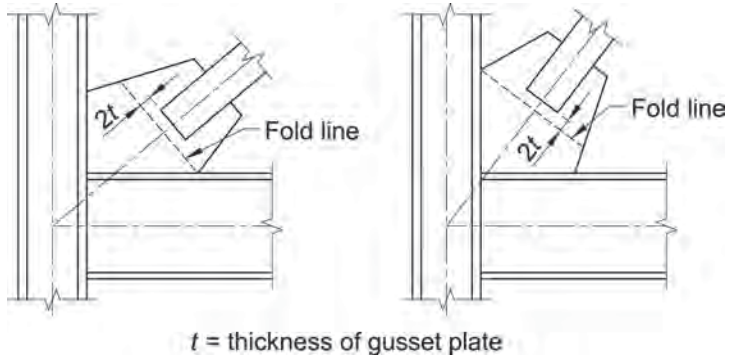


Fig. C-F2.19. Brace-to-gusset plate requirement for buckling out-of-plane bracing system.

Where fixed end 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.

4. Gusset Plates

Where a brace frames to a beam-column joint, the stresses on a corner gusset weld are a result of brace axial forces combined with gusset flexure (as the brace buckles) and frame moments (except where moment releases are provided). Accurate prediction of maximum stresses at large drifts is difficult, and early fracture of the welds has been noted in experiments where the welds are designed using the uniform force method and the expected tensile capacity of the brace (Lehman et al., 2008). To forestall such fracture, welds of gusset plates are required to be somewhat stronger than the plate, allowing local yielding in the plate to protect the weld. While the direction of weld stress may be difficult to assess, proportioning the weld to resist the expected gusset shear strength results in a condition that is likely to preclude weld failure and can be done with minimal calculations.

Out-of-plane brace buckling creates an additional demand that must be addressed when the edge of a corner gusset plate is welded directly to the beam flange or column flange with fillet welds. If the gusset deformation and corresponding

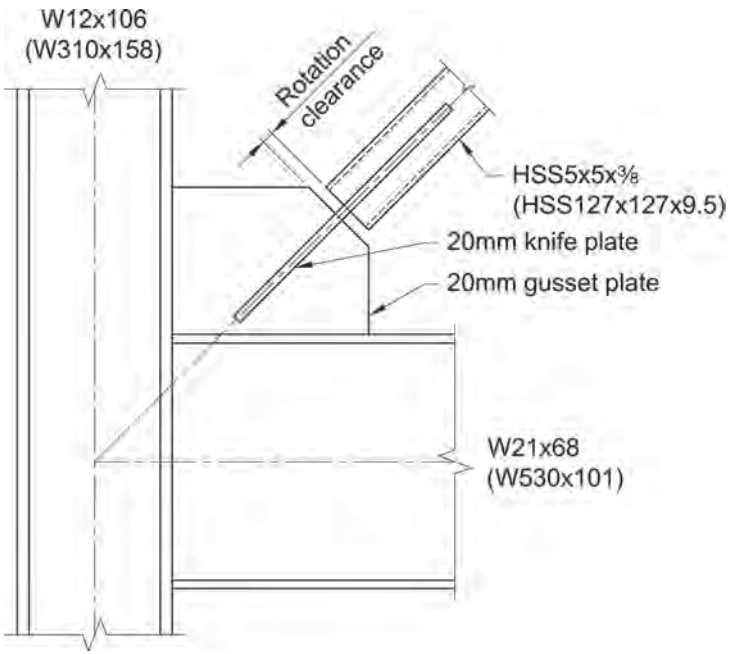


Fig. C-F2.20 Gusset designed for in-plane rotation (Tsai et al., 2013).

weak-axis bending moment at the gusset edge connection are known, the fillet welds can be designed directly for the combination of shear, compression and moment. Otherwise, this demand can be determined by calculating the utilization of the gusset plate edge for the brace force specified in Section F2.6c.2 and calculating the remaining capacity for weak-axis flexure considering a multiaxial yield model. The weld size can then be selected to develop the maximum weak-axis moment occurring in combination with the shear, compression, and strong-axis moment that result on the gusset plate edge from the brace compression force. Carter et al. (2016) developed such a method utilizing a generalized interaction equation recommended by Dowswell (2015).

6d. Column Splices

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. In addition, column splices are required to have sufficient strength to prevent failure under expected post-elastic forces. Analytical studies on SCBF that are not part of a dual system have shown that columns can carry as much as 40% of the story shear (Tang and Goel, 1987; Hassan and Goel, 1991). When columns are common to both SCBF and special moment frames (SMF) in a dual system, their contribution to story shear may be as high as 50%. 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-F2.21.

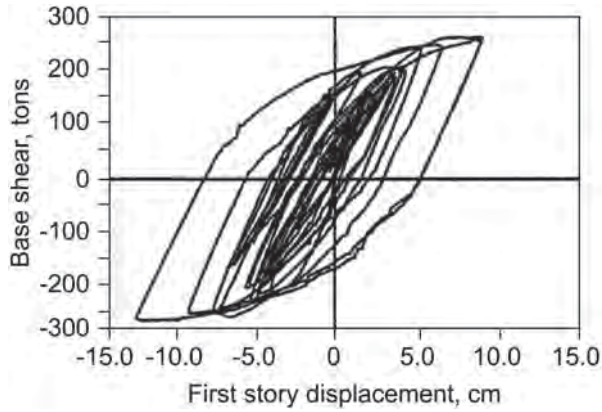


Fig. C-F2.21. Base shear versus story drift of an SCBF.

F3. ECCENTRICALLY BRACED FRAMES (EBF)

1. Scope

Eccentrically braced frames (EBF) are composed of columns, beams and braces. The distinguishing characteristic of an EBF is that at least one end of every brace is connected so that the brace force is transmitted through shear and bending of a short beam segment, called the link, defined by a horizontal eccentricity between the intersection points of the two brace centerlines with the beam centerline (or between the intersection points of the brace and column centerlines with the beam centerline for links adjacent to columns). In contrast with concentrically braced frames, beams in EBF are always subject to high shear and bending forces. Figure C-F3.1 illustrates some examples of eccentrically braced frames and the key components of an EBF: the links, the beam segments outside of the links, the diagonal braces, and the columns.

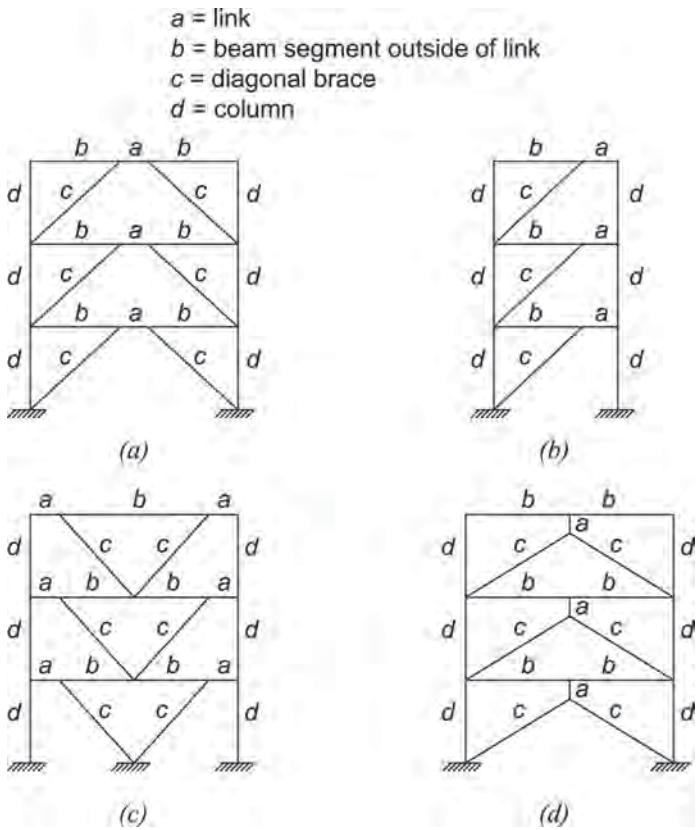


Fig. C-F3.1. Examples of eccentrically braced frames.

These provisions are primarily intended to cover the design of EBF in which the link is a horizontal framing member located between the column and a brace or between two braces. For the inverted Y-braced EBF configuration shown in Figure C-F3.1(d), the link is attached underneath the beam. If this configuration is to be used, lateral bracing should be provided at the intersection of the diagonal braces and the vertical link, unless calculations are provided to justify the design without such bracing.

2. Basis of Design

Research has shown that EBF can provide an elastic stiffness that is comparable to that of 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, provided that the links are not too short (Roeder and Popov, 1978; Libby, 1981; Merovich et al., 1982; Hjelmstad and Popov, 1983; Malley and Popov, 1984; Kasai and Popov, 1986a, 1986b; Ricles and Popov, 1987a, 1987b; Engelhardt and Popov, 1989a, 1989b; Popov et al., 1989). Inelastic action in EBF under seismic loading is restricted primarily to the links. 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 developed by fully yielded and strain-hardened links.

In some bracing arrangements, such as that illustrated in Figure C-F3.2, 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 of the link in the story below, the upper link will deform inelastically and limit the force that can be developed in 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.

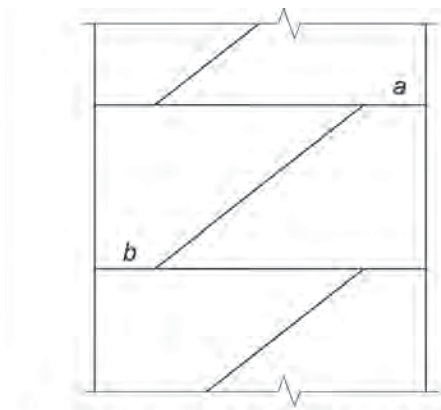


Fig. C-F3.2. EBF— active and inactive links.

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 those illustrated in Figure C-F3.1, is recommended. Further recommendations for the design of EBF are available (Popov et al., 1989).

Columns in EBF are designed following capacity design principles so that the full strength and deformation capacity of the frame can be developed without failure of any individual column and without the formation of a soft story. While this does not represent a severe penalty for low-rise buildings, it is difficult to achieve for taller structures, which may have link beam sizes governed by drift-control considerations. In such cases, it is anticipated that designers will adopt nonlinear analysis techniques as discussed in Chapter C.

Plastic hinge formation in columns should be avoided, because when combined with hinge formation in the links, it can result in the formation of a soft story. The requirements of Sections D1.4a and F3.3 address the required strength for column design.

Additional design requirements have been added to the Provisions to address the special case of box links (those consisting of built-up tubular cross sections). Box links are generally not susceptible to lateral-torsional buckling, and eccentrically braced frames having such links have been shown (Berman and Bruneau, 2007, 2008a, 2008b) to perform in a ductile manner without the need for lateral bracing of the link beam, provided the specified section compactness requirements are met. This can be of benefit when EBF are desirable in locations where such lateral bracing cannot be achieved, such as between two elevator cores, or along the facade of building atriums.

Because of the difficulties in providing adequate lateral bracing of the link beam where diaphragms are not present, EBF are generally considered impractical for multi-tiered braced frame applications, except where box links are used and proportioned such that lateral bracing is not required. Adequate research has not been performed on multi-tiered EBF with box links. Consequently, that system is not addressed in the Provisions.

3. Analysis

The required strength of links is typically determined based on the analysis required by ASCE/SEI 7. The analysis required by this section is used in determining the required strength of braces, beams outside the link and columns, as well as brace connections. The requirements presented here are essentially a reformatting of design rules for these elements into an analysis format.

The intent of the Provisions is to ensure that yielding and energy dissipation in an EBF occur primarily in the links. Consequently, the columns, diagonal braces, and

beam segment outside of the link must be designed to resist the loads developed by the fully yielded and strain-hardened link. That is, the brace and beam should be designed following capacity-design principles to develop the full inelastic capacity of the links. Limited yielding outside of the links, particularly in the beams, is sometimes unavoidable in an EBF. Such yielding is likely not detrimental to the performance of the EBF, as long as the beam and brace have sufficient strength to develop the link's full inelastic strength and deformation capacity.

In most EBF configurations, the diagonal brace and the beam are subject to large axial loads combined with significant bending moments. Consequently, both the diagonal brace and the beam should be designed as beam-columns.

The diagonal brace and beam segment outside of the link must be designed for some reasonable estimate of the maximum forces that can be developed by the fully yielded and strain hardened link. For this purpose, the nominal shear strength of the link, V_n, as defined by Equation F3-1 is increased by two factors. First, the nominal shear strength is increased by R_y to account for the possibility that the link material may have actual yield strength in excess of the specified minimum value. Secondly, the resulting expected shear strength of the link, $R_y V_n$, is further increased to account for strain hardening in the link.

Experiments have shown that links can exhibit a high degree of strain hardening. Recent tests on rolled wide-flange links constructed of ASTM A992/A992M steel (Arce, 2002) showed strength increases due to strain hardening ranging from 1.2 to 1.45, with an average value of about 1.30. Past tests on rolled wide-flange links constructed of ASTM A36/A36M steel have sometimes shown strength increases due to strain hardening in excess of 1.5 (Hjelmstad and Popov, 1983; Engelhardt and Popov, 1989a). Further, recent tests on very large welded built-up wide-flange links for use in major bridge structures have shown strain hardening factors close to 2.0 (McDaniel et al., 2002; Dusicka and Itani, 2002). These sections, however, typically have proportions significantly different from rolled shapes.

Past researchers have generally recommended a factor of 1.5 (Popov and Engelhardt, 1988) to account for expected link strength and its strain hardening in the design of the diagonal brace and beam outside of the link. However, for purposes of designing the diagonal brace, these Provisions have adopted a strength increase due to strain hardening only equal to 1.25. This factor was chosen to be less than 1.5 for a number of reasons, including the use of the R_y factor to account for expected material strength in the link but not in the brace, and the use of resistance factors or safety factors when computing the strength of the brace. Further, this value is close to, but somewhat below, the average measured strain hardening factor for recent tests on rolled wide-flange links of ASTM A992/A992M steel. Designers should recognize that strain hardening in links may sometimes exceed this value, and so a conservative design of the diagonal brace is appropriate. Additionally, if large built-up link sections are used with very thick flanges and very short lengths $\left(e < M_p/V_p\right)$, designers should consider the possibility of strain hardening factors substantially in excess of 1.25 (Richards, 2004).

Based on the preceding, the required strength of the diagonal brace can be taken as the forces developed by the following values of link shear and link end moment:

For $e \le \dfrac{2M_p}{V_p}$

$$\text{Link shear} = 1.25R_yV_p \tag{C-F3-1}$$

$$\text{Link end moment} = \frac{e(1.25R_yV_p)}{2} \tag{C-F3-2}$$

For $e > \dfrac{2M_p}{V_p}$

$$\text{Link shear} = \frac{2(1.25R_yM_p)}{e} \tag{C-F3-3}$$

$$\text{Link end moment} = 1.25R_yM_p \tag{C-F3-4}$$

The preceding equations assume link end moments will equalize as the link yields and deforms plastically. For link lengths less than $1.6M_p/V_p$ attached to columns, link end moments do not fully equalize (Kasai and Popov, 1986a). For this situation, the link ultimate forces can be estimated as follows:

For links attached to columns with $e \le \dfrac{1.6M_p}{V_p}$

$$\text{Link shear} = 1.25R_yV_p \tag{C-F3-5}$$

$$\text{Link end moment at column} = R_yM_p \tag{C-F3-6}$$

$$\text{Link end moment at brace} = \left[e(1.25R_yV_p) - R_yM_p\right] \ge 0.75R_yM_p \tag{C-F3-7}$$

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 preceding equations. Further, as long as the beam and brace are designed to remain essentially elastic, the distribution of link end moment to the beam and brace can be estimated from an elastic frame analysis.

This is typically done by multiplying the beam and brace forces by the ratio of the expected, strain-hardened link shear strength to the link shear demand from the analysis. One could also use a free-body diagram to determine these forces based on the link strength and apportion moments based on the elastic analysis. For example, if an elastic analysis of the EBF under lateral load shows that 80% of the link end moment is resisted by the beam and the remaining 20% is resisted by the brace, the ultimate

link end moments given by the above equations can be distributed to the beam and brace in the same proportions. Care should be taken in this latter approach if the centerline intersections fall outside the link; see commentary for Section F3.5b.

Finally, an inelastic frame analysis can be conducted for a more accurate estimate of how link end moment is distributed to the beam and brace in the inelastic range.

As described in the preceding, the Provisions assume that as a link deforms under large plastic rotations, the link expected shear strength will increase by a factor of 1.25 due to strain hardening. However, for the design of the beam segment outside of the link, the Provisions permit reduction of the seismic force by a factor of 0.88, consistent with the 1.1 factor in the 2005 Provisions $\left[1.25(0.88) = 1.1\right]$. This relaxation on link ultimate forces for purposes of designing the beam segment reflects the view that beam strength will be substantially enhanced by the presence of a composite floor slab, and also that limited yielding in the beam will not likely be detrimental to EBF performance, as long as stability of the beam is assured. Consequently, designers should recognize that the actual forces that will develop in the beam will be substantially greater than computed using this 1.1 factor, but this low value of required beam strength will be mitigated by contributions of the floor slab in resisting axial load and bending moment in the beam and by limited yielding in the beam. Based on this approach, a strain hardening factor of 1.25 is called for in the analysis for I-shaped links. The resulting axial force and bending moment in the beam can then be reduced by a factor of 1.1/1.25 = 0.88. In cases where no composite slab is present, designers should consider computing required beam strength based on a link strain hardening factor of 1.25.

Design of the beam segment outside of the link can sometimes be problematic in EBF. In some cases, the beam segment outside of the link is inadequate to resist the required strength based on the link ultimate forces. For such cases, increasing the size of the beam may not provide a solution because the beam and the link are typically the same member. Increasing the beam size therefore increases the link size, which in turn, increases the link ultimate forces and therefore increases the beam required strength. The relaxation in beam required strength based on the 1.1 factor on link strength was adopted by the Provisions largely as a result of such problems reported by designers, and by the view that EBF performance would not likely be degraded by such a relaxation due to beneficial effects of the floor slab and limited beam yielding, as discussed above. Design problems with the beam can also be minimized by using shear yielding links $\left(e \le 1.6M_p/V_p\right)$ as opposed to longer links. The end moments for shear yielding links will be smaller than for longer links, and consequently less moment will be transferred to the beam. Beam moments can be further reduced by locating the intersection of the brace and beam centerlines inside of the link, as described below. Providing a diagonal brace with a large flexural stiffness so that a larger portion of the link end moment is transferred to the brace and away from the beam can also substantially reduce beam moment. In such cases, the brace must be designed to resist these larger moments. Further, the connection between the brace and the link must be designed as a fully restrained moment-resisting connection. Test results on several brace connection details subject to axial load and bending moment

are reported in Engelhardt and Popov (1989a). Finally, built-up members can be considered for link design.

High axial forces in the beam outside the link can complicate beam selection if the beam outside the link and the link beam are the same member, as is typical. These axial forces can be reduced or eliminated by selection of a beneficial configuration. Frames with center links may be reconfigured to eliminate beam axial forces from levels above by adopting a two-story-X configuration as proposed by Engelhardt and Popov (1989b) and shown in Figure C-F3.3. Frames with the link at the column share the frame shear between the brace and the column at the link. Selection of beneficial bay size and link length can maximize the percentage of the frame shear resisted by the column, thus minimizing the horizontal component of the brace force and consequently minimizing the axial force in the beam outside the link of the level below. More specifically, avoiding very shallow angles (less than 40°) between the diagonal brace and the beam is recommended (Engelhardt et al., 1992).

The required strength of the diagonal brace connections in EBF is the same as the required strength of the diagonal brace. Similar to the diagonal brace and beam segment outside of the link, the columns of an EBF should also be designed using capacity-design principles. That is, the columns should be designed to resist the maximum forces developed by the fully yielded and strain hardened links. As discussed in Commentary Section F3.5b and in this section, the maximum shear force developed by a fully yielded and strain hardened link can be estimated as $1.25R_y$ times the link nominal shear strength, V_n, where the 1.25 factor accounts for strain hardening. For capacity design of the columns, this section permits reduction of the strain hardening factor to 1.1 by multiplying seismic forces by a factor of 0.88 [1.25(0.88) = 1.1]. This relaxation reflects the view that all links above the level of the column under consideration will not likely reach their maximum shear strength simultaneously.

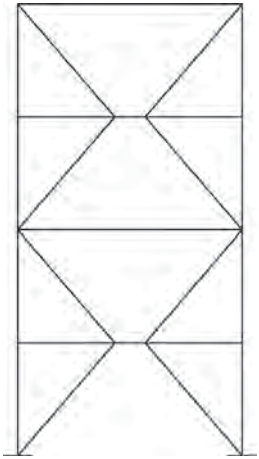


Fig. C-F3.3. Two-story-X EBF configuration (Engelhardt and Popov, 1989a).

Consequently, applying the 1.25 strain hardening factor to all links above the level of the column under consideration is likely too conservative for a multistory EBF. For a low-rise EBF with only a few stories, designers should consider increasing the strain hardening factor on links to 1.25 for capacity design of the columns, since there is a greater likelihood that all links may simultaneously reach their maximum shear strength. For taller buildings, this factor of 1.1 is likely overly conservative. No reliable methods have been developed for estimating such reduced forces on the basis of a linear analysis; designers may elect to perform a nonlinear analysis per Chapter C.

In addition to the requirements of this section, columns in EBF must also be checked in accordance with the requirements of Section D1.4a, which are applicable to all systems.

Tests showed (Berman and Bruneau, 2006, 2008a, 2008b) that strain hardening is larger for links with built-up box cross sections than for wide-flange links. Comparing the overstrength obtained for box links compared to that obtained for wide-flange links by Richards (2004), Berman and Bruneau indicated that built-up box rectangular links have a maximum strength typically 11% larger than wide-flange links. The forces to consider for the design of the braces, beams (outside the link), and columns are therefore increased accordingly.

4. System Requirements

4a. Link Rotation Angle

The total link rotation angle is the basis for controlling tests on link-to-column connections, as described in Section K2.4c. In a test specimen, the total link rotation angle is computed by simply taking the relative displacement of one end of the link with respect to the other end, and dividing by the link length. The total link rotation angle reflects both elastic and inelastic deformations of the link, as well as the influence of link end rotations. While the total link rotation angle is used for test control, acceptance criteria for link-to-column connections are based on the link inelastic rotation angle.

To ensure satisfactory behavior of an EBF, the inelastic deformation expected to occur in the links in a severe earthquake should not exceed the inelastic deformation capacity of the links. In the Provisions, the link rotation angle is the primary variable used to describe inelastic link deformation. The link rotation angle is the plastic rotation angle between the link and the portion of the beam outside of the link.

The link rotation angle can be estimated by assuming that the EBF bay will deform in a rigid-plastic mechanism as illustrated for various EBF configurations in Figure C-F3.4. In this figure, the link rotation angle is denoted by the symbol γ_p. The link rotation angle can be related to the plastic story drift angle, θ_p, using the relationships shown in Figure C-F3.4. The plastic story drift angle, in turn, can be computed as the plastic story drift, Δ_p, divided by the story height, h. The plastic story drift is equal to the difference between the design story drift and the elastic drift. Alternatively, the link rotation angle can be determined more accurately by inelastic dynamic analyses.

The inelastic response of a link is strongly influenced by the length of the link as related to the ratio, M_p/V_p, of the link cross section. 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 link lengths intermediate between these values, the inelastic response will occur through some combination of shear and flexural yielding. The inelastic deformation capacity of links is generally greatest for shear yielding links, and smallest for flexural yielding links. Based on experimental evidence, the link rotation angle is limited to 0.08 rad for shear yielding links $\left(e \le 1.6M_p/V_p\right)$ and 0.02 rad for flexural yielding links $\left(e \ge 2.6M_p/V_p\right)$. For links in the combined shear and flexural yielding range $\left(1.6M_p/V_p < e < 2.6M_p/V_p\right)$, the limit on link rotation angle is determined according to link length by linear interpolation between 0.08 and 0.02 rad.

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 providing links with an available shear strength at least 10% over the required shear strength.

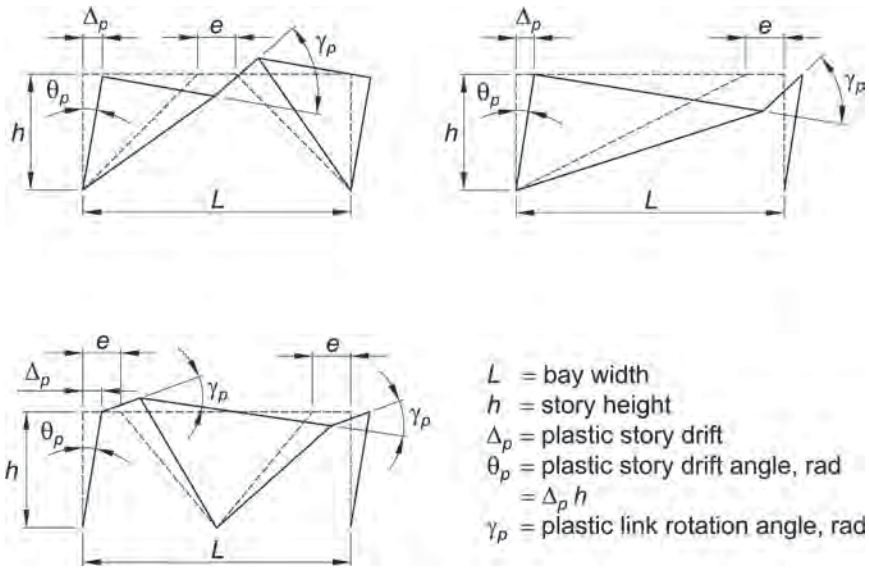


Fig. C-F3.4. Link rotation angle.

4b. Bracing of Link

Lateral restraint against out-of-plane displacement and twist is required at the ends of the link to ensure stable inelastic behavior. This section specifies the required strength and stiffness of link-end lateral bracing. In typical applications, a composite deck can likely be counted upon to provide adequate lateral bracing at the top flange of the link. However, a composite deck alone cannot be counted on to provide adequate lateral bracing at the bottom flange of the link and direct bracing through transverse beams or a suitable alternative is recommended.

A link with a built-up box cross section, tested without lateral bracing in a full EBF configuration, exhibited no lateral-torsional buckling (Berman and Bruneau, 2007). Slender box cross sections (significantly taller than wide) could develop lateral-torsional buckling, but the unbraced length required to do so for such sections is still considerably longer than for wide-flange links. As a result, except for unusual aspect ratios, links with built-up box cross sections will not require lateral bracing. While no physical lateral bracing is required to ensure satisfactory seismic performance of links with built-up box sections designed as specified in the Provisions, a lateral load acting outside of the frame plane and applied at the brace-to-beam points has been conservatively specified, together with a stiffness requirement, to prevent the use of link beams that would be too weak or flexible (out-of-plane of the frame) to provide lateral restraint to the brace.

5. Members

5a. Basic Requirements

The ductility demands in EBF are concentrated in the links. Braces, columns and beams outside the link should have very little yielding in a properly designed EBF. As long as the brace is designed to be stronger than the link, as is the intent of these provisions, the link will serve as a fuse to limit the maximum load transferred to the brace, thereby precluding the possibility of brace buckling. Consequently, many of the design provisions for braces in SCBF systems intended to permit stable cyclic buckling of braces are not needed in EBF. Similarly, the link also limits the loads transferred to the beam beyond the link, thereby precluding failure of this portion of the beam if it is stronger than the link.

For most EBF configurations, the beam and the link are a single continuous wide flange member. If this is the case, the available strength of the beam can be increased by R_y. If the link and the beam are the same member, any increase in yield strength present in the link will also be present in the beam segment outside of the link.

5b. Links

Inelastic action in EBF is intended to occur primarily within the links. The general provisions in this section are intended to ensure that stable inelasticity can occur in the link.

At brace connections to the link, the link length is defined by the edge of the brace connection; see Figure C-F3.5. (Bracing using HSS members is shown in Figure C-F3.6.) Brace connection details employing gussets are commonly configured so that the gusset edge aligns vertically with the intersection of the brace and beam centerlines. For brace connections not employing gussets, the intersection of the brace at the link end may not align vertically with the intersection of the brace and beam centerlines; the intersection of centerlines may fall within the link (Figure C-F3.5) or outside of the link (Figure C-F3.7). In either case, flexural forces in the beam outside the link and the brace may be obtained from an analysis that models the member centerline intersections, provided that the force level in the analysis corresponds to the expected strain-hardened link capacity as required by Section F3.3. However, such a centerline analysis will not produce correct link end moments. See Commentary Section F3.5b.1 and Figure C-F3.5. Link end moments for either case can more accurately be obtained using the following equation:

$$M = \frac{Ve}{2} \qquad \text{(C-F3-8)}$$

where V is the link beam shear in the condition under consideration (whether it be corresponding to the design base shear or to the fully yielded, strain-hardened link as required in Section F3.3).

However, link end moments are not directly used in selecting the link member in the typical design procedure. Section F3.5b.2 converts link flexural strength to an equivalent shear strength based on link length. Comparison of that equivalent shear strength

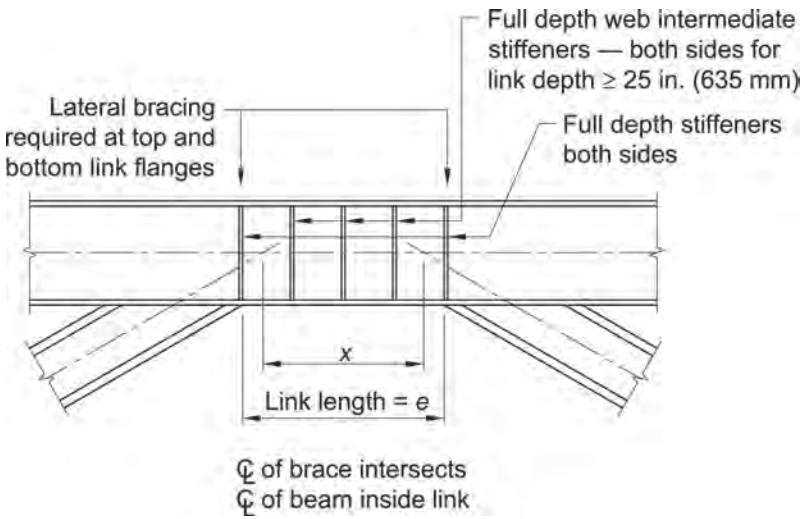


*Fig.C-F3.5. EBF with W-shape bracing (*x < e*).*

to the required shear strength is sufficient for design and the results of a centerline model analysis can be used without modification.

1. Limitations

Width-to-thickness limits for links are specified in Table D1.1. Previous editions of the Provisions required the link cross section to meet the same width-to-thickness criteria as is specified for beams in SMF. Based on research on local buckling in links (Okazaki et al., 2004a; Richards et al., 2004), the flange width-to-thickness limits for links are only required to meet the compactness limits for moderately ductile members. This new limit corresponds to λ_p in *Specification* Table B4.1b. Limits on slenderness of link built-up box cross sections are provided to prevent links that are significantly taller than wide (that could develop lateral-torsional buckling). Based on research by Berman and Bruneau (2008a, 2008b), the Provisions require that, for built-up box links with link lengths $e \leq 1.6M_p/V_p$, the web width-to-thickness ratio be limited to $1.67\sqrt{E/F_y}$, which is revised to $1.75\sqrt{E/(R_yF_y)}$ in Table D1.1 to address material overstrength. For built-up box links with link lengths $e > 1.6M_p/V_p$, it is recommended that the web width-to-thickness ratio be limited to $0.64\sqrt{E/F_y}$, which is revised to $0.67\sqrt{E/(R_yF_y)}$ in Table D1.1 to address material overstrength. Specimens with links other than at mid-width of the braced bay have not been tested.

The reinforcement of links with web doubler plates is not permitted as such reinforcement may not fully participate as intended in inelastic deformations. Additionally, beam web penetrations within the link are not permitted because they may adversely affect the inelastic behavior of the link.

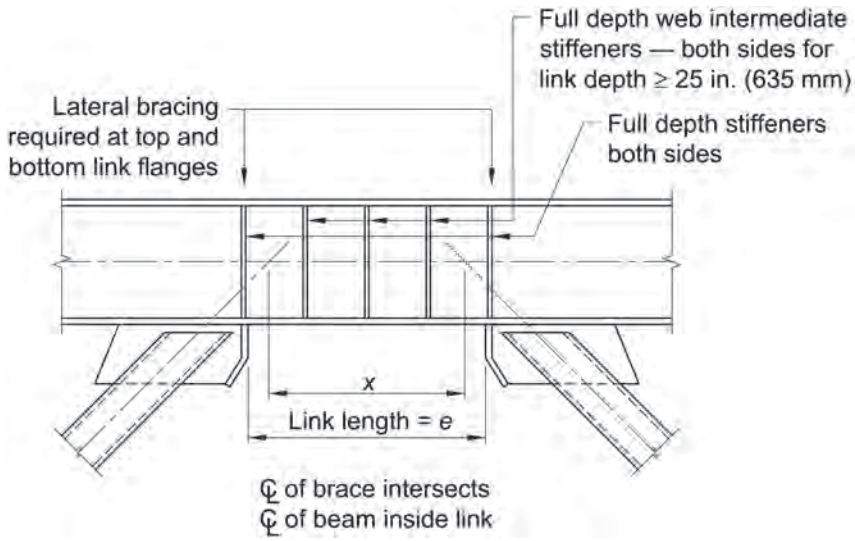


*Fig. C-F3.6. EBF with HSS bracing (*x < e*).*

The 2005 Provisions (AISC, 2005) required that the intersection of the beam and brace centerlines occur at the end of the link, or inside of the link. The reason for this restriction was that when the intersection of the beam and brace centerlines occurs outside of the link, additional moment is generated in the beam outside of the link. However, locating the intersection of the beam and brace centerline outside of the link is sometimes unavoidable for certain member sizes and brace connection geometries. Further, it is acceptable to locate the intersection outside of the link, as long as the additional moment in the beam is considered in the design. Consequently, the restriction has been removed to allow greater flexibility in EBF design.

When the distance between intersection of the beam and brace centerlines, x, exceeds the link length, e, as is shown in Figure C-F3.7, the total moment resisted by the beam outside the link and the brace (if moment-connected) exceeds the link end moment. Conversely if the link length, e, exceeds the distance between the intersection of the beam and brace centerlines, x, as is shown in Figures C-F3.5 and C-F3.6, the link end moment at the design level will exceed the forces indicated using a centerline model. In both conditions, care should be taken to ensure sufficient strength at the design level and proper estimation of forces in the beam outside the link and in the brace at drifts corresponding to a fully yielded, strain-hardened link.

2. Shear Strength

The nominal shear strength of the link, V_n, is the lesser of that determined from the plastic shear strength of the link section or twice the plastic moment divided by the link length, as dictated by statics assuming equalization of end moments

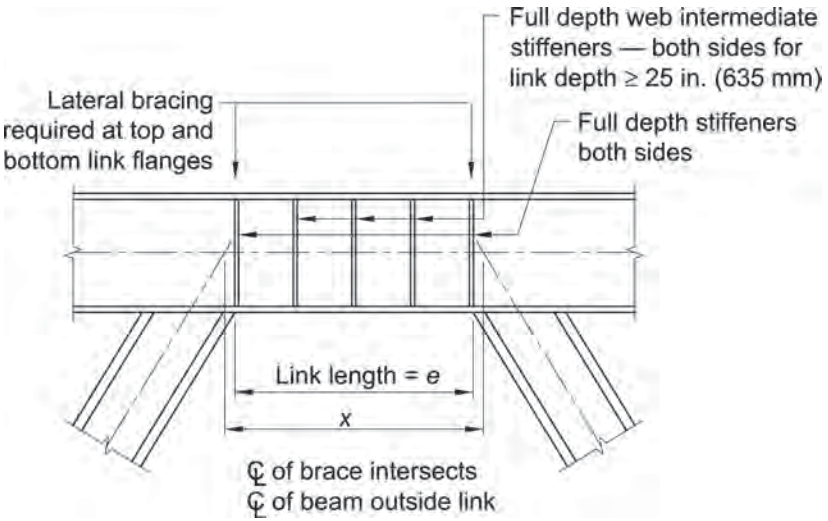


*Fig. C-F3.7. EBF with W-shape bracing (*x > e*).*

in the inelastic range of behavior. Accordingly, the nominal shear strength of the link can be computed as follows:

$$V_n = \begin{cases} V_p & \text{for } e \le \dfrac{2M_p}{V_p} \\ \dfrac{2M_p}{e} & \text{for } e > \dfrac{2M_p}{V_p} \end{cases} \qquad \text{(C-F3-9)}$$

The effects of axial load on the link can be ignored if the required axial strength of the link does not exceed 15% of the axial 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 available shear strength and the length of the link are reduced (according to Sections F3.5b.2 and F3.5b.3, respectively).

3. Link Length

The rotations that can be achieved in links subject to flexural yielding with high axial forces have not been adequately studied. Consequently, where high axial forces can develop in the link, its length is limited to ensure that shear yielding, rather than flexural yielding, governs to ensure stable inelastic behavior.

4. Link Stiffeners for I-Shaped Cross Sections

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.

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.

The maximum spacing of link intermediate web stiffeners in shear yielding links $\left(e \le 1.6M_p/V_p\right)$ is dependent upon the size of the link rotation angle (Kasai and Popov, 1986b) with a closer spacing required as the rotation angle increases. Intermediate web stiffeners in shear yielding links are provided to delay the onset of inelastic shear buckling of the web. Flexural yielding links having lengths greater than or equal to $2.6M_p/V_p$ 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 limit strength degradation due to flange local buckling and lateral-torsional buckling. Links of a length that are between the shear and flexural limits are required to meet the stiffener requirements for both shear and flexural yielding 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 to effectively limit strength degradation due to flange local buckling

and lateral-torsional buckling. Link intermediate web stiffeners are required on both sides of the web for links 25 in. (635 mm) in depth or greater. For links that are less than 25 in. (635 mm) deep, the stiffener need be on one side only.

All link stiffeners are required to be fillet welded to the link web and flanges. Link stiffeners should be detailed to avoid welding in the *k*-area of the link. Recent research has indicated that stiffener-to-link web welds that extend into the *k*-area of the link can generate link web fractures that may reduce the plastic rotation capacity of the link (Okazaki et al., 2004a; Richards et al., 2004).

5. Link Stiffeners for Box Sections

Similar to wide-flange links, the maximum spacing of stiffeners for shear yielding built-up box links $\left(e \le 1.6M_p/V_p\right)$ is dependent upon the magnitude of the link rotation angle. The equation for maximum spacing needed for the links to develop a link rotation angle of 0.08 rad [specified as $20t_w - (d-2t_f)/8$] is derived in Berman and Bruneau (2005a). A similar equation was also derived for a 0.02 rad limit, resulting in a maximum required stiffener spacing of $37t_w - (d-2t_f)/8$. However, experimental and analytical data is only available to support the closer stiffener spacing required for the 0.08 rad link rotation angle. Therefore, that more restrictive stiffener spacing is required for all links until other data becomes available.

The use of intermediate web stiffeners was shown (Berman and Bruneau, 2006, 2008a, 2008b) to be significant on the shear yielding strength in built-up box links with h/t_w greater than $0.64\sqrt{E/F_y}$ and less than or equal to $1.67\sqrt{E/F_y}$. For shear links with h/t_w less than or equal to $0.64\sqrt{E/F_y}$, flange buckling was the controlling limit state and intermediate stiffeners had no effect. Thus, intermediate web stiffeners are not required for links with web depth-to-thickness ratios less than $0.64\sqrt{E/F_y}$, which has been converted to $0.67\sqrt{E/R_yF_y}$ in this edition of the Provisions to address material overstrength. For links with lengths exceeding $1.6M_p/V_p$, compression local buckling of both webs and flanges (resulting from the compressive stresses associated with the development of the plastic moment) dominated link strength degradation. This buckling was unaffected by the presence of intermediate web stiffeners. As a result, intermediate web stiffeners are not required for links with lengths exceeding $1.6M_p/V_p$.

When intermediate stiffeners were used in the built-up box tested and simulated numerically by Berman and Bruneau (2006, 2008a, 2008b), these stiffeners were welded to both the webs and the flanges. A typical cross section is shown in Figure C-F3.8. However, presence of the stiffeners did not impact flange buckling, and these may therefore not need to be connected to the flange. This would have advantages over the detail in Figure C-F3.8. In particular, the intermediate stiffeners could be fabricated inside the built-up box link, improving resistance to corrosion and risk of accumulation of debris between the stiffeners (in cases of exterior exposures), and enhancing architectural appeal. Review of the literature (Malley and Popov, 1983; Bleich, 1952; Salmon and Johnson, 1996) showed that the derivation of minimum required areas and moment of inertia equations

for sizing intermediate stiffeners did not depend on connection to the flanges. Whereas web stiffeners in I-shaped links may also serve to provide stability to the flanges (Malley and Popov, 1983), this is not the case in built-up box cross sections. Thus, welding of intermediate stiffeners to the flanges of the built-up box section links is not critical and not required.

5c. Protected Zones

The link, as the expected area of inelastic strain, is a protected zone.

6. Connections

6a. Demand Critical Welds

Inelastic strain in the weld material is likely at column base plates, column splices, and in moment connections in eccentrically braced frames. In addition, it is likely in welds of a built-up link member. Thus these are required to be treated as demand critical welds. See Commentary Section F2.6a.

6b. Beam-to-Column Connections

See Commentary Section F2.6b.

6c. Brace Connections

In the 2005 Provisions, the brace connection was required to be designed for the same forces as the brace (which are the forces generated by the fully yielded and strain hardened link). The brace connection, however, was also required to be designed for a compressive axial force corresponding to the nominal buckling strength of the

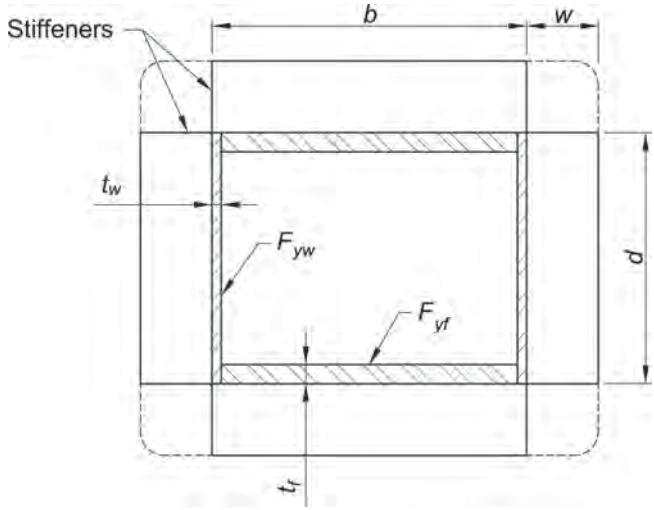


Fig. C-F3.8. Built-up box link cross section with intermediate stiffener.

brace. This second requirement has been eliminated. Braces in EBF are designed to preclude buckling, and it is considered unnecessarily conservative to design the brace connection for the buckling strength of the brace.

Bracing connections are required to be designed to resist forces corresponding to link yielding and strain hardening. The strain hardening factors used in Section F3.3—1.25 for I-shaped links and 1.4 for box links—are somewhat low compared to some values determined from testing; however, the reliability of connections remains sufficient due to the use of lower resistance factors for nonductile limit states.

Bolt slip has been removed as a limit state which must be precluded. The consequences of exceeding this limit state in the maximum credible earthquake are not considered severe if bearing failure and block-shear rupture are precluded.

A few EBF link fractures were observed following the Christchurch earthquake series of 2010 and 2011 (Clifton et al., 2011). Finite element analyses conducted to investigate this behavior revealed that when braces frame into the link beam and no gusset is used, eccentricity (misalignment) of link stiffeners with respect to the beam-to-brace flange connection point can lead to severe local ductility demands and premature failures outside of the link (Imani and Bruneau, 2015; Kanvinde et al., 2014), as shown in Figure C-F3.9. For cases where modifying the brace section to achieve the preceding condition is not possible, analyses showed that moving the link stiffener to eliminate the offset between the end stiffener and beam-to-brace flange connection point can be effective to improve the overall behavior of the EBF frame, even if the intersection of the brace-to-beam centerlines falls inside the link (Imani and Bruneau, 2015).

6d. Column Splices

See Commentary Section F2.6d.

6e. Link-to-Column Connections

Prior to the 1994 Northridge earthquake, link-to-column connections were typically constructed in a manner substantially similar to beam-to-column connections in SMF. Link-to-column connections in EBF are therefore likely to share many of the same problems observed in moment frame connections. Consequently, in a manner similar to beam-to-column connections in SMF, the Provisions require that the performance of link-to-column connections be verified by testing in accordance with Section K2, or by the use of prequalified link-to-column connections in accordance with Section K1; there are no prequalified connections at the time of publication.

The load and deformation demands at a link-to-column connection in an EBF are substantially different from those at a beam-to-column connection in an SMF. Link-to-column connections must therefore be tested in a manner that properly simulates the forces and inelastic deformations expected in an EBF. Designers are cautioned that beam-to-column connections which qualify for use in an SMF may not necessarily perform adequately when used as a link-to-column connection in an EBF. Link-to-column connections must therefore be tested in a manner that properly simulates the forces and inelastic deformations expected in an EBF. For example, the

reduced beam section (RBS) connection has been shown to perform well in an SMF. However, the RBS is generally not suitable for link-to-column connections due to the high moment gradient in links. Similarly, recent research (Okazaki, 2004; Okazaki et al., 2004b) has demonstrated that other details that have shown good performance in moment frame beam-to-column connections (such as the WUF-W and the free flange details) can show poor performance in EBF link-to-column connections.

At the time of publication of the Provisions, development of satisfactory link-to-column connection details is the subject of ongoing research. Designers are therefore advised to consult the research literature for the latest developments. Until further research on link-to-column connections is available, it may be advantageous to avoid EBF configurations with links attached to columns.

The Provisions permit the use of link-to-column connections without the need for qualification testing for shear yielding links when the connection is reinforced with haunches or other suitable reinforcement designed to preclude inelastic action in the reinforced zone adjacent to the column. An example of such a connection is shown in Figure C-F3.10. This reinforced region should remain essentially elastic for the fully

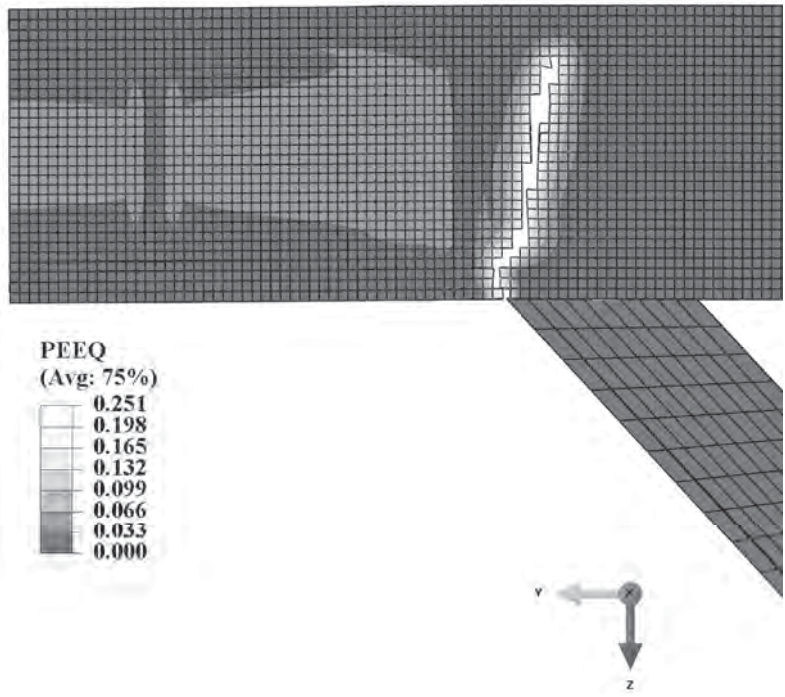


Fig. C-F3.9. Simulated fracture at offset between link stiffener and brace flange (with and without the equivalent plastic strain contour lines) from Imani and Bruneau (2015).

yielded and strain hardened link strength as required by Section F3.3; the exception for beams outside links does not apply. That is, the reinforced connection should be designed to resist the link shear and moment developed by the expected shear strength of the link, R_yV_n, multiplied by 1.25 to account for strain hardening. As an alternative to the reinforced link-to-column connection detail illustrated in Figure C-F3.10, preliminary testing and analysis have shown very promising performance for a reinforced connection detail wherein a pair of stiffeners is provided in the first link web panel next to the column, with the stiffeners oriented parallel to the link web. This link-to-column connection detail is described in Okazaki et al. (2009). Alternatively, the EBF can be configured to avoid link-to-column connections entirely.

The Provisions do not explicitly address the column panel zone design requirements at link-to-column connections. Based on limited research (Okazaki, 2004) it is recommended that the panel zone of link-to-column connections be designed in a manner similar to that for SMF beam-to-column connections with the required shear strength of the panel zone determined from the analysis required by Section F3.3.

F4. BUCKLING-RESTRAINED BRACED FRAMES (BRBF)

1. Scope

Buckling-restrained braced frames (BRBF) are a special class of concentrically braced frames. Just as in SCBF, the centerlines of BRBF members that meet at a joint intersect at a point to form a complete vertical truss system that resists lateral forces. BRBF have more ductility and energy absorption than SCBF because overall brace buckling, and its associated strength degradation, is precluded at forces and deformations corresponding to the design story drift. See Section F2 for the effects

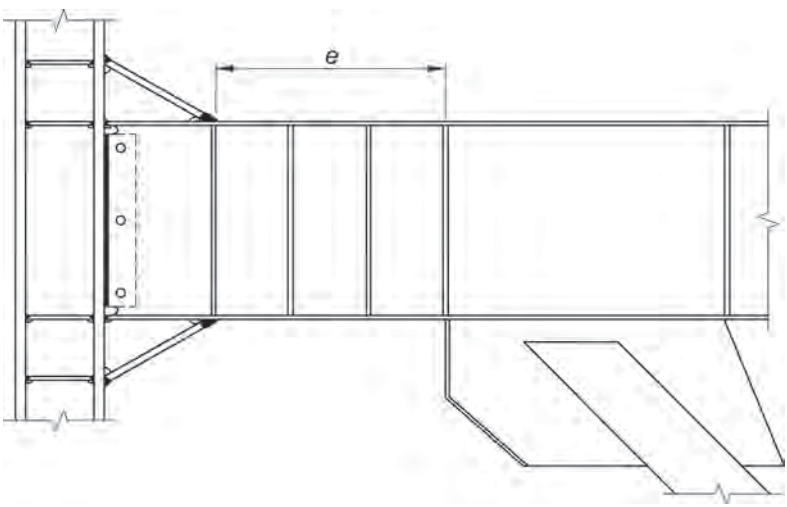


Fig. C-F3.10. Example of a reinforced link-to-column connection.

of buckling in SCBF. Figure C-F2.1 shows possible concentrically braced frame configurations; note that neither X-bracing nor K-bracing is an option for BRBF. Figure C-F4.1 shows a schematic of a BRBF bracing element [adapted from Tremblay et al. (1999)].

2. Basis of Design

BRBF are characterized by the ability of bracing elements to yield inelastically in compression as well as in tension. In BRBF, the bracing elements dissipate energy through stable tension-compression yield cycles (Clark et al., 1999). Figure C-F4.2 shows the characteristic hysteretic behavior for this type of brace as compared to that of a buckling brace. This behavior is achieved through limiting buckling of the steel core within the bracing elements. Axial stress is de-coupled from flexural buckling resistance; axial load is confined to the steel core while the buckling restraining mechanism, typically a casing, resists overall brace buckling and restrains high-mode steel core buckling (rippling).

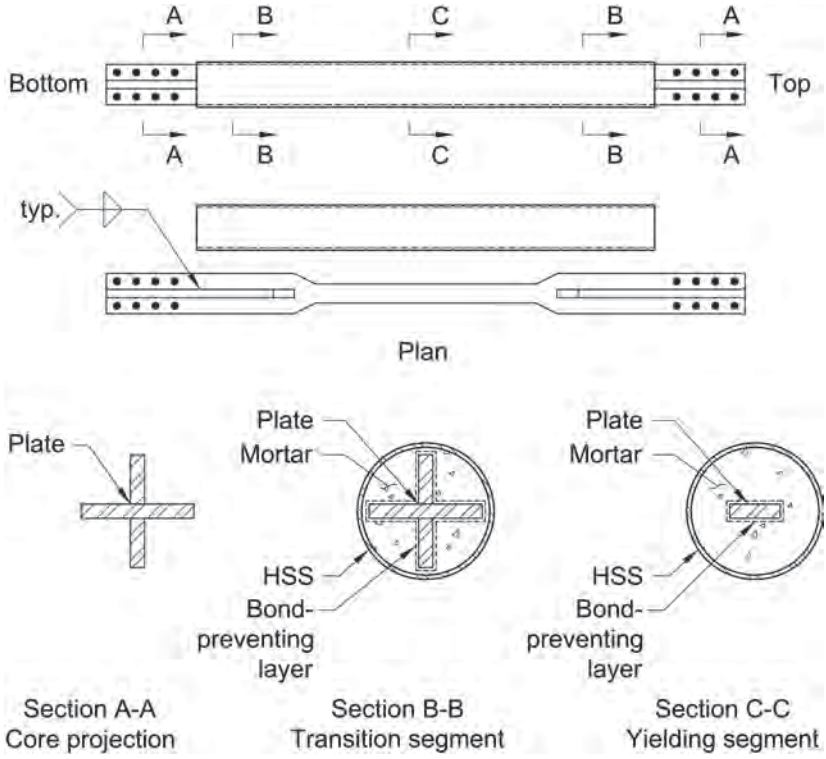


Fig. C-F4.1. Details of a type of buckling-restrained brace (courtesy of R. Tremblay).

Buckling-restrained braced frames are composed of columns, beams and bracing elements, all of which are subjected primarily to axial forces. Braces of BRBF, known as buckling-restrained braces (BRB), are composed of a steel core and a buckling-restraining system encasing the steel core. In addition to the schematic shown in Figure C-F4.1, examples of BRB elements are found in Watanabe et al. (1988); Wada et al. (1994); and Clark et al. (1999). The steel core within the BRB is intended to be the primary source of energy dissipation. During a moderate to severe earthquake the steel core is expected to undergo significant inelastic deformations.

BRBF can provide elastic stiffness that is comparable to that of EBF. Full-scale laboratory tests indicate that properly designed and detailed bracing elements of BRBF exhibit symmetrical and stable hysteretic behavior under tensile and compressive forces through significant inelastic deformations (Watanabe et al., 1988; Wada et al., 1998; Clark et al., 1999; Tremblay et al., 1999). The ductility and energy dissipation capability of BRBF is expected to be comparable to that of an SMF and greater than that of a SCBF. This high ductility is attained by limiting buckling of the steel core.

The Provisions are based on the use of brace designs qualified by testing. They are intended to ensure that braces are used only within their proven range of deformation capacity, and that yield and failure modes other than stable brace yielding are precluded at the maximum inelastic drifts corresponding to the design earthquake. For analyses performed using linear methods, the maximum inelastic drifts for this system are defined as those corresponding to 200% of the design story drift. For nonlinear time-history analyses, the maximum inelastic drifts can be taken directly

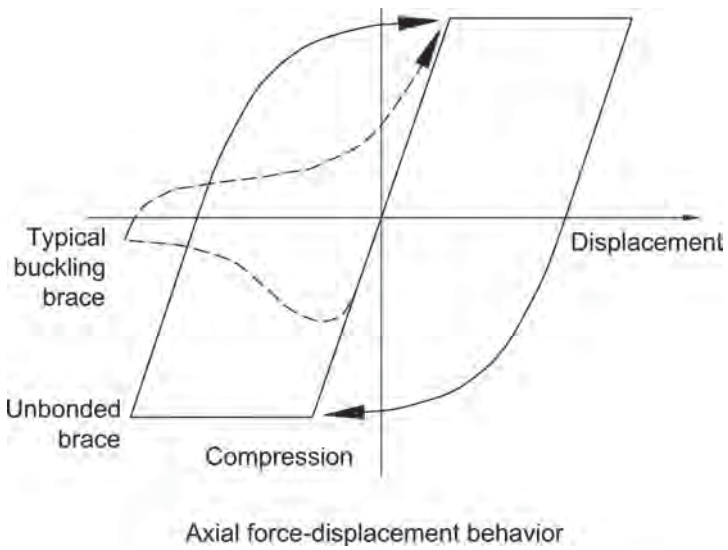


Fig. C-F4.2. Typical buckling-restrained (unbonded) brace hysteretic behavior (courtesy of Seismic Isolation Engineering).

from the analyses results. A minimum of 2% story drift is required for determining expected brace deformations for testing (see Section K3) and is recommended for detailing. This approach is consistent with the linear analysis equations for design story drift in ASCE/SEI 7 and the 2009 *NEHRP Recommended Provisions* FEMA P-750 (FEMA, 2009a). It is also noted that the consequences of loss of connection stability due to the actual seismic displacements exceeding the calculated values may be severe; braces are therefore required to have a larger deformation capacity than directly indicated by linear static analysis.

The value of 200% of the design story drift for expected brace deformations represents the mean of the maximum story response for ground motions having a 10% chance of exceedance in 50 years (Fahnestock et al., 2003; Sabelli et al., 2003). Near-fault ground motions, as well as stronger ground motions, can impose deformation demands on braces larger than those required by the Provisions. While exceeding the brace design deformation may result in poor brace behavior such as buckling, this is not equivalent to collapse. Detailing and testing braces for larger deformations will provide higher reliability and better performance.

The design engineer utilizing these provisions is strongly encouraged to consider the effects of configuration and proportioning of braces on the potential formation of building yield mechanisms. The axial yield strength of the core, P_{ysc}, can be set precisely with final core cross-sectional area determined by dividing the specified brace capacity by the actual material yield strength established by coupon testing, multiplied by the resistance factor. In some cases, cross-sectional area will be governed by brace stiffness requirements to limit drift. In either case, careful proportioning of braces can make yielding distributed over the building height much more likely than in conventional braced frames.

It is also recommended that engineers refer to the following documents to gain further understanding of this system: Uang and Nakashima (2003); Watanabe et al. (1988); Reina and Normile (1997); Clark et al. (1999); Tremblay et al. (1999); and Kalyanaraman et al. (1998).

The design provisions for BRBF are predicated on reliable brace performance. In order to ensure this performance, a quality assurance plan is required. These measures are in addition to those covered in the *Code of Standard Practice* (AISC, 2016c), and *Specification* Chapters J and N. Examples of measures that may provide quality assurance are:

(1) Special inspection of brace fabrication. Inspection may include confirmation of fabrication and alignment tolerances, as well as nondestructive testing (NDT) methods for evaluation of the final product.

(2) Brace manufacturer's participation in a recognized quality certification program. Certification should include documentation that the manufacturer's quality assurance plan is in compliance with the requirements of the *Specification*, the Provisions and the *Code of Standard Practice*. The manufacturing and

quality control procedures should be equivalent to, or better than, those used to manufacture brace test specimens.

2a. Brace Strength

Testing of braces is considered necessary for this system to ensure proper behavior. The applicability of tests to the designed brace is defined in Section K3.

Tests cited serve another function in the design of BRBF: the maximum forces that the brace can develop in the system are determined from test results. These maximum forces are used in the analysis required in Section F4.3.

2b. Adjustment Factors

The compression-strength adjustment factor, β, accounts for the compression overstrength (with respect to tension strength) noted in testing of buckling-restrained braces (SIE, 1999a, 1999b). The strain hardening adjustment factor, ω, accounts for strain hardening. Figure C-F4.3 shows a diagrammatic bilinear force-displacement relationship in which the compression strength adjustment factor, β, and the strain hardening adjustment factor, ω, are related to brace forces and nominal material yield strength. These quantities are defined as

$$\beta = \frac{\beta\omega F_{ysc} A_{sc}}{\omega F_{ysc} A_{sc}} = \frac{P_{max}}{T_{max}} \tag{C-F4-1}$$

$$\omega = \frac{\omega F_{ysc} A_{sc}}{F_{ysc} A_{sc}} = \frac{T_{max}}{F_{ysc} A_{sc}} \tag{C-F4-2}$$

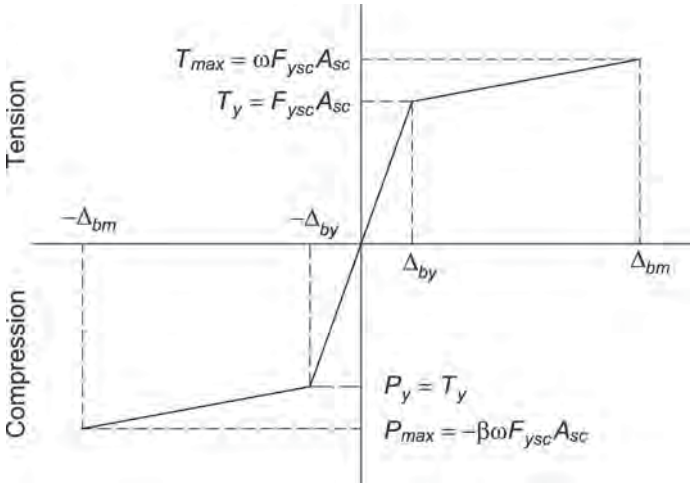


Fig. C-F4.3. Diagram of brace force-displacement.

where

A_{sc} = cross-sectional area of the yielding segment of steel core, in.^{2} (mm^2)
F_{ysc} = measured yield strength of the steel core, ksi (MPa)
P_{max} = maximum compression force, kips (N)
T_{max} = maximum tension force within deformations corresponding to 200% of the design story drift (these deformations are defined as $2.0\Delta_{bm}$ in Section K3.4c), kips (N)

Note that the specified minimum yield stress of the steel core, F_y, is not typically used for establishing these factors; instead, F_{ysc} is used which is determined by the coupon tests required to demonstrate compliance with Section K3. Braces with values of β and ω less than unity are not true buckling-restrained braces and their use is precluded by the Provisions.

The expected brace strengths used in the design of connections and of beams and columns are adjusted upwards for various sources of overstrength, including amplification due to expected material strength (using the ratio R_y) in addition to the strain hardening, ω, and compression adjustment, β, factors discussed previously. The amplification due to expected material strength can be eliminated if the brace yield stress is determined by a coupon test and is used to size the steel core area to provide the desired available strength precisely. Coupon testing, where used, should be performed at point of manufacture on each plate used for the fabrication of BRB yielding cores. The use of mill test report results is not equivalent to a coupon test. Where core plates are fabricated from bar stock, coupons should be made at intervals of (at most) each 5 tons of material of same heat and thickness. Other sources of overstrength, such as imprecision in the provision of the steel core area, may need to be considered; fabrication tolerance for the steel core is typically negligible.

3. Analysis

Beams and columns are required to be designed considering the maximum force that the adjoining braces are expected to develop. In the Provisions, these requirements are presented as an analysis requirement, although they are consistent with the design requirements in the 2005 and 2010 Provisions.

4. System Requirements

4a. V- and Inverted V-Braced Frames

In SCBF, V-bracing has been characterized by a change in deformation mode after one of the braces buckles. This is primarily due to the negative post-buckling stiffness, as well as the difference between tension and compression capacity, of traditional braces. Since buckling-restrained braces do not lose strength due to buckling and have only a small difference between tension and compression capacity, the practical requirements of the design provisions for this configuration are relatively minor. Figure C-F4.4 shows the effect of beam vertical displacement under the unbalanced load caused by the brace compression overstrength. The vertical beam deflection adds to the deformation demand on the braces, causing them to elongate more than they

compress (due to higher compression strength compared to tension strength). Therefore, where V-braced frames are used, it is required that a beam be provided that has sufficient strength to permit the yielding of both braces within a reasonable story drift considering the difference in tension and compression capacities determined by testing. The required brace deformation capacity must include the additional deformation due to beam deflection under this load. Since other requirements, such as the brace testing protocol (Section K3.4c) and the stability of connections (Section F4.6), depend on this deformation, engineers will find significant incentive to avoid flexible beams in this configuration. Where the special configurations shown in Figure C-F2.4 are used, the requirements of this section are not relevant.

4b. K-Braced Frames

K-braced frames are not permitted for BRBF due to the possibility of inelastic flexural demands on columns.

4d. Multi-Tiered Braced Frames

Multi-tiered braced frames (MTBF) are defined as braced-frame configurations with two or more tiers of bracing between diaphragm levels or locations of out-of-plane support. These tiers each incorporate a strut (beam) at each tier of bracing and are therefore not classified as K-braced frames. The strut required by these Provisions spans between frame columns, though a strut exterior to the frame can be incorporated as part of the design of the frame to resist in-plane moments from the analysis requirements.

In the multi-tiered BRBF system (MT-BRBF), in-plane column demands are imposed by varying tier capacities and unbalanced brace loads created by the difference between the BRB's overstrength in tension and compression. Studies done by

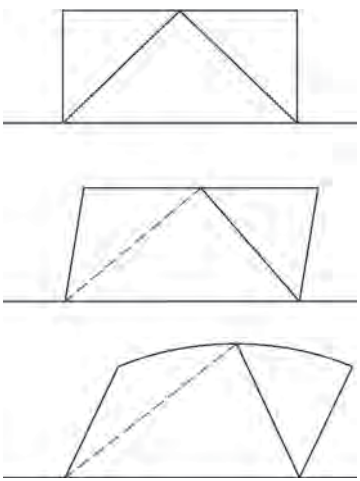


Fig. C-F4.4. Post-yield change in deformation mode for V- and inverted-V BRBF.

Imanpour et al. (2016a, 2016b, 2015) have shown BRBF frames to be the most stable of the MTBF configurations explored to date. In these studies, the MT-BRBF has not demonstrated a single tier mechanism, but some brace overstrength variation may occur from tier to tier. If the column or other resisting framework is not sufficient to support these loads, in-plane column yielding may occur. See the Commentary Section F2.4e for additional information. Although this phenomenon was studied for the SCBF and demonstrated primarily during a concentration of drift in a critical tier, the same precautions are being recommended for the BRBF as have been proposed for the SCBF to mitigate the potential column instability. The requirement for column torsional bracing at each brace connection location satisfying the requirements developed by Helwig and Yura (1999), which may be provided by the flexural stiffness of the tier strut, is necessary to provide stability to the column.

The Provisions allow for the design of MT-BRBF using similar design requirements as are used for typical BRBF frames. Adjusted brace strengths are determined for each tier and used for design of the struts and columns in the frame. Unlike the typical building case, for multi-tiered braced frames, tiers with varied capacities or the possibility of an overstrength imbalance between tiers will require the column to work in flexure. Imanpour and Tremblay (2014) have found that the application of adjusted brace strengths to the MT-BRBF frame overpredicts potential bending moments in the frame columns. However, the unique case where each tier is identical and braces are inclined in the same direction results in the applied moments in the columns being zero, a condition that would be unconservative. To address this, the minimum notional load requirement of 0.5% of the adjusted brace tier strength of each tier have been added to the design provisions. With an in-plane load at each tier, the static-equilibrium method may then be followed with the columns treated as members spanning simply supported between the base and top of the MT-BRBF frame. The resulting method of adjusted brace strengths and the 0.5% minimum notional load provides for column moments that may be incurred due to a variation in the strains in the braces, tolerances on the core cut widths, and possible small variations between the independently tested core yield strengths and the final core yield strength. However, it may not provide for column moments that may be incurred due to tier capacity differences caused by R_y or the specification of braces using a fixed area and a range of permitted yield strengths of the core material. Although there is no evidence that this material variation is detrimental to the MT-BRBF, a factor to account for the range of expected yield strengths of the braces has been included in the Provisions. The specification of the BRB by required capacity, P_{ysc}, rather than by core area, A_{sc}, is a simple method to control the capacity of each tier such that the tier capacities are similar in a given frame and column bending moments in the plane of the frame are reduced.

In Figure C-F4.5, "ABS" indicates unbalanced loads applied to the columns due to variation in adjusted brace strengths, and "NOT" indicates the required notional loads. Only the second and the fifth tier in this example have unbalanced loads due to

adjusted brace strengths alone, and these loads are greater than the minimum notional load. NOT loads are applied in the direction producing the maximum moment on the column and analysis considers seismic loads in each direction.

A series of columns may be used to support the loads from this analysis. In this case, these must be designed for the portion of in-plane tier loads combined with the axial compression on the column.

5. Members

5a. Basic Requirements

Previous editions of these Provisions required highly ductile sections for beams and columns. The development of the requirement stemmed primarily from consideration of moment frames and other systems where stable, fully developed plastic hinges of up to 0.04 rad are necessary for proper performance of the system. Beams and columns in BRBF are designed for the adjusted strength of the braces and are intended to remain essentially elastic in a seismic event.

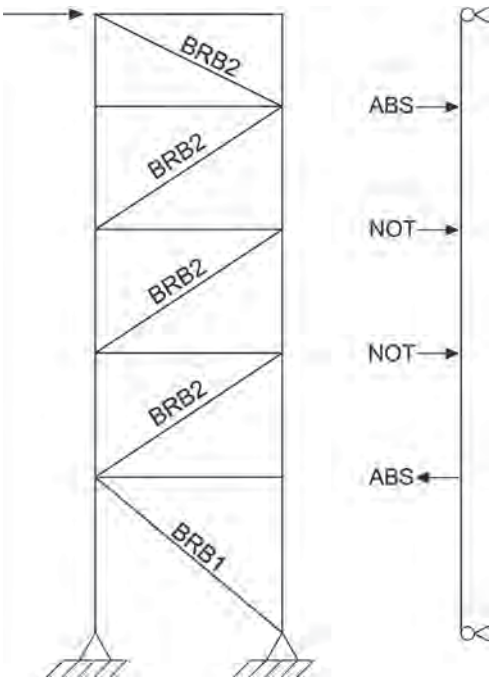


Fig. C-F4.5. MT-BRBF elevation.

5b. Diagonal Braces

1. Assembly

(a) Steel Core

The steel core is composed of a yielding segment and steel core projections; it may also contain transition segments between the projections and yielding segment. The cross-sectional area of the yielding segment of the steel core is expected to be sized so that its yield strength is fairly close to the demand calculated from the applicable building code. Designing braces close to the required strengths will help ensure distribution of yielding over multiple stories in the building. Conversely, over-designing some braces more than others (e.g., by using the same size brace on all floors) may result in an undesirable concentration of inelastic deformations in only a few stories. The length and area of the yielding segment, in conjunction with the lengths and areas of the nonyielding segments, determine the stiffness of the brace. The yielding segment length and brace inclination also determines the strain demand corresponding to the design story drift.

In typical brace designs, a projection of the steel core beyond its casing is necessary in order to accomplish a connection to the frame. Buckling of this unrestrained zone is an undesirable failure mode and must therefore be precluded.

In typical practice, the designer specifies the core plate dimensions as well as the steel material and grade. The steel stress-strain characteristics may vary significantly within the range permitted by the steel specification, potentially resulting in significant brace overstrength. This overstrength must be addressed in the design of connections as well as of frame beams and columns. The designer may specify a limited range of acceptable yield stress in order to more strictly define the permissible range of brace capacity. Alternatively, the designer may specify a limited range of acceptable yield stress if this approach is followed in order to more strictly define the permissible range of core plate area (and the resulting brace stiffness). The brace supplier may then select the final core plate dimensions to meet the capacity requirement using the results of a coupon test. The designer should be aware that this approach may result in a deviation from the calculated brace axial stiffness. The maximum magnitude of the deviation is dependent on the range of acceptable material yield stress. Designers following this approach should consider the possible range of stiffness in the building analysis in order to adequately address both the building period and expected drift.

The strength of the steel core has been defined in terms of a symbol, F_{ysc}, which is defined as either the specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon

test. The use of coupon tests in establishing F_{ysc} eliminates the necessity of using the factor R_y in calculating the adjusted brace strength (see Commentary Section F4.2a). This is in recognition of the fact that coupon testing of the steel core material is in effect required by the similitude provisions in Section K3, and such coupon tests can provide a more reliable estimation of expected strength.

(b) Buckling-Restraining System
This term describes those elements providing brace stability against overall buckling. This includes the casing as well as elements connecting the core. The adequacy of the buckling-restraining system must be demonstrated by testing.

2. Available Strength

The nominal strength of buckling-restrained braces is simply based on the core area and the material yield strength. Buckling is precluded, as is demonstrated by testing.

3. Conformance Demonstration

BRBF designs require reference to successful tests of a similarly sized test specimen and of a brace subassemblage that includes rotational demands. The former is a uniaxial test intended to demonstrate adequate brace hysteretic behavior. The latter is intended to verify the general brace design concept and demonstrate that the rotations associated with frame deformations do not cause failure of the steel core projection, binding of the steel core to the casing, or otherwise compromise the brace hysteretic behavior. A single test may qualify as both a subassemblage and a brace test subject to the requirements of Section K3; for certain frame-type subassemblage tests, obtaining brace axial forces may prove difficult and separate brace tests may be necessary. A sample subassemblage test is shown in Figure C-K3.1 (Tremblay et al., 1999).

5c. Protected Zones

The core, as the expected area of inelastic strain, is a protected zone along with all elements connecting the core to the beams and columns, which may include gusset plates and gusset connections.

6. Connections

6a. Demand Critical Welds

Inelastic strain in the weld material is likely at column base plates and column splices. Thus these are required to be treated as demand critical welds. See Commentary Section F2.6a.

6b. Beam-to-Column Connections

See Commentary Section F2.6b.

6c. Diagonal Brace Connections

Bracing connections must not yield at force levels corresponding to the yielding of the steel core; they are therefore designed for the maximum force that can be expected from the brace (see Commentary Section F4.5b). The engineer should recognize that the bolts are likely to slip at forces 30% lower than their design strength. This slippage is not considered to be detrimental to behavior of the BRBF system and is consistent with the design approach found in Section D2.2.

Recent testing in stability and fracture has demonstrated that gusset-plate connections may be a critical aspect of the design of BRBF (Tsai et al., 2003; Lopez et al., 2004). The tendency to instability may vary depending on the flexural stiffness of the connection portions of the buckling-restrained brace and the degree of their flexural continuity with the casing. This aspect of BRBF design is the subject of continuing investigation and designers are encouraged to consult research publications as they become available. The stability of gussets may be demonstrated by testing, if the test specimen adequately resembles the conditions in the building. It is worth noting that during an earthquake the frame may be subjected to some out-of-plane displacement concurrent with the in-plane deformations, so a degree of conservatism in the design of gussets may be warranted.

Fahnestock et al. (2006) tested a connection, shown in Figure C-F4.6, that effectively provided a pin in the beam outside of the gusset plate via the splice with a WT section on each side. In addition to satisfying the requirements of Section F4.6b, this connection relieves the gusset plate of in-plane moments and the related destabilization effects.

6d. Column Splices

See Commentary Section F2.6d.

F5. SPECIAL PLATE SHEAR WALLS (SPSW)

1. Scope

In special plate shear walls (SPSW), the slender unstiffened steel plates (webs) connected to surrounding horizontal and vertical boundary elements (HBE and VBE) are designed to yield and behave in a ductile hysteretic manner during earthquakes (see Figure C-F5.1). All HBE are also rigidly connected to the VBE with moment resisting connections able to develop the expected plastic moment of the HBE. Each web must be surrounded by boundary elements.

Experimental research on SPSW subjected to cyclic inelastic quasi-static and dynamic loading has demonstrated their ability to behave in a ductile manner and dissipate significant amounts of energy (Thorburn et al., 1983; Timler and Kulak, 1983; Tromposch and Kulak, 1987; Roberts and Sabouri-Ghomi, 1992; Caccese et al., 1993; Driver et al., 1997; Elgaaly, 1998; Rezai, 1999; Lubell et al., 2000; Grondin and Behbahannidard, 2001; Berman and Bruneau, 2003a; Zhao and Astaneh-Asl, 2004; Berman and Bruneau, 2005b; Sabouri-Ghomi et al., 2005; Deng et al., 2008;

Lee and Tsai, 2008; Qu et al., 2008; Choi and Park, 2009; Qu and Bruneau, 2009; Vian et al., 2009a). This has been confirmed by analytical studies using finite element analysis and other analysis techniques (Sabouri-Ghomi and Roberts, 1992; Elgaaly et al., 1993; Elgaaly and Liu, 1997; Driver et al., 1997; Dastfan and Driver, 2008; Bhowmick et al., 2009; Purba and Bruneau, 2009; Shishkin et al., 2009; Vian et al., 2009b; Qu and Bruneau, 2011; Purba and Bruneau, 2014a).

2. Basis of Design

Yielding of the webs occurs by development of tension field action at an angle close to 45° from the vertical, and buckling of the plate in the orthogonal direction. Past research shows that the sizing of VBE and HBE in a SPSW makes it possible to develop this tension field action across all of the webs. Except for cases with very stiff HBE and VBE, yielding in the webs develops in a progressive manner across each panel. Because the webs do not yield in compression, continued yielding upon repeated cycles of loading is contingent upon the SPSW being subjected to progressively larger drifts, except for the contribution of plastic hinging developing in the

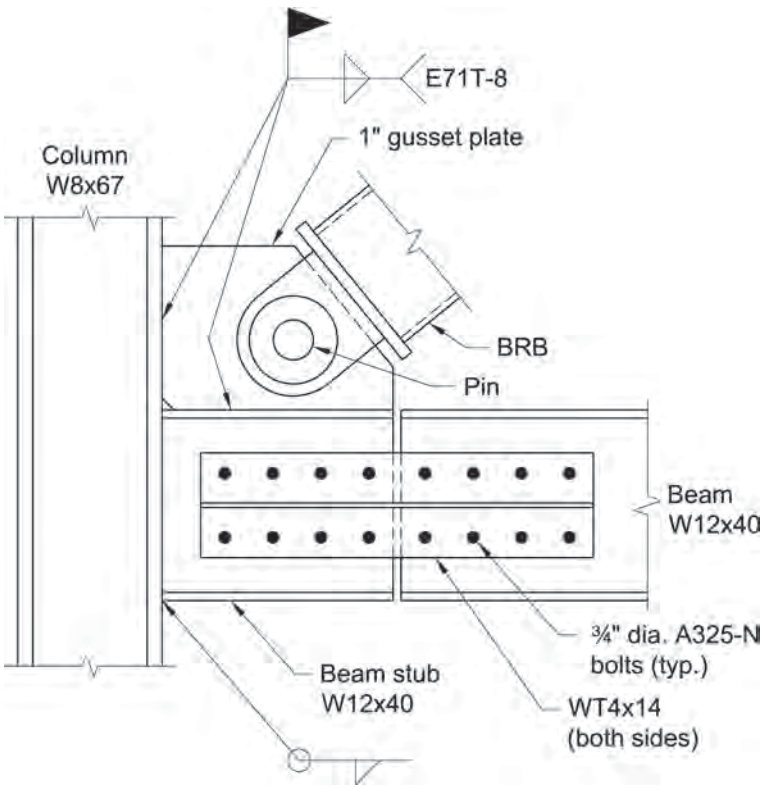


Fig. C-F4.6. Detail of connection with hinge (Fahnestock et al., 2006).

HBE to the total system hysteretic energy. In past research (Driver et al., 1997), the yielding of boundary elements contributed approximately 25 to 30% of the total load strength of the system. However, that contribution will vary as a function of the web aspect ratio (Qu and Bruneau, 2009).

With the exception of plastic hinging at the ends of HBE, the surrounding HBE and VBE are designed to remain essentially elastic when the webs are fully yielded. Plastic hinging at the base of VBE (when VBE are connected to foundations in a way that makes it possible to develop their plastic moment) and at the ends of HBE are needed to develop the plastic collapse mechanism of this system. Plastic hinging within the span of HBE, which could partly prevent yielding of the webs, is undesirable as it can result in: (1) significant accumulation of plastic incremental deformations on the HBE; (2) partial yielding of the infill plates; (3) correspondingly lower global plastic strength, and (4) total (elastic and plastic) HBE rotations equal to twice the values that develop when in-span hinging is prevented (Purba and Bruneau, 2012). Some designers have used reduced beam section (RBS) connections at the ends of HBE to ensure that yielding occurs only at the RBS. Location and strength of RBS plastic hinges in HBE differ from those typically calculated for special moment frames (SMF), and these should be established using equations developed for this purpose (Qu and Bruneau, 2010a, 2011; Bruneau et al., 2011).

Cases of both desirable and undesirable yielding in VBE have been observed in past testing. In the absence of a theoretical formulation to quantify the conditions leading

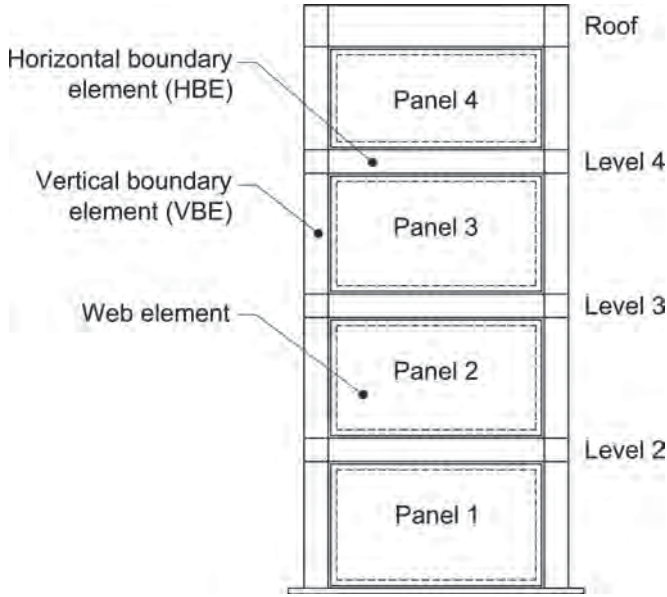


Fig. C-F5.1. Schematic of special plate shear wall.

to acceptable yielding (and supporting experimental validation of this formulation), the conservative requirement of elastic VBE response is justified.

Research literature often compares the behavior of SPSW to that of a vertical plate girder, indicating that the webs of an SPSW resist shears by tension field action and that the VBE of an SPSW resist overturning moments. While this analogy is useful in providing a conceptual understanding of the behavior of SPSW, many significant differences exist in the behavior and strength of the two systems. Past research shows that the use of structural shapes for the VBE and HBE in SPSW (as well as other dimensions and details germane to SPSW) favorably impacts orientation of the angle of development of the tension field action, and makes possible the use of very slender webs (having negligible diagonal compressive strength). Sizeable top and bottom HBE are also required in the SPSW to anchor the significant tension fields that develop at the ends of the structural system. Limits imposed on the maximum web slenderness of plate girders to prevent flange buckling, or due to transportation requirements, are also not applicable to SPSW which are constructed differently. For these reasons, the use of beam design provisions in the *Specification* for the design of SPSW is not appropriate (Berman and Bruneau, 2004).

3. Analysis

Incremental dynamic analyses in compliance with FEMA P-695 procedures (FEMA, 2009b) have demonstrated that SPSW designed by distributing the applied story shear force between the webs and their boundary frame do not have a satisfactory margin of safety against collapse and have a high probability of developing excessive drifts (Purba and Bruneau, 2014a, 2014b, 2014c), contrary to SPSW having webs designed to resist the entire code-specified story shears.

An additional and unrelated requirement specifies that the strength of the frame consisting of VBE and HBE shall be at least 25% of the story shear force distributed to the SPSW. This requirement is to ensure the presence of a minimum boundary frame, to prevent excessive drifts, given that the boundary frame alone resists seismic forces until dynamic response excites the SPSW to drifts that exceed previously reached maximum values. Shake table tests by Dowden and Bruneau (2014) illustrated how SPSW with weak boundary frames can develop substantially greater drifts when subjected to identical earthquake excitations but after prior yielding of the infill plate. Although post-tensioned self-centering frames were used in that study, SPSW having weak boundary frames would behave similarly, but worse, without the benefit of self-centering capabilities.

Per capacity design principles, all edge boundary elements (HBE and VBE) shall be designed to resist the maximum forces developed by the tension field action of the webs fully yielding. Axial forces, shears and moments develop in the boundary elements of the SPSW as a result of the response of the system to the overall overturning and shear, and this tension field action in the webs. Actual web thickness must be considered for this calculation, because webs thicker than required may have to be used due to availability, or minimum thickness required for welding.

At the top panel of the wall, the vertical components of the tension field should be anchored to the HBE. The HBE shall have sufficient strength to allow development of full tensile yielding across the panel width.

At the bottom panel of the wall, the vertical components of the tension field should also be anchored to the HBE. The HBE should have sufficient strength to allow development of full tensile yielding across the panel width. This may be accomplished by continuously anchoring the HBE to the foundation.

For intermediate HBE of the wall, the anticipated variation between the top and bottom web normal stresses acting on the HBE is usually small, or null when webs in the panel above and below the HBE have identical thickness. While top and bottom HBE are typically of substantial size, intermediate HBE are relatively smaller.

For the design of HBE, it may be important to recognize the effect of vertical stresses introduced by the tension field forces in reducing the plastic moment of the HBE. Concurrently, free-body diagrams of HBE should account for the additional shear and moments introduced by the eccentricity of the horizontal component of the tension fields acting at the top and bottom of the HBE (Qu and Bruneau, 2008, 2010a).

Forces and moments in the members (and connections), including those resulting from tension field action, may be determined from a plane frame analysis. The web is represented by a series of inclined pin-ended strips, as described in Commentary Section F5.5b. A minimum of ten equally spaced pin-ended strips per panel should be used in such an analysis.

A number of analytical approaches are possible to achieve capacity design and determine the same forces acting on the vertical boundary elements. Some example methods applicable to SPSW follow. In all cases, actual web thickness should be considered.

Nonlinear pushover analysis. A model of the SPSW can be constructed in which bilinear elasto-plastic web elements of strength $R_yF_yA_s$ are introduced in the direction α. Bilinear plastic hinges can also be introduced at the ends of the horizontal boundary elements. Standard pushover analysis conducted with this model will provide axial forces, shears and moments in the boundary frame when the webs develop yielding. Separate checks are required to verify that plastic hinges do not develop in the horizontal boundary elements, except at their ends.

Indirect capacity design approach. The Canadian Standards Association *Limit States Design of Steel Structures* (CSA, 2001), proposes that loads in the vertical boundary members can be determined from the gravity loads combined with the seismic loads increased by the amplification factor,

$$B = \frac{V_e}{V_u} \qquad \text{(C-F5-1)}$$

where

V_e = expected shear strength, at the base of the wall, determined for the web thickness supplied, kips (N)
$= 0.5R_yF_yt_wL\sin 2\alpha$

V_u = factored lateral seismic force at the base of the wall, kips (N)

In determining the loads in VBE, the amplification factor, B, need not be taken as greater than the seismic response modification coefficient, R.

The VBE design axial forces shall be determined from overturning moments defined as follows:

(1) The moment at the base is BM_u, where M_u is the factored seismic overturning moment at the base of the wall corresponding to the force V_u

(2) The moment BM_u extends for a height H but not less than two stories from the base

(3) The moment decreases linearly above a height H to B times the overturning moment at one story below the top of the wall, but need not exceed R times the factored seismic overturning moment at the story under consideration corresponding to the force V_u

The local bending moments in the VBE due to tension field action in the web should be multiplied by the amplification factor B.

This method is capable of producing reasonable results for approximating VBE capacity design loads; however, as described previously, it can be unconservative as shown in Berman and Bruneau (2008c). This procedure relies on elastic analysis of a strip model (or equivalent) for the design seismic loads, followed by amplification of the resulting VBE moments by the factor B. Therefore, it produces moment diagrams and SPSW deformations that are similar in shape to those obtained from a pushover analysis. Similarly, the determination of VBE axial forces from overturning calculations based on the design lateral loads amplified by B results in axial force diagrams that are of the proper shape. However, following the above procedure, the amplification factor is found only for the first story and does not include the possibly significant strength of the surrounding frame. HBE and VBE for SPSW are large and the portion of the base shear carried by the surrounding moment frame can be substantial. As a result, estimates of VBE demands per this method are less than those required to develop full web yielding on all stories prior to development of hinges in VBE. In addition, in some cases, the ratio of web thickness provided to web thickness needed for the design seismic loads can be larger on the upper stories than on the lower stories. In these situations, the indirect capacity design approach would underestimate the VBE design loads for the upper stories and capacity design would not be achieved. Neglecting these effects in the determination of B will result in VBE design loads that are underestimated for true capacity design. Therefore, the full collapse mechanism should be used when determining the factor B. Such an equation is proposed in the following procedure (in Equation C-F5-15).

Combined Plastic and Linear Analysis. This procedure has been shown to give accurate VBE results compared to pushover analysis (Berman and Bruneau, 2008c). Assuming that the web plates and HBE of a SPSW have been designed according to the Provisions to resist the factored loads (or, for the case of HBE design, the maximum of the factored loads or web plate yielding), the required capacity of VBE may be found from VBE free body diagrams such as those shown in Figure C-F5.2 for a generic four-story SPSW. Those free body diagrams include distributed loads representing the web plate yielding at story i, ω_{xci} and ω_{yci}; moments from plastic hinging of HBE, M_{prli} and M_{prri}; axial forces from HBE, P_{bli} and P_{bri}; applied lateral seismic loads, found from consideration of the plastic collapse mechanism, F_i; and base reactions for those lateral seismic loads, R_{yl}, R_{xl}, R_{yr} and R_{xr}. Each of these loads can then be determined as follows:

(1) The distributed loads to be applied to the VBE (ω_{yci} and ω_{xci}) and HBE (ω_{ybi} and ω_{xbi}) from plate yielding on each story, i, may be determined as:

$$\omega_{yci} = (½)F_{yp}t_{wi}\sin 2\alpha \qquad \text{(C-F5-2)}$$

$$\omega_{xci} = F_{yp}t_{wi}(\sin \alpha)^2 \qquad \text{(C-F5-3)}$$

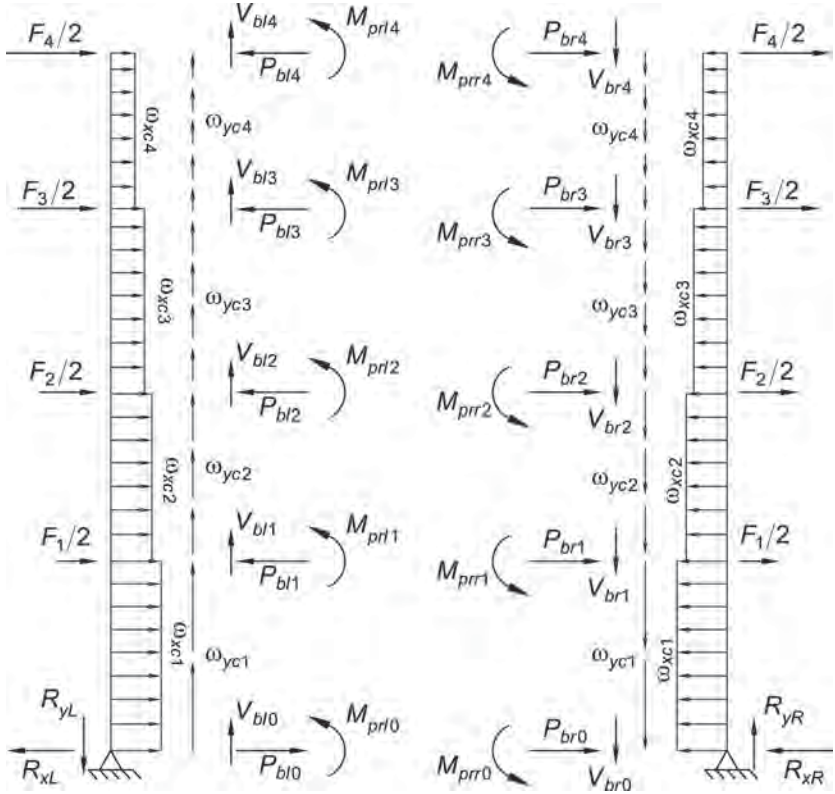


Fig. C-F5.2. VBE free body diagrams.

$$\omega_{ybi} = F_{yp} t_{wi} (\cos \alpha)^2 \quad \text{(C-F5-4)}$$

$$\omega_{xbi} = (1/2) F_{yp} t_{wi} \sin 2\alpha \quad \text{(C-F5-5)}$$

where R_y and F_y are for the web plate material and t_{wi} is the web thickness at level i, respectively.

(2) As part of estimating the axial load in the HBE, an elastic model of the VBE is developed as shown in Figure C-F5.3. The model consists of a continuous beam element representing the VBE which is pin-supported at the base and supported by elastic springs at the intermediate and top HBE locations. HBE spring stiffnesses at each story i, k_{bi}, can be taken as the axial stiffness of the HBE considering one half of the bay width (or HBE length for a considerably deep VBE), i.e.:

$$k_{bi} = \frac{A_{bi} E}{L/2} \quad \text{(C-F5-6)}$$

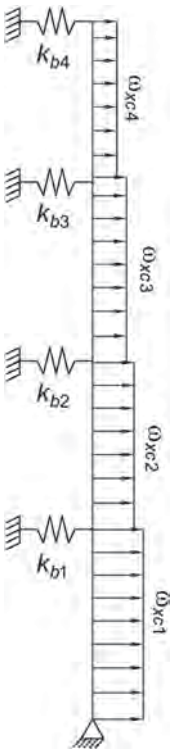


Fig. C-F5.3. Elastic VBE model with HBE springs.

where A_{bi} is the HBE cross-sectional area, L is the bay width, and E is the modulus of elasticity. This VBE model is then loaded with the horizontal component of the forces from the web plates yielding over each story, namely, ω_{xci}, and analysis return spring forces, P_{si}.

(3) The axial force component in the intermediate and top HBE resulting from the horizontal component of the plate yield forces on the HBE, ω_{xbi}, is assumed to be distributed as shown in Figure C-F5.4. Note that for the bottom HBE, this distribution is the reverse of that in the top beam. These axial force components are then combined with the spring forces from the linear VBE model, resulting in the following equations for the axial force at the left and right sides of the intermediate and top HBE (P_{bli} and P_{bri}, respectively):

$$P_{bli} = -(\omega_{xbi} - \omega_{xbi+1})\frac{L}{2} + P_{si} \quad \text{(C-F5-7)}$$

$$P_{bri} = (\omega_{xbi} - \omega_{xbi+1})\frac{L}{2} + P_{si} \quad \text{(C-F5-8)}$$

where the spring forces, P_{si}, should be negative indicating that they are adding to the compression in HBE. As mentioned previously, the axial forces from ω_{xbi} and ω_{xbi+1} in the bottom HBE may be taken as the mirror image of those shown in Figure C-F5.4, where ω_{xbi} is zero in that particular case as there is no web below the bottom HBE. Furthermore, there are no spring forces to consider at the bottom HBE location as the horizontal component of force from web plate yielding on the lower portion of the bottom VBE is added to the base reaction determined as part of the plastic collapse mechanism analysis, as described below. Therefore, the bottom HBE axial forces on the right and left hand sides, P_{bl0} and P_{br0}, are:

$$P_{bl0} = \omega_{xb1}\frac{L}{2} \quad \text{(C-F5-9)}$$

$$P_{br0} = -\omega_{xb1}\frac{L}{2} \quad \text{(C-F5-10)}$$

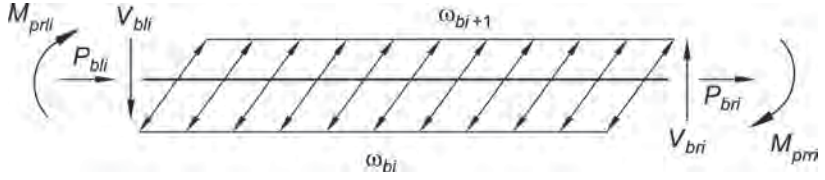


Fig. C-F5.4. HBE free body diagram.

(4) The reduced plastic moment capacity at the HBE ends can be approximated by:

$$\text{If } 1.18\left(1-\frac{|P_{bli}|}{F_{yb}A_{bi}}\right)\le 1.0$$

$$M_{prli}=1.18\left(1-\frac{|P_{bli}|}{F_{yb}A_{bi}}\right)Z_{xbi}F_{yb} \qquad \text{(C-F5-11)}$$

$$\text{If } 1.18\left(1-\frac{|P_{bli}|}{F_{yb}A_{bi}}\right)>1.0$$

$$M_{prli}=Z_{xbi}F_{yb} \qquad \text{(C-F5-12)}$$

where F_{yb} is the HBE expected yield strength multiplied by 1.1 to account for some strain hardening (i.e., $1.1R_yF_y$), A_{bi} is the HBE cross-sectional area for story i, and Z_{xbi} is the HBE plastic section modulus for story i.

(5) The shear forces at the left and right ends of all HBE, V_{br} and V_{bl}, can be found from:

$$V_{bri}=\frac{M_{prri}+M_{prli}}{L}+\left(\omega_{ybi}-\omega_{ybi+1}\right)\frac{L}{2} \qquad \text{(C-F5-13)}$$

$$V_{bli}=V_{bri}-\left(\omega_{ybi}-\omega_{ybi+1}\right)L \qquad \text{(C-F5-14)}$$

(6) The applied loads for the SPSW collapse mechanism can be found from:

$$\sum_{i=1}^{n_s}F_iH_i=\sum_{i=0}^{n_s}M_{prli}+\sum_{i=0}^{n_s}M_{prri}+\sum_{i=1}^{n_s}\frac{1}{2}\left(t_{wi}-t_{wi+1}\right)F_{yp}LH_i\sin\left(2\alpha_i\right) \qquad \text{(C-F5-15)}$$

where F_i is the applied lateral load at each story to cause the mechanism, H_i is the height from the base to each story, and other terms are as previously defined. Note that the indices for the HBE plastic moment summations begin at zero so that the bottom HBE (denoted HBE_0) is included. To employ Equation C-F5-15 in calculating the applied lateral loads that cause this mechanism to form, it is necessary to assume some distribution of those loads over the height of the structure, i.e., a relationship between F_1, F_2, etc. For this purpose, a pattern equal to that of the design lateral seismic loads from the appropriate building code may be used.

(7) Horizontal reactions at the column bases, R_{xL} and R_{xR}, are then determined by dividing the collapse base shear by 2 and adding the pin-support reaction from the VBE model, R_{bs}, to the reaction under the left VBE and subtracting it off the reaction under the right VBE. Vertical base reactions can be estimated from overturning calculations using the collapse loads as:

$$R_{yl}=\frac{\sum_{i=1}^{n_s}F_iH_i}{L} \qquad \text{and} \qquad R_{yr}=-R_{yl} \qquad \text{(C-F5-16)}$$

(8) The moment, axial and shear force diagrams for the VBE are established once all the components of the VBE free body diagrams are estimated. The diagrams give minimum design actions for those VBE such that they can resist full web plate yielding and HBE hinging.

VBE should be designed to remain elastic under the large shears resulting from this analysis. Existing literature shows instances of undesirable inelastic behavior when shear yielding occurred in the VBE (Qu and Bruneau, 2008; Qu and Bruneau, 2010b).

Preliminary Design. For preliminary proportioning of HBE, VBE and webs, an SPSW wall may be approximated by a vertical truss with tension diagonals. Each web is represented by a single diagonal tension brace within the story. For an assumed angle of inclination of the tension field, the web thickness, t_w, may be taken as

$$t_w = \frac{2A\Omega_s \sin\theta}{L \sin 2\alpha} \qquad \text{(C-F5-17)}$$

where

A = area of the equivalent tension brace, in.^{2} (mm^2)
θ = angle between the vertical and the longitudinal axis of the equivalent diagonal brace
L = distance between VBE centerlines, in. (mm)
α = assumed angle of inclination of the tension field measured from the vertical per Section F5.5a
Ω_s = system overstrength factor, as defined by FEMA 369 (FEMA, 2001), and taken as 1.2 for SPSW (Berman and Bruneau, 2003b)

A is initially estimated from an equivalent brace size to meet the structure's drift requirements.

4. System Requirements

Panel Aspect Ratio. The 2005 Provisions for the design of special plate shear walls (SPSW) limited their applicability to wall panels having aspect ratios of $0.8 < L/h \leq 2.5$. This limit was first introduced in the 2003 Edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures,* FEMA 450 (FEMA, 2003), as a most conservative measure in light of the relatively limited experience with that structural system in the U.S. at the time. Since then, SPSW designed in compliance with the Provisions and having lower aspect ratios have been observed to perform satisfactorily. For example, SPSW specimens having L/h of 0.6 (Lee and Tsai, 2008) exhibited ductile hysteretic behavior comparable to that of walls with larger aspect ratios.

No theoretical upper bound exists on L/h, but as the SPSW aspect ratio increases, progressively larger HBE will be required, driven by the capacity design principles embodied in the design requirements. This will create a de facto practical limit beyond which SPSW design will become uneconomical and impractical, and no arbitrary

limit (such as 2.5) needs to be specified provided the engineer ensures that all strips yield at the target drift response (Bruneau and Bhagwagar, 2002).

Past research has focused on walls with an L/t_w ratio ranging from 300 to 800. Although no theoretical upper bound exists on this ratio, drift limits will indirectly constrain this ratio. The requirement that webs be slender provides a lower bound on this ratio. For these reasons, no limits are specified on that ratio.

4a. Stiffness of Boundary Elements

The stiffness requirement in the 2005 and 2010 Provisions was originally intended to prevent excessive in-plane flexibility and buckling of VBE. However, subsequent research showed that the specified limits on stiffness were uncorrelated to satisfactory in-plane and out-of-plane VBE performance, and that stiffer boundary elements principally served to ensure full yielding of the webs at smaller drifts (Qu and Bruneau, 2010b). It was also experimentally demonstrated that SPSW having VBE stiffness exceeding these prescribed limits could perform satisfactorily (Lee and Tsai, 2008). The stiffness limits provided in Section F5.4a can be expedient to design boundary elements with adequate stiffness to develop full yielding of the webs at the design drift. The engineer may also demonstrate by other methods, such as pushover analysis, that this design objective is attained.

4c. Bracing

Providing stability of SPSW system boundary elements is necessary for proper performance of the system. Past experience has shown that SPSW can behave in a ductile manner with beam-to-column requirements detailed in accordance with intermediate moment frame requirements. As such, lateral bracing requirements are specified to meet the requirements for moderately ductile members. In addition, all intersections of HBE and VBE must be braced to ensure stability of the entire panel.

4d. Openings in Webs

Large openings in webs create significant local demands and thus must have HBE and VBE in a similar fashion as the remainder of the system. When openings are required, SPSW can be subdivided into smaller SPSW segments by using HBE and VBE bordering the openings. With the exception of the structural systems described in Section F5.7, SPSW with holes in the web not surrounded by HBE/VBE have not been tested. The provisions will allow other openings that can be justified by analysis or testing.

5. Members

5a. Basic Requirements

Dastfan and Driver (2008) demonstrated that the strength of SPSW designed in compliance with current requirements is not substantially sensitive to the angle of inclination of the strips, and that using a single value of 40° throughout the design will generally lead to slightly conservative results.

Some amount of local yielding is expected in the HBE and VBE to allow the development of the plastic mechanism of SPSW systems. For that reason, HBE and VBE comply with the requirements in Table D1.1 for SMF.

5b. Webs

The lateral shears are carried by tension fields that develop in the webs stressing in the direction α, defined in Section F5.5b. When the HBE and VBE boundary elements of a web are not identical, the average of HBE areas may be taken in the calculation of A_b, and the average of VBE areas and inertias may be respectively used in the calculation of A_c and I_c to determine α.

The plastic shear strength of panels is given by $0.5R_yF_yt_wL_{cf}(\sin 2\alpha)$. The nominal strength is obtained by dividing this value by a system overstrength, as defined by FEMA 369 (FEMA, 2003), and taken as 1.2 for SPSW (Berman and Bruneau, 2003b).

The plastic shear strength is obtained from the assumption that, for purposes of analysis, each web may be modeled by a series of equally spaced inclined pin-ended strips (Figure C-F5.5), oriented at angle α. Past research has shown that this model provides realistic results, as shown in Figure C-F5.6 for example, provided that at least 10 equally spaced strips are used to model each panel.

The specified minimum yield stress of steel used for SPSW is per Section A3.1. However, the webs of SPSW could also be of special highly ductile low yield steel having specified minimum yield in the range of 12 to 33 ksi (80 to 230 MPa).

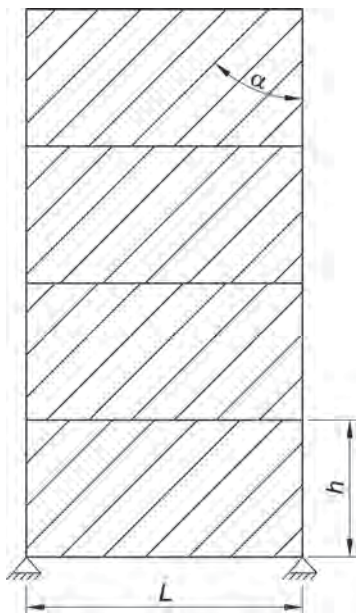


Fig. C-F5.5. Strip model of an SPSW.

5c. HBE

Purba and Bruneau (2012, 2014a) demonstrated that plastic hinging within the span of HBE can produce excessive accumulation of plastic incremental deformations on the HBE, as well as partial yielding of the infill plates and correspondingly lower global plastic strength. Section F5.5c offers two design approaches to prevent in-span HBE plastic hinges:

(1) Provide an HBE plastic section modulus equal to

$$Z_i = \frac{\omega_{ybi} L_b^2}{4 F_{yb}} \qquad \text{(C-F5-18)}$$

where L_b and F_{yb} are HBE span and yield stress, respectively; and ω_{ybi} is the vertical component of infill plate stress, defined as

$$\omega_{ybi} = F_{yp} t_{pi} \cos^2 \alpha \qquad \text{(C-F5-19)}$$

where F_{yp} and t_{pi} are the infill plate yield stress and the infill thickness, respectively, and α is the tension field inclination angle. This is equivalent to designing the HBE to resist a moment equal to $\frac{\omega_{ybi} L_b^2}{4}$.

(2) Use reduced beam sections (RBS) at the ends of HBE to ensure plastic hinging develops only at the RBS. Note that location and strength of RBS plastic hinges

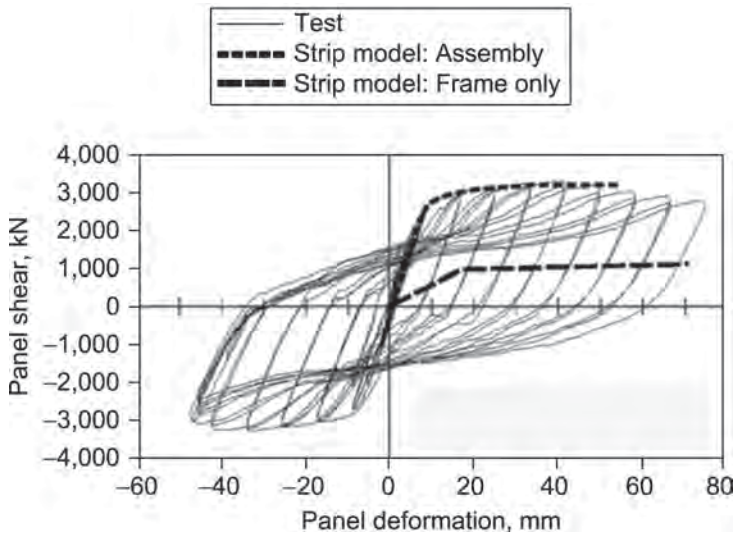


Fig. C-F5.6. Comparison of experimental results for lower panel of multi-story SPSW frame and strength predicted by strip model (after Driver et al., 1997).

in HBE differ from those typically calculated for special moment frames, and these should be established using equations developed for this purpose (Qu and Bruneau, 2010a, 2011; Bruneau et al., 2011).

Further details on these two design approaches are provided in Vian and Bruneau (2005).

5d. Protected Zone

Parts of SPSW expected to develop large inelastic deformations, and their connections, are designated as protected zones to meet the requirements of Section D1.3.

6. Connections

6a. Demand Critical Welds

Demand critical welds are required per Section A3.4b consistently with similar requirements for all SFRS.

6b. HBE-to-VBE Connections

Due to the large initial stiffness of SPSW, total system drift and plastic hinge rotation demands at the ends of HBE are anticipated to be smaller than for special moment frames. The requirements of Section E2.6b for intermediate moment frames (IMF) are deemed adequate for HBE-to-VBE connections.

1. Required Strength

Connections of the HBE to VBE shall be able to develop the plastic strength of the HBE given that plastic hinging is expected at the ends of HBE.

2. Panel Zones

Panel zone requirements are not imposed for intermediate HBE where generally small HBE connect to sizeable VBE. The engineer should use judgment

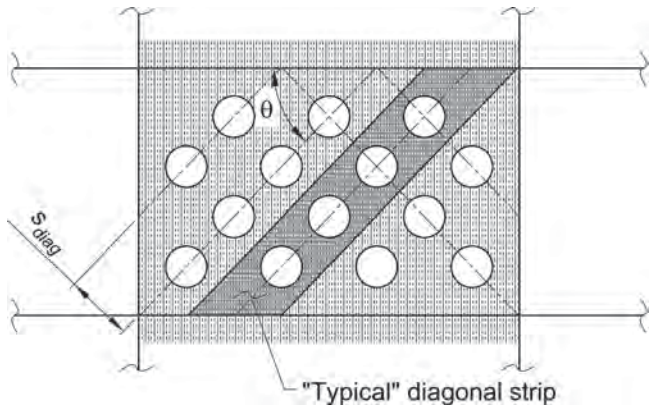


Fig. C-F5.7. Schematic detail of special perforated steel plate wall and typical diagonal strip.

to identify special situations in which the panel zone adequacy of VBE next to intermediate HBE should be verified.

6c. Connections of Webs to Boundary Elements

Web connections to the surrounding HBE and VBE are required to develop the expected tensile strength of the webs. Net sections must also provide this strength for the case of bolted connections.

The strip model can be used to model the behavior of SPSW and the tensile yielding of the webs at angle, α. A single angle of inclination taken as the average for all the panels may be used to analyze the entire wall. The expected tensile strength of the web strips shall be defined as $R_y F_y A_s$,
where

A_s = area of a strip = $(L \cos\alpha + H \sin\alpha)/n$, in.^{2} (mm^2)
L = width of panel, in. (mm)
H = height of panel, in. (mm)
n = number of strips per panel; taken greater than or equal to 10

This analysis method has been shown, through correlation with physical test data, to adequately predict SPSW performance. It is recognized, however, that other advanced analytical techniques [such as the finite element method (FEM)] may also be used for design of SPSW. If such nonlinear (geometric and material) FEM models are used, they should be calibrated against published test results to ascertain reliability for application. Designs of connections of webs to boundary elements should also anticipate buckling of the web plate. Some minimum out-of-plane rotational restraint of the plate should be provided (Caccese et al., 1993).

6d. Column Splices

The importance of ensuring satisfactory performance of column splices is described in Commentary Section D2.5.

7. Perforated Webs

7a. Regular Layout of Circular Perforations

Special perforated steel plate walls (SPSPW) are a special case of SPSW in which a special panel perforations layout is used to allow utilities to pass through and which may be used to reduce the strength and stiffness of a solid panel wall to levels required in a design when a thinner plate is unavailable. This concept has been analytically and experimentally proven to be effective and the system remains ductile up to the drift demands corresponding to severe earthquakes (Vian and Bruneau, 2005; Vian et al., 2009a; Vian et al., 2009b; Purba and Bruneau, 2007). A typical hole layout for this system is shown in Figure C-F5.7, for a case having four horizontal lines of holes, and seven vertical lines of holes. The design equations provided in Section F5.7a have been validated for webs having at least four horizontal and vertical lines of holes.

Note that while general equations could be derived for lines of holes aligned at any angle from the horizontal, Equation F5-3 is applicable only to the special case of holes that align diagonally at 45° from the horizontal because it is was deemed to be the simplest and most practical configuration, and because it is the only orientation that has been considered while developing Equation F5-3 (Purba and Bruneau, 2007). As shown in Figure C-F5.7, perforating webs in accordance with this section result in the development of web yielding in a direction parallel to that of the holes alignment. As such, Equation F5-2 is not applicable for perforated steel plate shear walls.

Designing SPSW in low- to medium-rise buildings using hot-rolled steel often results in required panel thicknesses less than the minimum plate thickness available from steel producers. In such cases, using the minimum available thickness would result in large panel force over-strength, proportionally larger design demands on the surrounding VBE and HBE, and an overall less economical system. Attempts at alleviating this problem were addressed by the use of light-gauge, cold-formed steel panels (Berman and Bruneau, 2003a, 2005b). SPSPW instead reduce the strength of the web by adding to it a regular grid of perforations. This solution simultaneously helps address the practical concern of utility placement across SPSW. In a regular SPSW, the infill panel which occupies an entire frame bay between adjacent HBE and VBE is a protected element, and utilities that may have otherwise passed through at that location must either be diverted to another bay, or pass through an opening surrounded by HBE and VBE. This either results in additional materials (for the extra stiffening) or in labor (for the relocation of ductwork in a retrofit, for example). SPSPW provide a more economical alternative.

7b. Reinforced Corner Cut-Out

It is also possible to allow utility passage through a reinforced cutout designed to transmit the web forces to the boundary frame. While providing utility access, this proposed system provides strength and stiffness similar to a solid panel SPSW system. The openings are located immediately adjacent to the column in each of the top corners of the panel, a location where large utilities are often located. A cut-out radius as large as 19.6 in. (490 mm) for a half-scale specimen having a 6.5 ft (2 m) center-to-center distance between HBE has been successfully verified experimentally and analytically by Vian and Bruneau (2005) and Purba and Bruneau (2007).

Forces acting in the reinforcing arch (the curved plate at the edge of the opening) are a combination of effects due to arching action under tension forces due to web yielding, and thrusting action due to change of angle at the corner of the SPSW (Figures C-F5.8 and C-F5.9). The latter is used to calculate the required maximum thickness of the "opening" corner arch (top left side of Figure C-F5.8, with no web stresses assumed to be acting on it). The arch plate width is not a parameter that enters the solution of the interaction equation in that calculation, and it is instead conservatively obtained by considering the strength required to resist the axial component of force in the arch due to the panel forces at the closing corner (top right side of Figure C-F5.8). Since the components of arch forces due to panel forces are opposing those due to frame corner opening (Figure C-F5.9), the actual forces acting in the arch plate will be

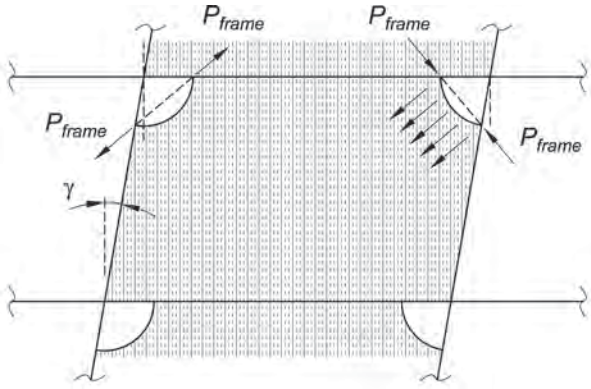


Fig. C-F5.8. Arch end reactions due to frame deformations, and infill panel forces on arches due to tension field action on reinforced cut-out corner.

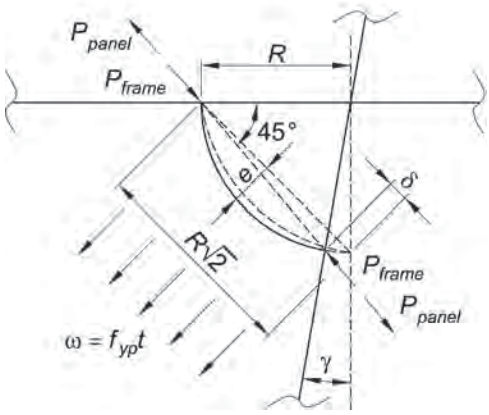


Fig. C-F5.9. Deformed configurations and forces acting on right arch.

smaller than the forces calculated by considering the components individually as is done previously for design.

Note that when a plate in the plane is added to the reinforcement arch to facilitate infill panel attachment to the arch in the field, it results in a stiffer arch section that could (due to compatibility of frame corner deformation) partly yield at large drifts. However, Vian and Bruneau (2005) and Purba and Bruneau (2007) showed that the thickness of the flat plate selected per the above procedure is robust enough to withstand the loads alone, and that the presence of the stiffer and stronger T section (due to the attachment plate discussed above) is not detrimental to the system performance.

Nonlinear static pushover analysis is a tool that can be used to confirm that the selected reinforcement section will not produce an undesirable "knee-brace effect" or precipitate column yielding or beam yielding outside of the hinge region.

CHAPTER G

COMPOSITE MOMENT-FRAME SYSTEMS

G1. COMPOSITE ORDINARY MOMENT FRAMES (C-OMF)

2. Basis of Design

Composite ordinary moment frames (C-OMF) represent a type of composite moment frame that is designed and detailed following the *Specification* and ACI 318 (ACI, 2014), excluding Chapter 18. ASCE/SEI 7 (ASCE, 2016) limits C-OMF to seismic design categories A and B. This is in contrast to steel ordinary moment frames (OMF), which are permitted in higher seismic design categories. The design requirements for C-OMF recognize this difference and provide minimum ductility in the members and connections. The R and C_d values for C-OMF are chosen accordingly.

G2. COMPOSITE INTERMEDIATE MOMENT FRAMES (C-IMF)

2. Basis of Design

ASCE/SEI 7 limits the use of composite intermediate moment frames (C-IMF) in seismic design category C through F. The provisions for C-IMF, as well as the associated R and C_d values in ASCE/SEI 7, are comparable to those required for reinforced concrete IMF and between those for steel intermediate moment frames (IMF) and OMF.

The inelastic drift capability of C-IMF is permitted to be derived from inelastic deformations of beams, columns and panel zones. This is more permissive than the design requirements for composite special moment frames (C-SMF) as defined in Section G3, which are intended to limit the majority of the inelastic deformation to the beams.

The C-IMF connection is based on a tested design with a qualifying story drift angle of 0.02 rad.

4. System Requirements

4a. Stability Bracing of Beams

The requirement for spacing of lateral bracing in this section is less severe than that for C-SMF in Section G3.4b because of the lower required drift angle for C-IMF as compared to C-SMF. In this case, the required spacing of bracing is approximately double that of the C-SMF system.

5. Members

5a. Basic Requirements

This section refers to Section D1.1, which provides requirements for moderately ductile members. Because the rotational demands on C-IMF beams and columns are

expected to be lower than C-SMF, the requirements and limitations for C-IMF members are less severe than for C-SMF.

5b. Beam Flanges

For relevant commentary on changes in cross section of beam flanges, see Commentary Section E3.5b.

5c. Protected Zones

For commentary on protected zones, see Commentary Section D1.3.

6. Connections

6a. Demand Critical Welds

There are no demand critical welds in C-IMF members because the story drift angle is 0.02 rad, which is half the value for C-SMF members, and ASCE/SEI 7 limits the use of C-IMF in seismic design category C through F.

6b. Beam-to-Column Connections

The minimum story drift angle required for qualification of C-IMF connections is 0.02 rad, which is half the value for C-SMF members, reflecting the lower level of inelastic response that is anticipated in the system.

6c. Conformance Demonstration

The requirements for conformance demonstration for C-IMF connections are the same as for C-SMF connections, except that the required story drift angle is smaller. Refer to Commentary Section G3.6c.

6d. Required Shear Strength

The requirements for shear strength of the connection for C-IMF are comparable to those of SMF, with the exception that the calculation of the expected flexural strength must account for the different constituent materials. Refer to Commentary Section E3.6d.

6e. Connection Diaphragm Plates

Connection diaphragm plates are permitted for filled composite columns both external and internal to the column. These diaphragm plates facilitate the transfer of beam flange forces into the column panel zone. These plates are required to have (i) thickness at least equal to the beam flange, and (ii) complete-joint-penetration groove or two-sided fillet welds. They are designed with a required strength not less than the available strength of the contact area of the plate with column sides. Internal diaphragms are required to have a circular opening for placing concrete.

6f. Column Splices

The requirements for column splices for C-IMF are comparable to those of SMF, with the exception that the calculation of the expected flexural strength must account for the different constituent materials.

G3. COMPOSITE SPECIAL MOMENT FRAMES (C-SMF)

1. Scope

Composite special moment frames (C-SMF) 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 as a cost-effective alternative to frames with reinforced concrete floors (Griffis, 1992; Furlong, 1997; Viest et al., 1997).

2. Basis of Design

Based on ASCE/SEI 7, C-SMF are primarily intended for use in seismic design categories D, E and F. 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 and column bases. 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.

C-SMF are generally expected to experience significant inelastic deformation during a large seismic event. It is expected that most of the inelastic deformation will take place as rotation in beam "hinges" with limited inelastic deformation in the panel zone of the column. The beam-to-column connections for these frames are required to be qualified based on tests that demonstrate that the connection can sustain a story drift angle of at least 0.04 rad based on a specified loading protocol. Other provisions are intended to limit or prevent excessive panel zone distortion, failure of connectivity plates or diaphragms, column hinging, and local buckling that may lead to inadequate frame performance in spite of good connection performance.

C-SMF and C-IMF connection configurations and design procedures are based on the results of qualifying tests; the configuration of connections in the prototype structure must be consistent with the tested configurations. Similarly, the design procedures used in the prototype connections must be consistent with tested configurations.

4. System Requirements

4a. Moment Ratio

The strong-column weak-beam (SC/WB) mechanism implemented for composite frames is based on the similar concept for steel SMF. Refer to Commentary Section E3.4a for additional details and discussion. It is important to note that compliance with the SC/WB requirement and Equation G3-1 does not ensure that individual columns will not yield, even when all connection locations in the frame

comply. However, yielding of beams will predominate and the desired inelastic performance will be achieved in frames with members sized to meet the requirement of Equation G3-1.

Commentary Section E3.4a discusses the three exceptions to Equation E3-1. The same discussion applies here for Equation G3-1, with the exception that the axial force limit is $P_{rc} < 0.1P_c$, which is done to ensure ductile behavior of composite and reinforced concrete columns.

4b. Stability Bracing of Beams

For commentary on stability bracing of beams, see Commentary Section E3.4b.

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

The stability bracing requirements at beam-to-column connections are similar to those for unbraced connections in steel SMF. Composite columns are typically not susceptible to flexural-torsional buckling modes due to the presence of concrete. The requirements of Section E3.4c.2 are applicable because composite columns are susceptible to flexural buckling modes in the out-of-plane direction.

5. Members

5a. Basic Requirements

Reliable inelastic deformation for highly ductile members requires that width-to-thickness ratios be limited to a range that provides composite cross sections resistant to local buckling well into the inelastic range. Although the width-to-thickness ratio for compact elements in *Specification* Table I1.1 are sufficient to prevent local buckling before the onset of yielding, the available test data suggest that these limits are not adequate for the required inelastic deformations in C-SMF (Varma et al., 2002, 2004; Tort and Hajjar, 2004).

Encased composite columns classified as highly ductile members shall meet the additional detailing requirements of Sections D1.4b.1 and D1.4b.2 to provide adequate ductility. For additional details, refer to Commentary Section D1.4b.

Filled composite columns shall meet the additional requirements of Section D1.4c.

When the design of a composite beam satisfies Equation G3-2, 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.

5b. Beam Flanges

For relevant commentary on changes in cross section of beam flanges, see Commentary Section E3.5b.

5c. Protected Zones

For commentary on protected zones see Commentary Section D1.3.

6. Connections

While the Provisions permit the design of composite beams based solely upon the requirements in the *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 loads are calculated assuming large member ductility and toughness. In the absence of test data to support specific requirements in the Provisions, the following special measures should be considered in C-SMF: (1) implementation of an inspection and quality assurance plan to verify proper welding of steel headed stud anchors to the beams (see Sections A4.3 and Chapter J); and (2) use of additional steel headed stud anchors beyond those required in the *Specification* immediately adjacent to regions of the beams where plastic hinging is expected.

6a. Demand Critical Welds

For general commentary on demand critical welds see Commentary Section A3.4.

6b. Beam-to-Column Connections

Connections to Reinforced Concrete Columns. A schematic connection drawing for composite moment frames with reinforced concrete columns is shown in Figure C-D2.10 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, 1992). Numerous large-scale tests of this type of connection have been conducted in the United States and Japan under both monotonic and cyclic loading (e.g., Sheikh et al., 1989; Kanno and Deierlein, 1997; Nishiyama et al., 1990; Parra-Montesinos and Wight, 2000; Chou and Uang, 2002; Liang and Parra-Montesinos, 2004). 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-D2.10 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 in the immediate connection region near the column. Tests have shown that, of the many possible ways of strengthening the joint, face bearing plates (see Figure C-G3.1) and steel band plates (Figure C-G3.2) 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 published guidelines (e.g., Nishiyama et al., 1990; Parra-Montesinos and Wight, 2001). Note that while the scope of the 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; Parra-Montesinos et al., 2003).

Connections to Encased Columns. Prior research has been conducted on the cyclic performance of encased columns and their connections (e.g., Kanno and Deierlein, 1997). Connections between steel beams and encased composite columns (see

Figure C-G3.1) have been used and tested extensively in Japan. 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 reinforced concrete columns (ASCE, 1994). One disadvantage of this connection detail compared to the one shown in Figure C-D2.10 is that, like standard steel construction, the detail in Figure C-G3.1 requires welding of the beam flange to the steel column.

Connections to Filled Columns. Prior research has also been conducted on the cyclic performance of filled columns and their connections, and there has been substantial recent research to support design strategies (see Figure C-G3.3) (Azizinamini and Schneider, 2004; Ricles et al., 2004a; Herrera et al., 2008).

The results of these tests and the corresponding design details can be used to design the connections and prepare for the qualification according to Chapter K. For example,

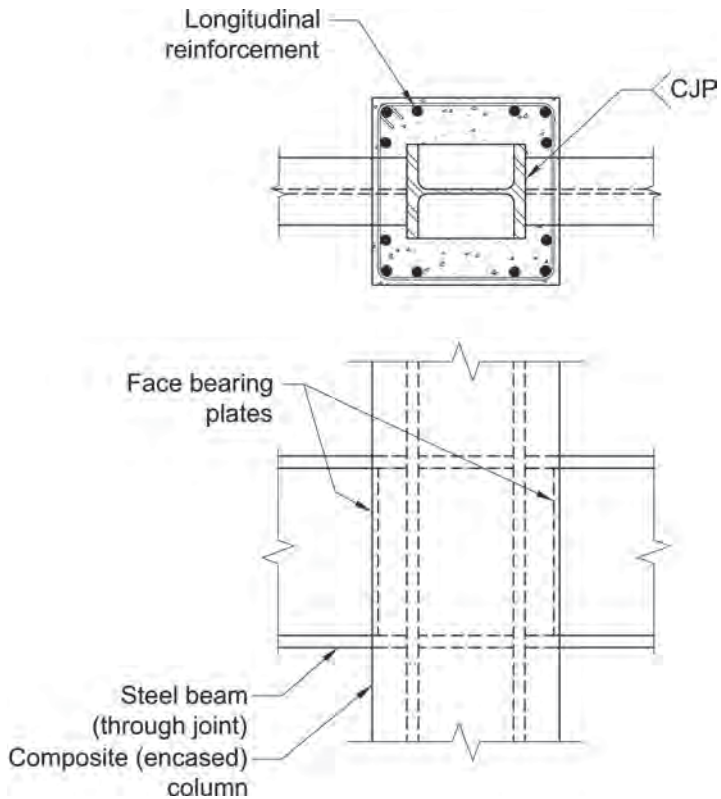


Fig. C-G3.1. Encased composite column-to-steel beam moment connection.

Figure C-G3.4 shows a large-scale filled composite column-to-steel beam connection that was tested by Ricles et al. (2004a) and demonstrated to exceed a story drift angle of 0.04 rad. In this same publication, the authors report test results for other large-scale filled composite column-to-beam connections that meet or exceed the story drift angle of 0.02 rad (for C-IMF) and 0.04 rad (for C-SMF).

For the special case where the steel beam runs continuously through the composite column, the internal load transfer mechanisms and behavior of these connections are similar to those for connections to reinforced concrete columns (Figure C-G3.2). Otherwise, where the beam is interrupted at the column face, special details are needed to transfer the column flange loads through the connection (Azizinamini and Schneider, 2004).

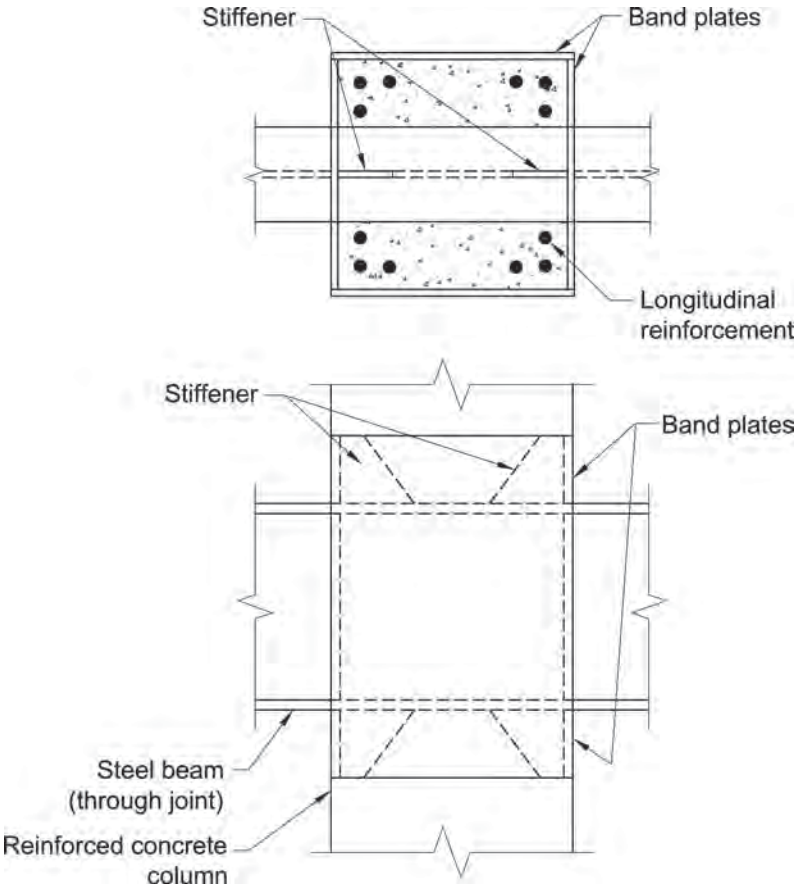


Fig. C-G3.2. Steel band plates used for strengthening the joint.

6c. Conformance Demonstration

The Provisions require that connections in C-SMF meet the same story drift capacity of 0.04 rad as required for steel SMF. Section G3.6c provides conformance demonstration requirements. This provision permits the use of connections qualified by prior tests or project specific tests. The engineer is responsible for substantiating the connection.

For the special case where beams are uninterrupted or continuous through composite or reinforced concrete columns, and beam flange welded joints are not used, the performance requirements shall be demonstrated through large-scale testing in accordance with Section K2, or other substantiating data available in the literature (e.g., Kanno and Deierlein, 1997; Nishiyama et al., 1990; Parra-Montesinos and Wight, 2001; Parra-Montesinos et al., 2003).

6d. Required Shear Strength

The requirements for shear strength of the connection for C-SMF are comparable to those of SMF, with the exception that the calculation of the expected flexural strength must account for the different constituent materials. See Commentary Section E3.6d.

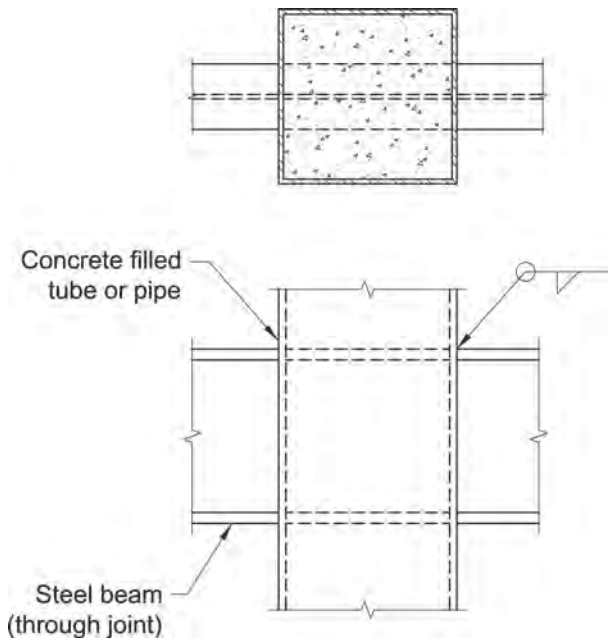


Fig. C-G3.3. Filled composite column-to-steel beam moment connection (beam flange uninterrupted).

6e. Connection Diaphragm Plates

The requirements for continuity plates and diaphragms are the same for C-SMF as for C-IMF. Refer to Commentary Section G2.6e.

6f. Column Splices

The requirements for column splices are the same for C-SMF as for C-IMF. Refer to Commentary Section G2.6f.

G4. COMPOSITE PARTIALLY RESTRAINED MOMENT FRAMES (C-PRMF)

1. Scope

Composite partially restrained moment frames (C-PRMF) consist of structural steel columns and composite steel beams, connected with partially restrained (PR) composite joints (Leon and Kim, 2004; Thermou et al., 2004; Zandonini and Leon, 1992). In PR composite joints, flexural resistance is provided by a couple incorporating a conventional steel bottom flange connection (welded or bolted plates, angles, or T-stubs) and the continuous reinforcing steel in the slab at the top of the girder (see

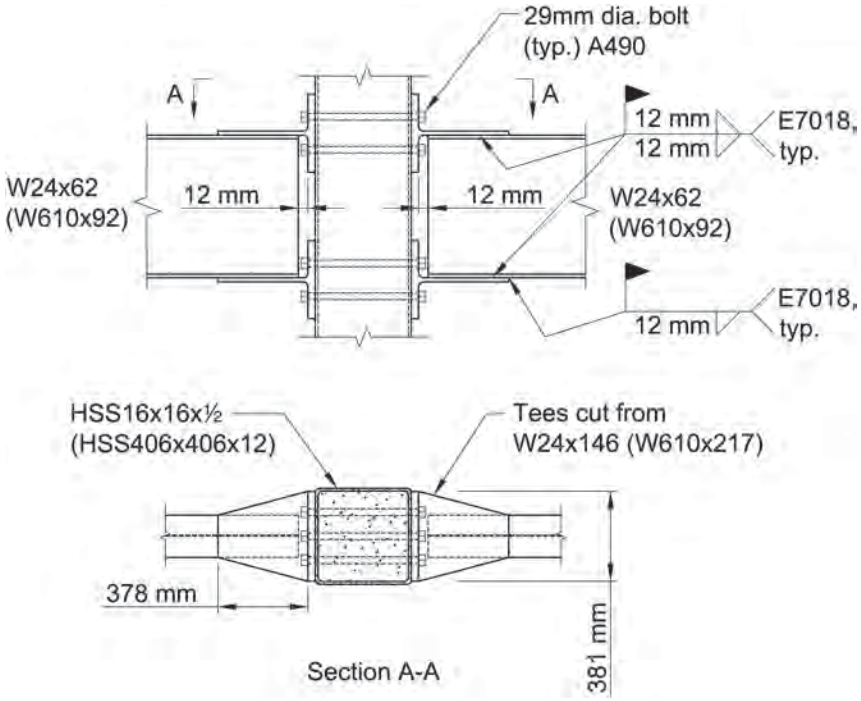


Fig. C-G3.4. Filled composite column-to-steel beam moment connection (beam flange interrupted).

Figure C-G4.1). The steel beam and the concrete slab are connected by steel anchors, such as headed anchor studs. Shear resistance is provided through a conventional steel frame shear connection (welded or bolted plates or angles). The use of the slab reinforcing steel results in a stronger and stiffer connection, a beneficial distribution of strength and stiffness between the positive and negative moment regions of the beams, and redistribution of loads under inelastic action. In most cases, the connections in this seismic force-resisting system at the roof level will not be designed as composite.

C-PRMF were originally proposed for areas of low to moderate seismicity in the eastern United States (seismic design categories A, B and C). 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 loads on partially restrained (PR) moment frames can be lower than those for fully restrained (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-Asl, 1992; DiCorso et al., 1989). Thus, in some cases, C-PRMF can be designed for lower seismic loads than ordinary moment frames (OMF).

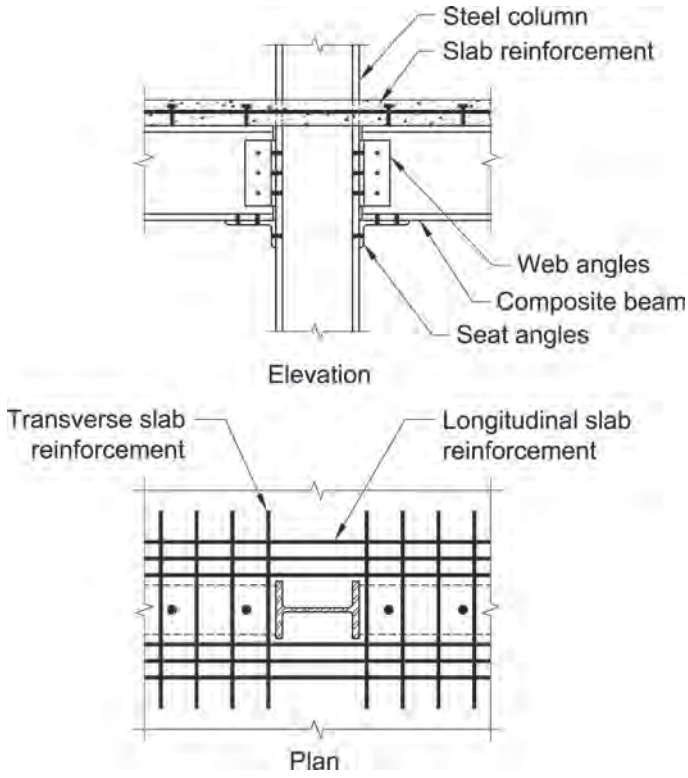


Fig. C-G4.1. Composite partially restrained connection.

2. Basis of Design

Design methodologies and standardized guidelines for composite partially restrained moment frames (C-PRMF) and connections have been published (Ammerman and Leon, 1990; Leon and Forcier, 1992; Leon et al., 1996; ASCE, 1998). In the design of PR composite connections, it is assumed that bending and shear forces can be considered separately.

3. Analysis

For frames up to four stories, the design of C-PRMF should be made using an analysis that, as a minimum, accounts for the partially restrained connection behavior of the connections by utilizing linear springs with reduced stiffness (Bjorhovde, 1984). The effective connection stiffness should be considered for determining member load distributions and deflections, calculating the building's period of vibration, and checking frame stability. Different connection stiffnesses may be required for these checks (Leon et al., 1996). Frame stability can be addressed using conventional procedures. However, the connection flexibility should be considered in determining the rotational restraint at the ends of the beams. For structures taller than four stories, drift and stability need to be carefully checked using analysis techniques that incorporate both geometric and connection nonlinearities (Rassati et al., 2004; Ammerman and Leon, 1990; Chen and Lui, 1991). 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 inaccuracies. Therefore, the use of a weighted average, as discussed in the Commentary to *Specification* Chapter I, is recommended (Zaremba, 1988; Ammerman and Leon, 1990; Leon and Ammerman, 1990; AISC, 2016a).

4. System Requirements

The system should be designed to enforce a strong column-weak beam mechanism except for the roof level. Leon et al. (1996) suggest using the following equation, analogous to Equation E3-1 for SMF, to achieve this behavior:

$$\sum \frac{M_{pc}^{*}}{M_{pb}^{*}} > 1.2 \qquad \text{(C-G4-1)}$$

where appropriate overstrength factors (typically 1.1 for the steel beams and 1.25 for the reinforcing bars) are incorporated into the M_{pb}^{*} calculation. The value of 1.2 instead of the 1.0 in Equation E3-1 is intended to ensure a weak beam-strong column mechanism, which Equation E3-1 does not (see Commentary to Section E3.4a).

5. Members

5a. Columns

Column panel zone checks per the *Specification* should be carried out assuming the connection moment is given by concentrated forces at the bottom flange and at the center of the concrete slab.

5b. Beams

Only fully composite beams are used in this system, as the effect of partial interaction in the composite beams has not been adequately justified. Because the force transfer relies on bearing of the concrete slab against the column flange, the bearing strength of the concrete should be checked. (See Figure C-G4.2.) The full nominal slab depth should be available for a distance of at least 12 in. (300 mm) from the column flange (see Figure C-G4.3).

6. Connections

The connecting elements should be designed with a yield force 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.

6c. Beam-to-Column Connections

Most PR connections do not exhibit a simple elasto-plastic behavior and thus the moment strength of the connection must be tied to a connection rotation value. A connection rotation of 0.02 rad has been used as the requirement in the *Specification*; however, for most composite PR connections, it is more appropriate to use 0.01 rad when considering the positive moment strength (tension at the bottom flange) of the connection. Most PR connections will achieve at least 80% of their ultimate strength

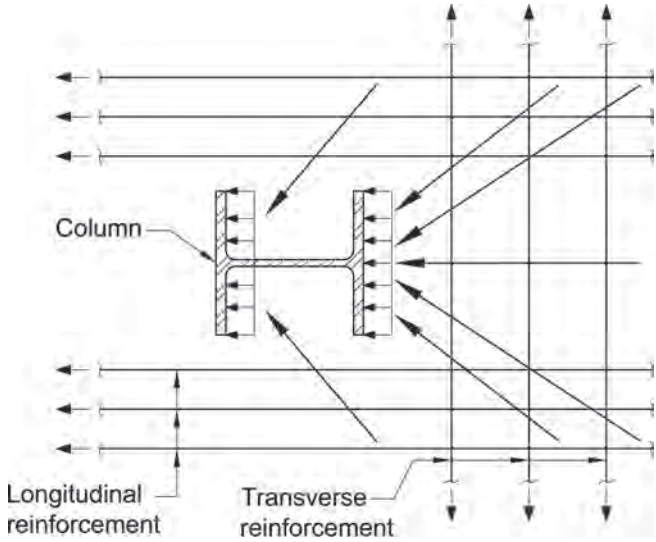


Fig. C-G4.2. Concrete slab bearing force transfer.

at these rotation levels. The 50% M_p requirement is intended to apply to both positive and negative connection strength. This requirement is intended to prevent a potential incremental collapse mechanism from developing.

6d. Conformance Demonstration

Tests results that show general conformance with Section K2 have been reported in the literature (Leon et al., 1987; Leon, 1994). Section K2 is written in terms of story drift rather than in terms of connection rotation; however, the intent of Section K2 for this seismic frame system is to show that the connection is capable of sustaining cyclic strength through a connection rotation of 0.02 rad. Therefore, the loading sequence of Section K2.4b should be considered in the context of connection rotation rather than story drift and need only be taken through step (f) of the loading sequence.

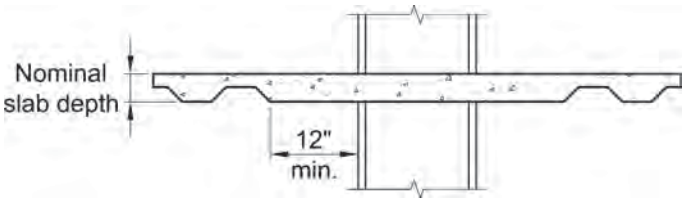


Fig. C-G4.3. Solid slab to be provided around column.

CHAPTER H

COMPOSITE BRACED-FRAME AND SHEAR-WALL SYSTEMS

H1. 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 composite ordinary braced frame (C-OBF) category is provided for systems without special seismic detailing that are used in seismic design categories A, B and C. Thus, the C-OBF systems are considered comparable to structural steel systems that are designed according to the *Specification* using a seismic response modification coefficient, *R,* of 3. Because significant inelastic load 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.

1. Scope

The combination of steel, concrete and/or composite member types that is permitted for C-OBF is intended to accommodate any reasonable combination of member types as permitted by the *Specification* and ACI 318 (ACI, 2014).

6. Connections

Examples of connections used in C-OBF are shown in Figures C-H1.1 through C-H1.3. As with other systems designed in accordance with the *Specification* for a

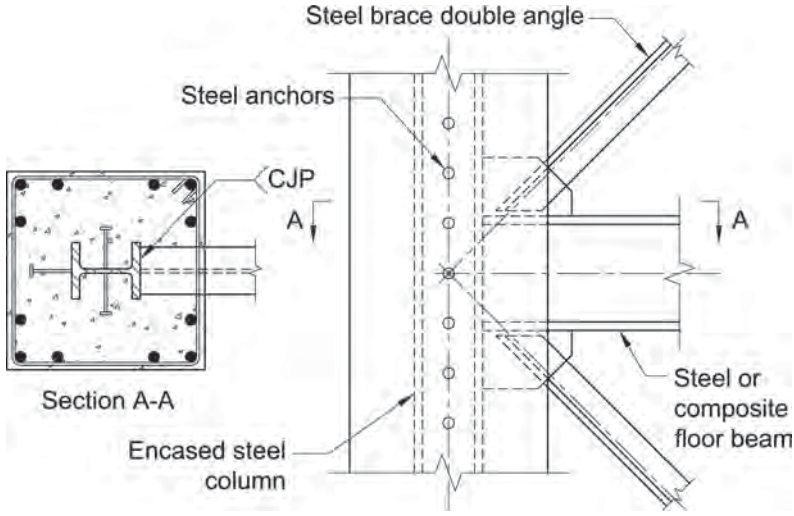


Fig. C-H1.1. Reinforced concrete (or composite) column-to-steel concentric brace.

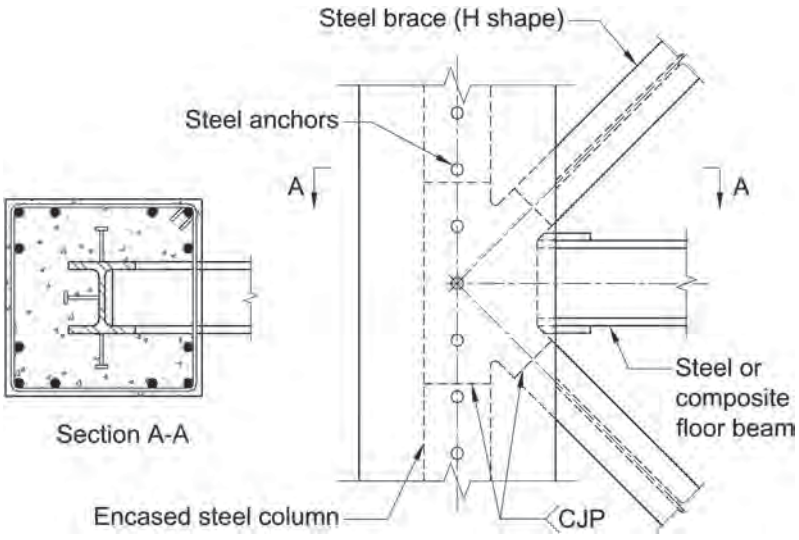


Fig. C-H1.2. Reinforced concrete (or composite) column-to-steel concentric brace.

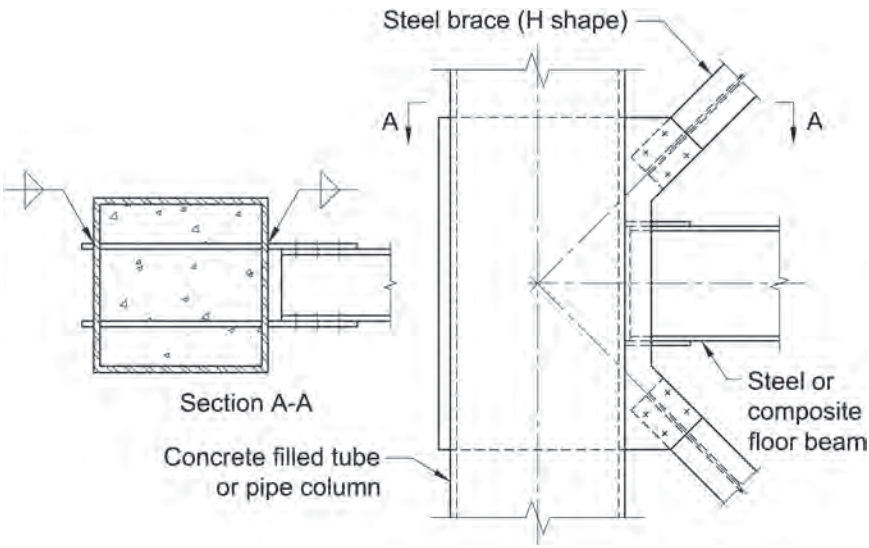


Fig. C-H1.3. Filled HSS or pipe column-to-steel concentric base.

seismic response modification coefficient, *R,* of 3, the connections in C-OBF should have design strengths that exceed the required strengths for the earthquake loads in combination with gravity and other significant loads. The provisions of Section D2.7 should be followed insofar as they outline basic assumptions for calculating the strength of force transfer mechanisms between structural steel and concrete members and components.

H2. COMPOSITE SPECIAL CONCENTRICALLY BRACED FRAMES (C-SCBF)

The composite special concentrically braced frame (C-SCBF) is one of two types of composite braced frames that are specially detailed for seismic design categories D, E and F; the other is the composite eccentrically braced frame (C-EBF). While experience using C-SCBF is limited in high seismic regions, the design provisions for C-SCBF are intended to provide behavior that is comparable to steel SCBF, wherein the braces often are the elements most susceptible to inelastic deformations (see Commentary Section F2). Values and usage limitations for the response modification coefficient, R, and deflection amplification factor, C_d, for C-SCBF are similar to those for steel SCBF.

1. Scope

Unlike C-OBF, which permit the use of concrete columns, the scope for C-SCBF is limited to systems with composite columns to help ensure reliable force transfer from the steel or composite braces and beams into the columns.

2. Basis of Design

The basis of design is comparable to steel SCBF. Thus, the provisions for analysis, system requirements, members and connections make reference to the provisions of Section F2. Refer to the associated commentary for Section F2 where reference is made to that section in the Provisions.

3. Analysis

Just as the SCBF requires the system to be designed for the effects of the brace member tensile capacity and the cyclic post-buckling behavior, so does the composite system. Composite braces can develop higher forces than the steel brace member itself, due to compressive capacity of the concrete area as well as tension capacity of developed longitudinal reinforcing in the concrete. The maximum loads the connection may be required to resist will need to consider the concrete and reinforcing steel overstrength.

4. System Requirements

Multi-tiered braced frames (MTBF) are permitted for C-SCBF consistent with the scope of Section H2, with the exception that composite braces are not permitted for MTBF, as there is insufficient basis for developing appropriate strength and stiffness requirements for composite braces in MTBF.

5. Members

Composite columns in C-SCBF are detailed with similar requirements to highly ductile composite columns in C-SMF. Special attention should be paid to the detailing of the connection elements (MacRae et al., 2004).

5b. Diagonal Braces

Braces that are all steel should be designed to meet all requirements for steel braces in Section F2.

In cases where composite braces are used (either filled or 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 hollow structural sections (HSS) with concrete has been shown to effectively stiffen the HSS walls and inhibit local buckling (Goel and Lee, 1992). For encased steel braces, the concrete should be sufficiently reinforced and confined to prevent the steel shape from buckling. To provide high ductility, the composite braces are required to be designed to meet all requirements for encased composite columns as specified in Section D1.4b. Composite braces in tension should be designed based on the steel section alone unless test data justify higher strengths.

6. Connections

Careful design and detailing of the connections in a C-SCBF is required to prevent connection failure before developing the full strength of the braces in either tension or compression. Where the brace is composite, the added brace strength afforded by the concrete should be considered in the connection design. 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. Therefore, as in steel SCBF, the brace connection should either be designed to accommodate the inelastic rotations associated with brace buckling or to have sufficient strength and stiffness to accommodate plastic hinging of the brace adjacent to the connection.

6a. Demand Critical Welds

For general commentary on demand critical welds see Commentary Section A3.4.

6b. Beam-to-Column Connections

Ductile connections between the beam and column are required for C-SCBF. Rotation requirements for both simple and moment-resisting connections are provided. See Commentary Section F2.6b for further discussion.

6d. Column Splices

The requirements for column splices are comparable to those of C-IMF. Refer to Commentary Section G2.6f.

H3. COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF)

1. Scope

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 Commentary Section F3). While there has been little use of composite eccentrically braced frames (C-EBF), the inelastic behavior of the critical steel link region should be comparable to that of steel EBF and inelastic deformations in the encased composite or filled composite columns should be minimal as well as in the structural steel or filled composite braces. 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.

2. Basis of Design

The basic design requirements for C-EBF are the same as those for steel EBF, with the primary energy absorption being provided by the structural steel link.

A small eccentricity of less than the beam depth is allowed for brace-to-beam or brace-to-column connections away from the link. Small eccentricities are sometimes required for constructability reasons and will not result in changing the location of predominate inelastic deformation capacity away from the link as long as the resulting secondary forces are properly accounted for.

3. Analysis

As with EBF, satisfactory behavior of C-EBF is dependent on making the braces and columns strong enough to remain essentially elastic under loads 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 region of the link not be encased in concrete.

6. Connections

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-OBF (Figures C-H1.1 through C-H1.3). An example where the link is adjacent to the column is shown in Figure C-H3.1. In this case, the link-to-column connection is similar to composite beam-to-column moment connections in C-SMF (Section G3) and to steel coupling beam-to-wall connections (Section H5).

6a. Beam-to-Column Connections

While the majority of the energy dissipation is anticipated to occur at the link, beam-to-column connections in C-EBF are anticipated to go through large rotations as the system undergoes large inelastic deformations. The maximum inelastic deformations are anticipated to be on the order of 0.025 rad, resulting in the requirement that when simple beam-to-column connections are used that they be capable of undergoing this

rotation demand. Alternatively, fully restrained, ordinary moment connections can also be used since they have been shown to accommodate this rotation demand. See Commentary Section F2.6b for further discussion.

H4. COMPOSITE ORDINARY SHEAR WALLS (C-OSW)

1. Scope

This section applies to uncoupled reinforced concrete shear walls with composite boundary elements (see Figure C-H4.1), and coupled reinforced concrete walls, with or without composite boundary elements, in which structural steel or composite coupling beams connect two or more adjacent walls (see Figure C-H4.2).

Structural steel or composite boundary elements may be used as wall boundary elements or for erection purposes only. In the latter case, the structural steel members may be relatively small. The detailing of coupling beam-to-wall connections depends on whether structural shapes are embedded in the wall boundaries or the wall has conventional reinforced concrete boundary elements. If steel or composite column boundary elements are used, the coupling beams can frame into the columns and transmit the coupling forces through a moment connection with the steel column [see Figure C-H4.3(a)]. The use of a moment connection is, however, not preferred given the cost and difficulty of constructing ductile connections. Alternatively, the coupling beam may be connected to the embedded boundary column with a shear connection while the moment resistance is achieved by a combination of bearing along the embedment length and shear transfer provided by steel anchors along the coupling beam flanges [see Figure C-H4.3(b)].

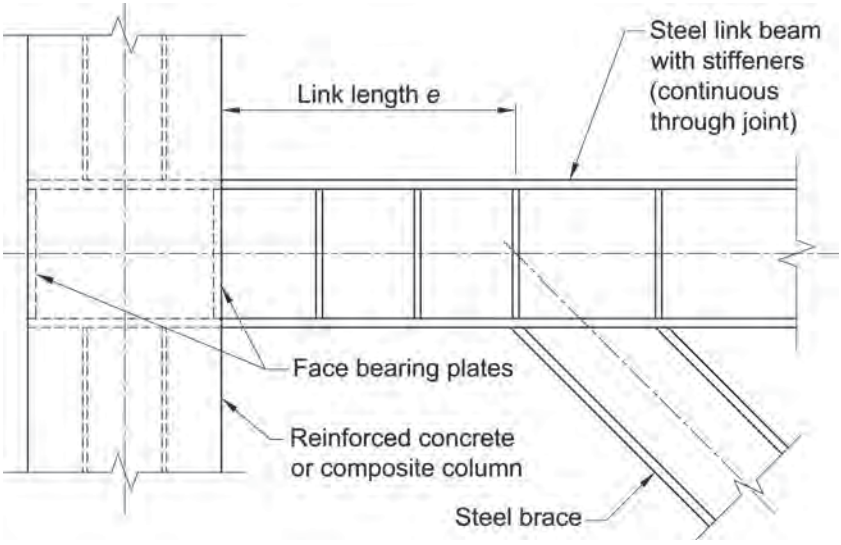


Fig. C-H3.1. Reinforced concrete (or composite) column-to-steel eccentric brace. (Note: Stiffeners are designed according to Section F3.5a.)

If structural steel or composite boundary elements are not present, the coupling beam should be embedded a sufficient distance into the wall so that the coupling forces are transmitted entirely through the interaction that occurs between the embedded coupling beam and the surrounding concrete.

2. Basis of Design

The level of inelastic deformation in C-OSW is limited. Equations H4-1 and H4-1M predict the shear strength of the beam-to-wall connection and inherently provide the required flexural strength through interaction of the embedded portion of the beam

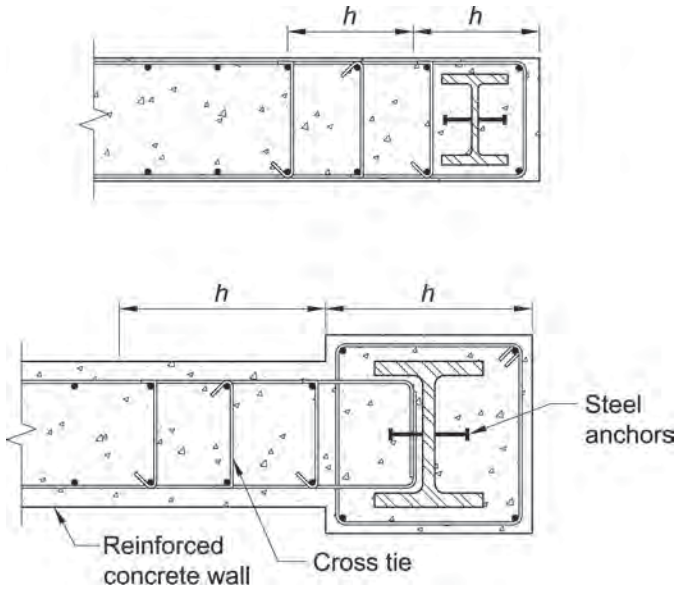


Fig. C-H4.1. Reinforced concrete walls with composite boundary element.

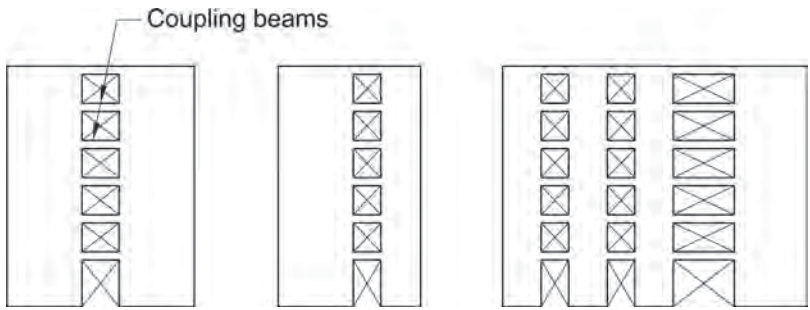
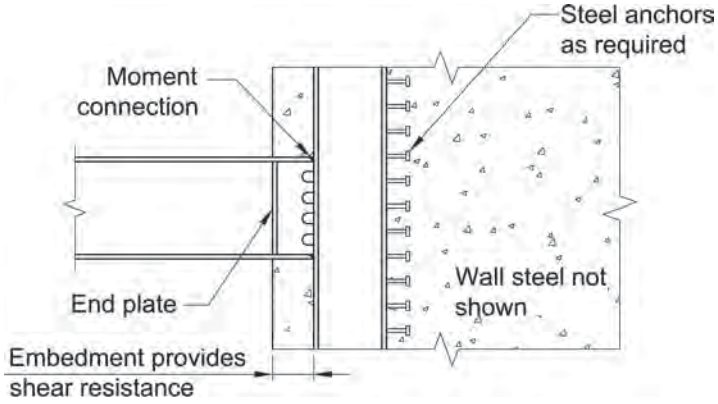
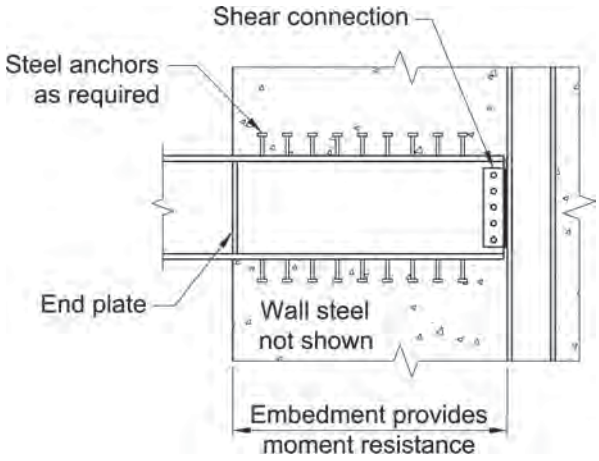


Fig. C-H4.2. Examples of coupled wall geometry.

with the surrounding concrete. Equations H4-2 and H4-2M allow for yielding and implicit ductility in shear. It is, thus, expected that the h/t_w requirements of *Specification* Section G2 will be satisfied such that $C_v = 1.0$ in the calculation of the nominal shear strength of a steel beam within a composite coupling beam. For a composite coupling beam, the minimum shear reinforcement requirements from ACI 318 (ACI, 2014) are satisfied. The wall piers are to be designed based on nonseismic provisions of ACI 318, i.e., the requirements of Chapter 18 do not have to be satisfied for these ordinary systems.



(a) Steel coupling beam attached to steel wall boundary element column



(b) Steel coupling beam attached to steel erection column

Fig. C-H4.3. Steel coupling beam details.

3. Analysis

In order to compute the design forces and deformations, the wall piers, coupling beam elements, and the coupling beam-to-wall connections need to be modeled considering cracked section properties for concrete. Guidance from ACI 318 Chapter 6 (Sections 6.6.3.1.1, 6.6.4.2, 6.7.1.3) and ASCE 41 (ASCE, 2013) is available.

Modeling of the wall piers falls into three main classes (in increasing degree of complexity): 1) equivalent frame models, 2) multi-spring models, and 3) continuum finite element model (ASCE, 2009). Previous studies (Shahrooz et al., 1993; Gong and Shahrooz, 2001b; Harries et al., 1997) have demonstrated that steel or steel-concrete composite coupling beams do not behave as having a fixed boundary condition at the face of the wall. The additional flexibility needs to be taken into account in equivalent frame or multi-spring models to ensure that wall forces and lateral deflections are computed with reasonable accuracy. If the embedment length of the beam is known, the effective fixed point of steel or steel-concrete composite coupling beams may be taken at approximately one-third of the embedment length from the face of the wall (Shahrooz et al., 1993; Gong and Shahrooz, 2001b). Thus, the effective span of the equivalent fixed-end beam used for analysis, $g_{effective}$, is $g + 0.6L_e$ where g is the clear span and L_e is the embedment length. If the value of L_e is not available, the procedure proposed by Harries et al. (1997) may be used. In this procedure, the effective flexural stiffness (reduced to account for the presence of shear) of a steel coupling beam is reduced to 60% of its gross section value:

$$I_{eff} = 0.60I\left(1+\frac{\lambda 12EI}{g^2GA_w}\right) \qquad \text{(C-H4-1)}$$

where

A_w = area of steel section assumed to resist shear, which is typically the area of the steel web, in.^{2} (mm^2)
E = modulus of elasticity of steel, ksi (MPa)
G = shear modulus of steel, ksi (MPa)
I = moment of inertia of steel coupling beam, in.^{4} (mm^4)
λ = cross-section shape factor for shear (1.5 for W-shapes)

4. System Requirements

The coupling beam forces can be redistributed vertically, both up and down the structure, in order to optimize the design (Harries and McNeice, 2006). Redistribution can also help to lower the required wall overstrength and improve constructability by permitting engineers to use one beam section over larger vertical portions of the wall. Given the benefits of redistribution and the inherent ductility of steel coupling beams, a 20% redistribution of coupling beam design forces is recommended provided the sum of the resulting shear strength (e.g., the design strength, ϕV_n) exceeds the sum of the coupling beam design force determined from the lateral loading (e.g., the required strength, V_u) (CSA, 2004); for example, $\sum\phi V_n/\sum V_u \geq 1$. This concept is schematically illustrated in Figure C-H4.4.

5. Members

5b. Coupling Beams

Coupling beam response is intended to be similar to shear link response in eccentrically braced frames (EBF). The expected coupling beam chord rotation plays an important role in how the coupling beam is detailed. This angle may be computed from

$$\theta_b = \frac{L - g_{effective}}{g_{effective}} \theta_d \qquad \text{(C-H4-2)}$$

where

L = distance between the centroids of the wall piers, in. (mm)
$g_{effective}$ = effective clear span as discussed in Commentary Section H4.3, in. (mm)
θ_d = story drift angle, computed as the story drift divided by the story height, rad (Harries et al., 2000)

For cases in which the coupling beam embedment into the wall piers is the only mechanism of moment resistance, the embedment length has to be long enough to develop the required shear demand determined from structural analysis that considers all the applicable load combinations. Models have been developed for connections between steel brackets and reinforced concrete columns (e.g., Mattock and Gaafar, 1982). These models are used to compute an embedment length required to prevent bearing failure of concrete surrounding the flanges of the embedded steel members. A number of studies (Shahrooz et al., 1993; Gong and Shahrooz, 2001a, 2001b; Fortney, 2005) have demonstrated the adequacy of Mattock and Gaafar's model for coupling beams subjected to reversed cyclic loading. Other models (Harries et al., 1997) may

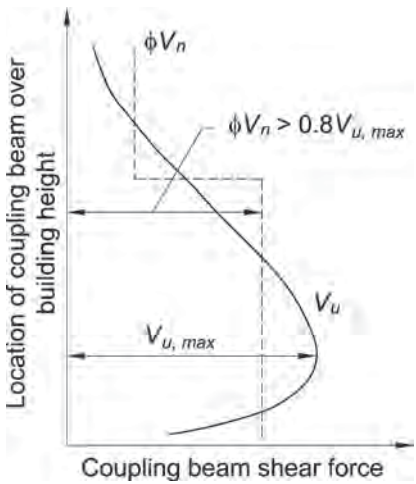


Fig. C-H4.4. Vertical distribution of coupling beam shear.

also be used. Equations H4-1 and H4-2 are based on the model developed by Mattock and Gaafar (1982) and recommended by ASCE (2009). The strength model in this equation is intended to mobilize the moment arm, Z, between bearing forces C_f and C_b shown in Figure C-H4.5.

A parabolic distribution of bearing stresses is assumed for C_b, and C_f is estimated by a uniform stress equal to

$$f_b = 1.54\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66} \quad \text{(C-H4-3)}$$

where

b_f = width of flange, in. (mm)
b_w = width of wall, in. (mm)
f_c' = specified compressive strength of concrete, ksi (MPa)

The b_f/b_w term accounts for spreading of the compressive stress beneath the beam flange as shown in Section A-A of Figure C-H4.5 and was calibrated based on experimental data. In Equation H4-1, the ratios of c/L_e and k_2 as shown in Figure C-H4.5 are assumed to be 0.66 and 0.36 respectively, as recommended by Mattock and Gaafar

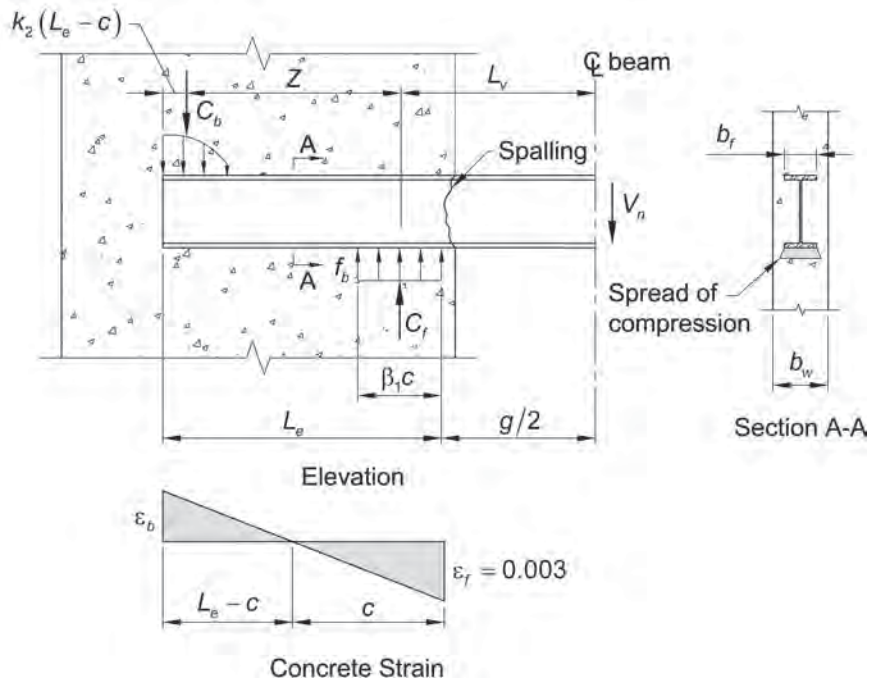


Fig. C-H4.5. Method for computing the embedment capacity.

(1982). The $g/2$ parameter, shown in Figure C-H4.5, is the parameter, a, used by Mattack and Gaafar to define one-half the effective span of the coupling beam.

Vertical wall reinforcement sufficient to develop the required shear strength of the coupling beam will provide adequate control of the gaps that open at the beam flanges under reversed cyclic loading (Harries et al., 1997). Harries et al. (1997) recommends that two-thirds of the required vertical wall reinforcement be located within a distance of one-half the embedment length from the face of the wall. The vertical bars must have adequate tension development length above and below the flanges of the coupling beam. The vertical reinforcement in wall boundary elements, if present, is typically sufficient to meet these requirements.

Steel coupling beams may be encased in reinforced concrete. Previous research (Gong and Shahrooz, 2001a, 2001b) indicates that nominal encasement significantly improves resistance to flange and web buckling, and enhances the strength of the coupling beam. The required embedment length must be computed recognizing the beneficial effects of encasement. Equations H4-2 and H4-2M for computing the shear strength of encased coupling beams are based on meeting the ACI 318 minimum shear reinforcement requirements. Hence, minimum shear reinforcement needs to be provided regardless of the calculated value of shear force in the coupling beam.

H5. COMPOSITE SPECIAL SHEAR WALLS (C-SSW)

1. Scope

The provisions in this section apply to coupled wall systems with steel or composite coupling beams. The reinforced concrete walls may or may not have structural steel or composite sections serving as boundary elements. Examples of systems with such boundary element conditions are discussed in Commentary Section H4.1. The focus of this section is on composite special shear walls.

For cases in which special reinforcement detailing in the wall boundary region is required, it is not necessary, nor is it typically practical, to pass wall boundary transverse reinforcing bars through the web of the embedded coupling beam. A practical alternative is to place hooked ties on either side of the web, and to provide short vertical bars between the flanges to anchor these ties, as shown in Figure C-H5.1.

2. Basis of Design

The preferred sequence of yielding for coupled walls is for the coupling beams to yield over the entire height of the structure prior to yielding of the walls at their bases (Santhakumar, 1974). This behavior relies on coupling beam-to-wall connections that can develop the expected flexural and shear strengths of the coupling beams. For steel coupling beams, or steel beams embedded within composite coupling beams, satisfying the requirements of Section F3.5b ensures adequate ductility for shear yielding. For a composite coupling beam, the shear strengths in Equations H5-5 and H5-5M are assessed assuming the minimum shear reinforcement requirements are satisfied from ACI 318, thus enabling the coupling beam to yield in shear.

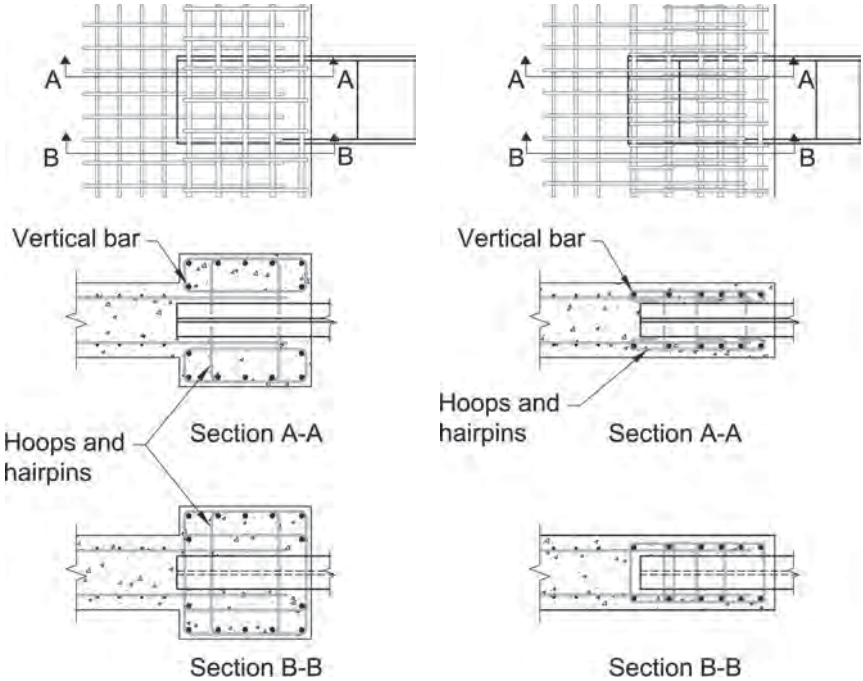
3. Analysis

Wall piers in special shear walls will experience significant plastic deformations. Appropriate stiffness values need to be selected to account for the differences between the cracked section properties of the walls in the plastic hinge region and regions that are expected to remain elastic. Guidance from ACI 318 Chapter 6 (Sections 6.6.3.1.1, 6.6.4.2, 6.7.1.3) and ASCE 41 is available (see also Commentary Chapter C).

To account for spalling at the coupling beam-to-wall connection, the value of $g_{effective}$ (discussed in Commentary Section H4.3) needs to be computed based on g = clear span + 2(clean cover) to the first layer of confining reinforcement in the wall boundary member.

4. System Requirements

In order to ensure the preferred plastic mechanism in coupled walls, for example, that the coupling beams yield prior to the wall piers, a wall overstrength factor, ω_o, is applied to the wall design forces. The required wall overstrength is taken as the ratio of the sum of the nominal shear strengths of the coupling beams, V_n, magnified by $1.1R_y$, to the sum of the coupling beam required shear strengths determined for the



(a) Steel coupling framing into "barbell" wall boundary region

(b) Steel coupling beam framing into rectangular wall boundary region

Fig. C-H5.1. Example details of a steel coupling beam embedded in reinforced concrete wall.

case of factored lateral loading, V_u, (excluding the effects of torsion) (CSA, 2004) where

$$\omega_o = \sum 1.1 R_y V_n / \sum V_u \qquad \text{(C-H5-1)}$$

This factor, therefore, includes the natural overstrength resulting from the design procedure and strength reduction factors and the overstrength resulting from designing for critical beams and using this design over a vertical cluster of beams (or all the beams) in the structure. The 20% vertical redistribution of beam forces described in Section H4.4 is permitted for special wall systems and will help to mitigate large wall overstrength factors.

The required wall overstrength can have a significant effect on wall pier design forces (Fortney, 2005; Harries and McNeice, 2006) and can adversely affect the economy of the system. Required wall overstrength will typically be greater in structures having a higher coupling ratio due to the relatively steep gradient of beam shear demand over the height of the structure (Figure C-H4.4). An advantage of a greater coupling ratio is that wall pier forces are reduced, but the larger wall overstrength factor may negate this advantage. Permitting the redistribution of beam forces as described in Section H4.4 may minimize this effect.

5. Members

5a. Ductile Elements

Coupling beams must be able to undergo substantial inelastic deformation reversals; therefore, coupling beams are designated as protected zones. Well-established guidelines for shear links in eccentrically braced frames need to be followed.

5b. Boundary Members

Concerns have been raised that walls with encased steel boundary members may have a tendency to split along planes 1 and 2 shown in Figure C-H5.2. Transverse reinforcement within a distance 2*h* (*h* = width of the wall*)* will resist splitting along plane 1 while the wall horizontal reinforcement will be adequate to prevent failure along plane 2.

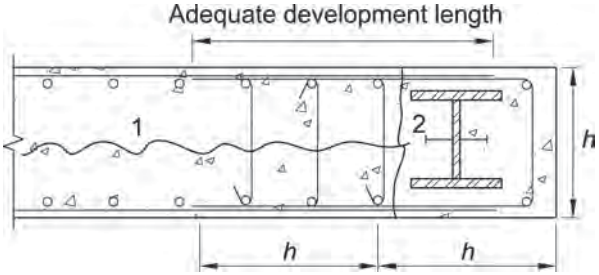


Fig. C-H5.2. Reinforcement to prevent splitting failures.

5c. Steel Coupling Beams

A coupling beam rotation equal to 0.08 rad reflects the upper limit of link rotation angle in eccentrically braced frames (EBF). It should be noted that 0.08 rad may be conservative for coupled walls, in which case using this rotation will result in extra stiffeners in the coupling beam. A smaller value of link rotation may be used if established by rational analysis to determine the inelastic deformational demands expected at the design story drift.

In addition to the potential use of stiffeners along the span between the reinforced concrete walls, face-bearing plates must be provided at the face of the wall. Face bearing plates are full-width stiffeners located on both sides of the web, in effect, that close the opening in the concrete form required to install the beam. Face bearing plates provide confinement and assist in transfer of loads to the concrete through direct bearing. If it is convenient for formwork, face-bearing plates may extend beyond the flanges of the coupling beam although the plate must be installed on the inside of the form and is thereby flush with the face of the wall. The face bearing plates are detailed as a stiffener at the end of a link beam as in Section F3.5b.4. Near the end of the embedded region, additional stiffeners similar to the face bearing plates need to be provided. These stiffeners are to be aligned with the vertical transfer bars near the end of the embedded region.

In addition to boundary element reinforcing, two regions of vertical "transfer bars" are to be provided to assist in the transfer of vertical forces and thus improve the embedment capacity (Shahrooz et al., 1993; Gong and Shahrooz, 2001a, 2001b; Fortney, 2005). Evaluation of experimental data in which transfer bars had been used (Gong and Shahrooz, 2001a, 2001b; Fortney, 2005) indicates that the minimum required area of vertical transfer reinforcement is (see Figure C-H5.3):

$$A_{tb} \geq 0.03 f_c' L_e b_f / F_{ysr} \qquad \text{(C-H5-2)}$$

where

F_{ysr} = specified minimum yield stress of transfer reinforcement, ksi (MPa)
L_e = embedment length of coupling beam, in. (mm)
b_f = width of flange, in. (mm)
f_c' = specified compressive strength of concrete, ksi (MPa)

The transfer bars need to be placed close to the face of the wall and near the end of embedment length in order to develop an internal force couple that can alleviate the bearing stresses around the flanges and improve the energy dissipation characteristics of coupling beam-to-wall connections (Gong and Shahrooz, 2001a, 2001b). Although the required embedment length of the coupling beam may be reduced if the contribution of these bars is taken into account (Qin, 1993), to avoid excessive inelastic damage in the connection region, it is recommended by Harries et al. (1997) and Shahrooz et al. (1993) that the contribution of the transfer bars be neglected in the determination of the required embedment length. The vertical transfer bars may be attached directly to the top and bottom flanges or be passed through holes in the flanges and mechanically anchored by bolting or welding. The use of mechanical

half couplers that are welded to the flanges has been successfully tested (Gong and Shahrooz, 2001a, 2001b; Fortney, 2005). U-bar hairpin reinforcement anchored by the embedded coupling beam may also be used (Figure C-H5.4). These hairpins will be alternated to engage the top and bottom flanges. The transfer bars have to be fully developed in tension either by providing an adequate tension development length or through the use of headed bars. In order to prevent congestion, the sum of the areas of transfer bars and wall longitudinal bars over the embedment length (A_s shown in Figure C-H5.3 or the area of U-bar hairpins in Figure C-H5.4) is limited to 8% of the wall cross section taken as the wall width times the embedment length.

The vertical transfer bars shown in Figure C-H5.3 is a suggested detail for beams located at a floor level where the wall piers extend far enough above the floor/roof level to accommodate the vertical transfer bars. For coupling beams located at the roof level where the wall piers do not extend far enough above the floor/roof level, alternate details will need to be considered. Such alternate details are presented and discussed in El-Tawil et al. (2009).

Equation H5-1 is derived using the same method as described for Equation H4-1 (see Commentary Section H4.5b).

5d. Composite Coupling Beams

The required embedment length needs to be calculated to ensure that the capacity of the composite coupling beam is developed. Based on analytical studies, which were verified against experimental data, Gong and Shahrooz (2001a) proposed an equation in which a single material overstrength factor had been specified for computing

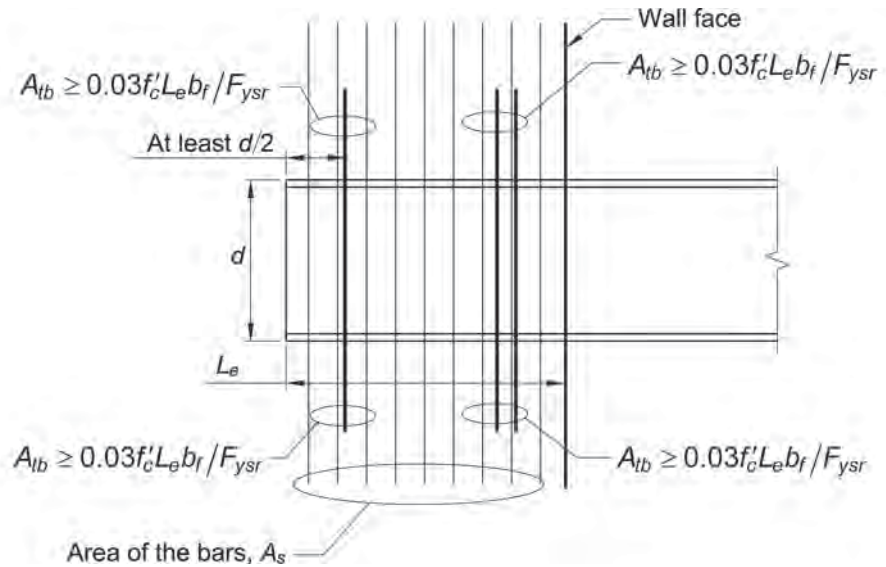


Fig. C-H5.3. Transfer bars.

the contribution of concrete and transverse reinforcement towards the shear strength of composite coupling beams. In that equation, the specified concrete compressive strength, f'_c, and nominal yield strength of transverse reinforcement, F_{ysr}, were to be used. Equation H5-5 and H5-5M are revised versions of the original equation in order to more transparently differentiate between the material overstrength factors for concrete and reinforcing steel. The coefficients in this equation were calibrated in order to obtain the same values as those from the original form of the equation published by Gong and Shahrooz (2001a, 2001b).

5e. Protected Zones

Coupling beams are expected to undergo significant inelastic deformations. With the exception of transfer bars, face bearing plates, and web stiffeners, the entire clear span is designated as a protected zone.

6. Connections

Structural steel sections as boundary elements in C-SSW are anticipated to undergo significant inelastic deformations, particularly in the plastic hinge region. The boundary columns have to be adequately anchored to the foundation system. Equally important are the splices along the boundary columns. These connections are designated as demand critical welds.

H6. COMPOSITE PLATE SHEAR WALLS— CONCRETE ENCASED (C-PSW/CE)

In previous edition of these provisions, composite plate shear walls were included in a single section. In the 2016 Provisions, composite plate shear walls have been

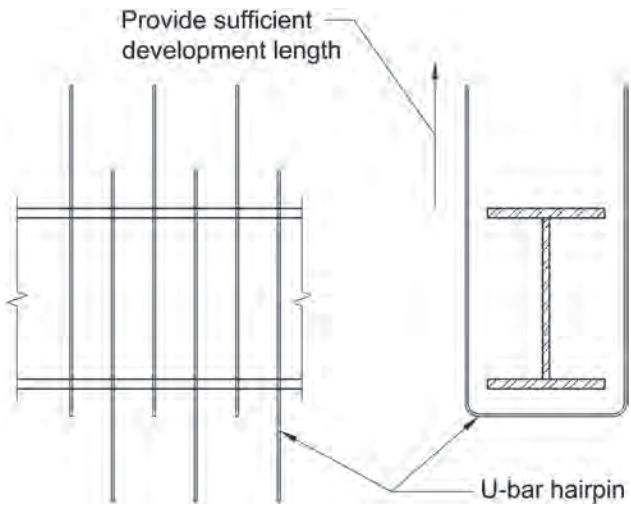


Fig. C-H5.4. Alternating U-shaped hairpins.

distinguished as concrete encased (C-PSW/CE) in Section H6 and concrete filled (C-PSW/CF) in Section H7. Both of these systems are designated as a single system, composite plate shear walls (C-PSW), in ASCE/SEI 7 Table 12.2-1 (ASCE, 2016).

1. Scope

Composite plate shear walls—concrete encased (C-PSW/CE) can be used most effectively where story shear loads are large and the required thickness of conventionally reinforced shear walls is excessive. Limited research on these types of systems has included configurations in which reinforced concrete is used on one side of the steel plate to mitigate the effects of local buckling (Zhao and Astaneh-Asl, 2004), and cases where two steel plates are used with reinforced concrete between them (e.g., Ozaki et al., 2004), as covered in Section H7.

3. Analysis

3a. Webs

In keeping with the intended system response, the provisions of this section target having the steel webs of the C-PSW/CE system be the primary structural elements that first attain inelastic response.

3b. Other Members and Connections

The provisions of this section target having the boundary elements of the C-PSW/CE system remain essentially elastic under the maximum forces that can be generated by the fully yielded steel webs, along with the engaged portions of the reinforced concrete webs after the steel webs have fully yielded, except that plastic hinging at the ends of horizontal boundary elements (HBE) and the column base are permitted.

4. System Requirements

The provisions of Section F5 are invoked for Sections H6.4b and H6.4c to ensure the boundary elements have adequate stiffness and strength.

4e. Openings in Webs

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 steel plates around the openings.

5. Members

5b. Webs

The Provisions limit the shear strength of the wall to the yield stress of the plate because there is insufficient basis from which to develop design rules for combining the yield stress 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. ASCE/SEI 7 assigns 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.

5c. Concrete Stiffening Elements

Minimum reinforcement in the concrete cover is required to maintain the integrity of the wall under reversed cyclic in-plane loading and out-of-plane loads. Consideration should be given to splitting of the concrete element on a plane parallel to the steel plate. 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. Examples of such reinforcement are shown in Figures C-H6.1 through C-H6.4.

5d. Boundary Members

C-PSW/CE systems can develop significant diagonal compressions struts, particularly if the concrete is activated directly at the design story drift. These provisions ensure that the boundary elements have adequate strength to resist this force.

6. Connections

Two examples of connections between composite walls to either steel or composite boundary elements are shown in Figures C-H6.1 and C-H6.2.

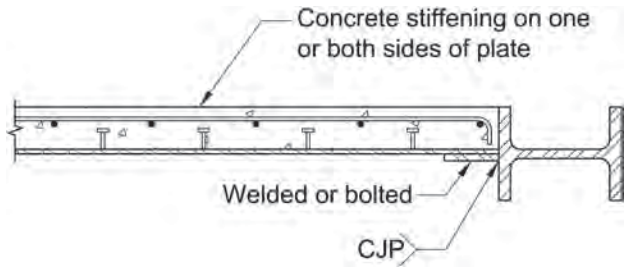


Fig. C-H6.1. Concrete stiffened steel shear wall with steel boundary member.

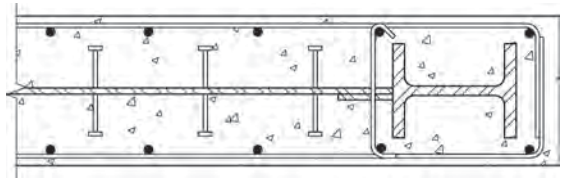


Fig. C-H6.2. Concrete stiffened steel shear wall with composite (encased) boundary member.

6a. Demand Critical Welds

In addition to the welds at the column splices and base plates, the welds at the connections between the boundary elements are potentially subjected to large inelastic excursions and so are designated as demand critical.

6b. HBE-to-VBE Connections

The provisions of Section F5 are invoked to provide adequate strength in the boundary element connections.

6c. Connections of Steel Plate to Boundary Elements

The Provisions require that the connections between the plate and the boundary members be designed to develop the nominal shear strength of the plate.

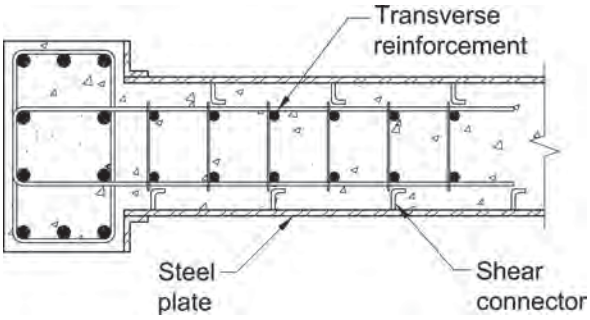


Fig. C-H6.3. Concrete filled C-PSW with a boundary element and transverse reinforcement.

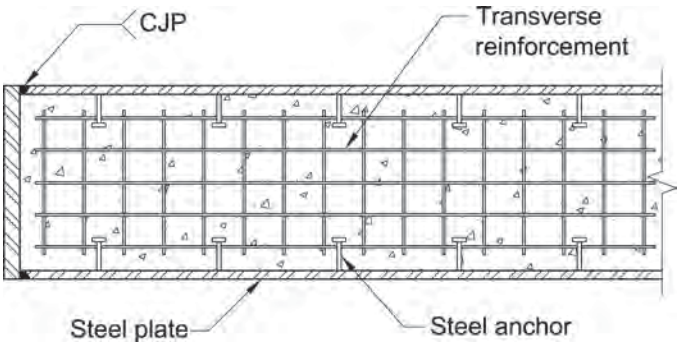


Fig. C-H6.4. Concrete filled C-PSW with transverse reinforcement to provide integrity of the concrete infill.

6d. Connections of Steel Plate to Reinforced Concrete Panel

The thickness of the concrete encasement and the spacing of shear stud connectors should be calculated to allow the steel plate to reach yield prior to overall or local buckling. It is recommended that overall buckling of the composite panel be checked using elastic buckling theory with a transformed section stiffness for the wall. It is recommended that local steel plate buckling be checked using elastic buckling theory considering steel connectors as fixed plate support points (Choi et al., 2009).

H7. COMPOSITE PLATE SHEAR WALLS— CONCRETE FILLED (C-PSW/CF)

In the previous edition of these provisions, composite plate shear walls were included in a single section. In these Provisions, composite plate shear walls have been distinguished as being concrete encased (C-PSW/CE) in Section H6 and concrete filled (C-PSW/CF) in Section H7. Both of these systems are designated as a single system, composite plate shear walls (C-PSW) in ASCE/SEI 7 Table 12.2-1 (ASCE, 2016).

1. Scope

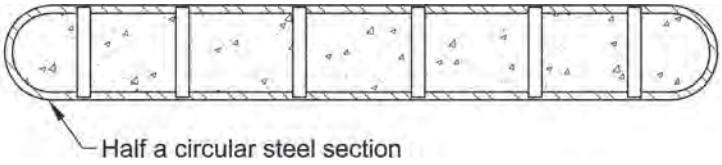
Composite plate shear walls—concrete filled (C-PSW/CF) are an alternative to reinforced concrete walls especially when relatively large seismic demand on the walls leads to dense reinforcement and large thicknesses in conventional concrete shear walls, or to relatively large walls thicknesses of the web infill and boundary elements in SPSW. C-PSW/CF can also be provided with concrete-filled tube (CFT) boundary elements to address high seismic demands.

The use of half-circular steel sections at the end of the C-PSW/CF cross section avoids premature failure of the welds between the steel web plate and the flange in the case of rectangular corners (i.e., when end plates are used at the ends of the wall) due to large strains at that location of welding (e.g., El-Bahey and Bruneau, 2010, 2012). Examples of the types of wall cross-sections addressed by Section H7 are shown in Figure C-H7.1.

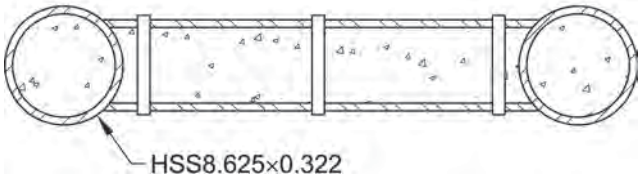
Figures C-H7.1a and C-H7.1b show the C-PSW/CF system with half-circular and full-circular boundary elements, respectively. Figure C-H7.2 shows representative cyclic hysteresis behavior of C-PSW/CF with boundary elements, with interstory drift ratio capacities exceeding 3% (Bruneau et al., 2013; Alzeni and Bruneau, 2014).

Figure C-H7.3 shows C-PSW/CF without boundary elements. The steel plates are connected to each other using tie bars. They can be additionally anchored to the concrete infill using ties or a combination of ties and shear studs to achieve the slenderness ratio (w_1/t) limit in the Provisions.

As discussed in Kurt et al. (2016), C-PSW/CF without boundary elements detailed according to these provisions have cyclic behavior better than equivalent reinforced concrete walls with orthogonal grids of curtain reinforcement. Reinforced concrete shear walls typically have interstory drift ratio capacities of 0.5 to 0.75%. As shown



(a) Composite plate shear wall – concrete filled with half-circular boundary elements (Alzeni and Bruneau, 2014)



(b) Composite plate shear wall – concrete filled with CFT boundary elements (Alzeni and Bruneau, 2014)

Fig. C-H7.1. Two types of C-PSW.

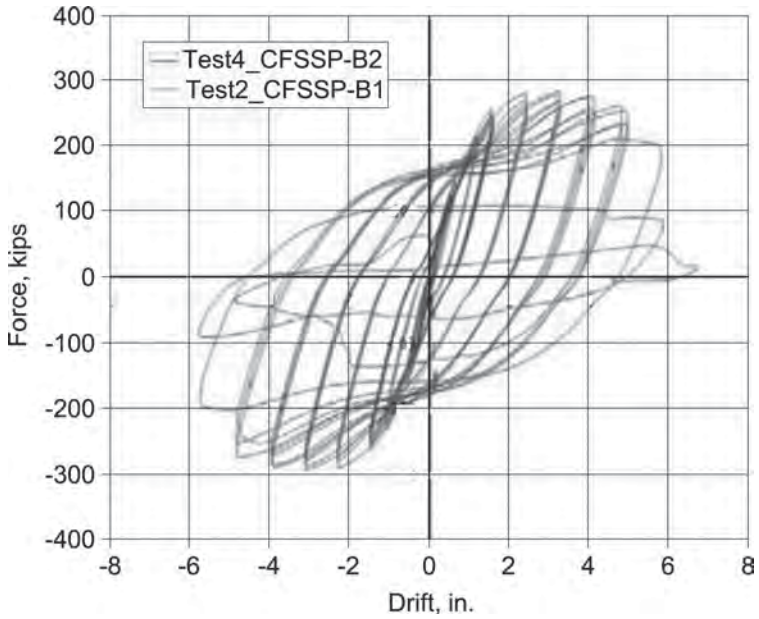


Fig. C-H7.2. Hysteretic behavior of C-PSW/CF with boundary elements (Alzeni and Bruneau, 2014).

by the cyclic hysteretic behavior in Figure C-H7.4, C-PSW/CF walls without boundary elements can reach interstory drift ratio capacities exceeding 1.0 to 1.5%.

The scope covered by Section H7 is limited to plane walls. While walls with large flanges and box walls are desirable, the flanges of such walls would be subjected to axial cyclic behavior during earthquake excitations, and although more rapid cyclic strength degradation is expected in such case, the rate and severity of this degradation as a function of ductility demands is unknown at this time. Specimens subjected to monotonic pure compression loading have exhibited non-ductile behavior (Zhang et al., 2014).

2. Basis of Design

Section H7 focuses on walls developing flexural hinging. C-PSW/CF with boundary elements can develop flexural hinging with a strength equal to the wall cross-section plastic moment strength, M_{pc}. C-PSW/CF without boundary elements can develop flexural hinging with a strength equal to the wall cross-section yield moment strength, M_y.

Past research (e.g., Kurt et al., 2016; Alzeni and Bruneau, 2014) has shown that the design of C-PSW/CF having a height-to-length aspect ratio greater than 1.5 is governed by flexural strength. However, this can vary depending on the relative distribution of material between boundary elements and webs.

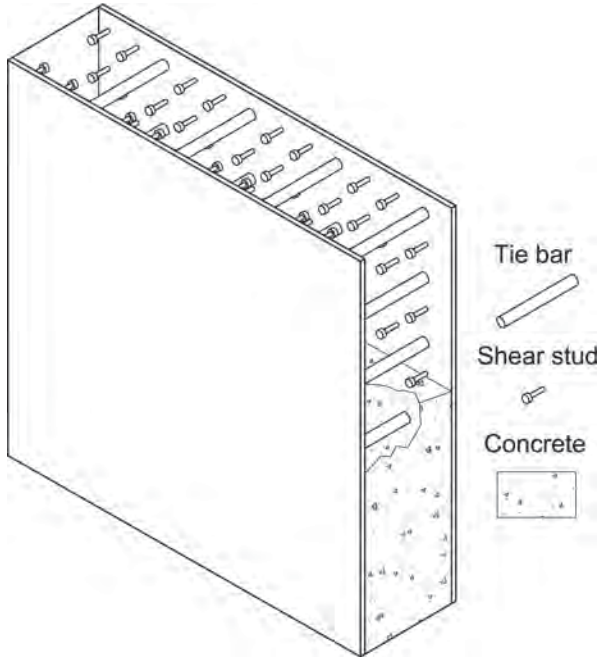


Fig. C-H7.3. C-PSW/CF without boundary elements.

3. Analysis

The value of $C_3 = 0.40$, which defines the contribution of concrete to the elastic stiffness of the wall, is based on calibration with flexural test results. It is to be used in an equation such as:

$$EI_{eff} = E_s I_s + C_3 E_c I_c \qquad \text{(C-H7-1)}$$

where all symbols are defined in *Specification* Section I2. For short walls, a similar factor could be used to calculate effective shear stiffness, if supported by experimental calibration.

4. System Requirements

4a. Steel Web Plate of C-PSW/CF with Boundary Elements

The maximum spacing of the ties is specified such that the steel plate can develop F_y before local buckling. The specified limit been validated experimentally.

4b. Steel Plate of C-PSW/CF without Boundary Elements

The specified limit on the spacing of the ties has been validated experimentally.

4d. Spacing of Tie Bars in C-PSW/CF with or without Boundary Elements

Tie bars serve to develop effective composite action in the sandwich panel. Tie bars provide shear transfer between the steel plate and the concrete core, and are used to control local buckling of the web steel plates as well as to prevent splitting of the concrete.

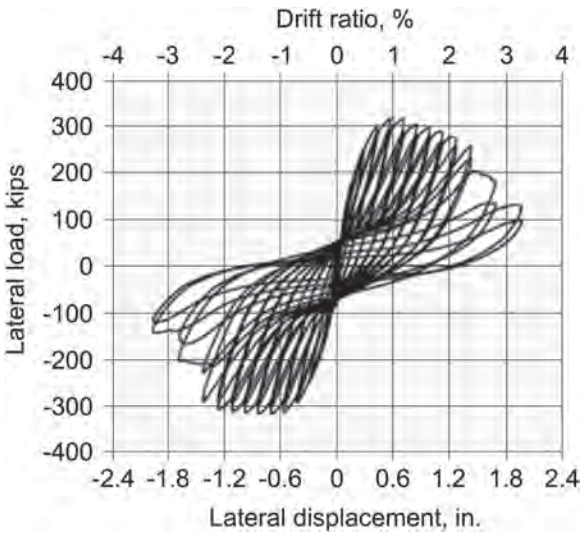


Fig. C-H7.4. Cyclic behavior of C-PSW/CF without boundary elements (Epackachi et al., 2015).

4f. Connection between Tie Bars and Steel Plates

The full yielding force of the tie bar must be transferred to the steel plate, through plug welds over at least half the thickness of the web plate, or by other mechanisms. Examples of possible tie bar connections are shown in Figure C-H7.5.

If plug welds are used to connect tie bars, the practicality of providing plug welds over at least half the steel plate thickness may lead to additional constraints on plate thickness or tie bar diameter.

4h. C-PSW/CF and Foundation Connection

To achieve capacity design principles, the flexural strength of the wall to be transferred to the foundation shall be computed considering expected strengths of the HSS and steel web of the C-PSW/CF, expected strength of the concrete, and strain hardening of the steel. An overstrength factor of 1.1 is applied to the expected flexural strength of the wall to account for strain hardening, but the engineer may consider higher values if appropriate for capacity design of such connections.

5. Members

5a. Flexural Strength

The plastic flexural strength of the C-PSW/CF with boundary elements can be calculated using the following equations:

For C-PSW/CF with half-circular filled boundary elements

$$M_{pc} = M_n = 0.5A_{HSS}F_{y,HSS}\left(\frac{2d_{HSS}}{\pi} + b\right) + \left(b^2 + 2C^2 - 2Cb\right)t_s F_{y,web} + \left(\frac{2d_{in}^3 + 3\pi d_{in}^2 C}{24} + \frac{C^2 t_c}{2}\right)f_c' \quad \text{(C-H7-2)}$$

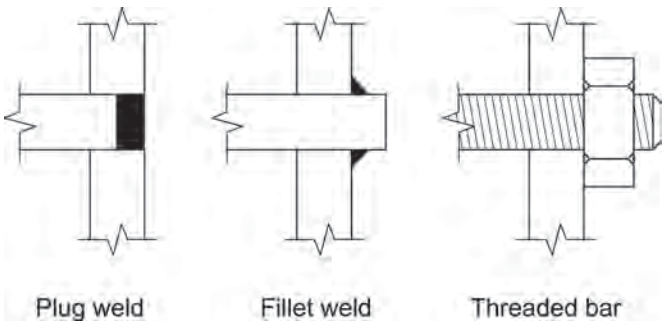


Fig. C-H7.5. Examples of tie-to-plate connection detail.

where C is given by

$$C = \frac{2bt_s F_{y,web} - 0.125(\pi d_{in}^2) f_c'}{4t_s F_{y,web} + t_c f_c'} \quad \text{(C-H7-3)}$$

For C-PSW/CF with filled composite (CFT) boundary elements

$$M_{pc} = M_n = A_{HSS} F_{y,HSS} (b - 2X + d_{HSS}) + \left(b^2 + 2C^2 - 2Cb\right) t_s F_{y,web} + [0.25\pi d_{in}^2 (0.5 d_{HSS} + C - X) + 0.33 X t_c (C - 0.67X) + 0.5 t_c (C - X)^2] f_c' \quad \text{(C-H7-4)}$$

where

$$C = \frac{2bt_s F_{y,web} - (0.25\pi d_{in}^2 - 0.67 X t_c) f_c'}{4t_s F_{y,web} + t_c f_c'} \quad \text{(C-H7-5)}$$

where

$$X = 0.5\left(d_{in} - \sqrt{d_{in}^2 - t_c^2}\right) \quad \text{(C-H7-6)}$$

$\phi = 0.90$

where

A_{HSS} = cross-sectional area of a half-circular or full circular section used at wall end, in.^{2} (mm^2)
C = depth of cross section subjected to yield compressive stress, in. (mm)
$F_{y,HSS}$ = specified minimum yield stress of the half-circular or full-circular end section, ksi (MPa)
$F_{y,web}$ = specified minimum yield stress of the web, ksi (MPa)
b = depth of the steel web, in. (mm)
d_{HSS} = diameter of the HSS section, in. (mm)
d_{in} = inner diameter of the half-circular or full-circular end section, in. (mm)
f_c' = specified compressive strength of concrete, ksi (MPa)
t_c = thickness of concrete, in. (mm)

The plastic flexural strengths are limited to cross sections that have been experimentally demonstrated to have adequate cyclic behavior without significant loss of strength up to expected drifts. Equations for plastic moment have been developed from a fully plastic stress diagram, considering compression and tension stress of F_y in the steel, and concrete compression stress of f_c' for all concrete in compression above the neutral axis (see Figures C-H7.6 and C-H7.7 for assumed stress distribution for walls with and without boundary elements, respectively).

The flexural yield strength, M_y, of C-PSW/CF without boundary elements can be calculated using the following equations:

$$M_y = \frac{0.7}{3} f_c' t_c C^2 + F_y t_s \left(\frac{4}{3} C^2 - 2LC + L^2\right) \quad \text{(C-H7-7)}$$

where C is given by

$$C = \frac{2F_y t_s L}{0.35 f_c' t_c + 4F_y t_s} \quad \text{(C-H7-8)}$$

where

C = depth of cross section under compression, in. (mm)
F_y = specified minimum yield stress of the steel plates, ksi (MPa)
L = length of the wall, in. (mm)
f_c' = specified compressive strength of concrete, ksi (MPa)
t_c = concrete infill thickness, in. (mm)
t_s = thickness of the steel plates, in. (mm)

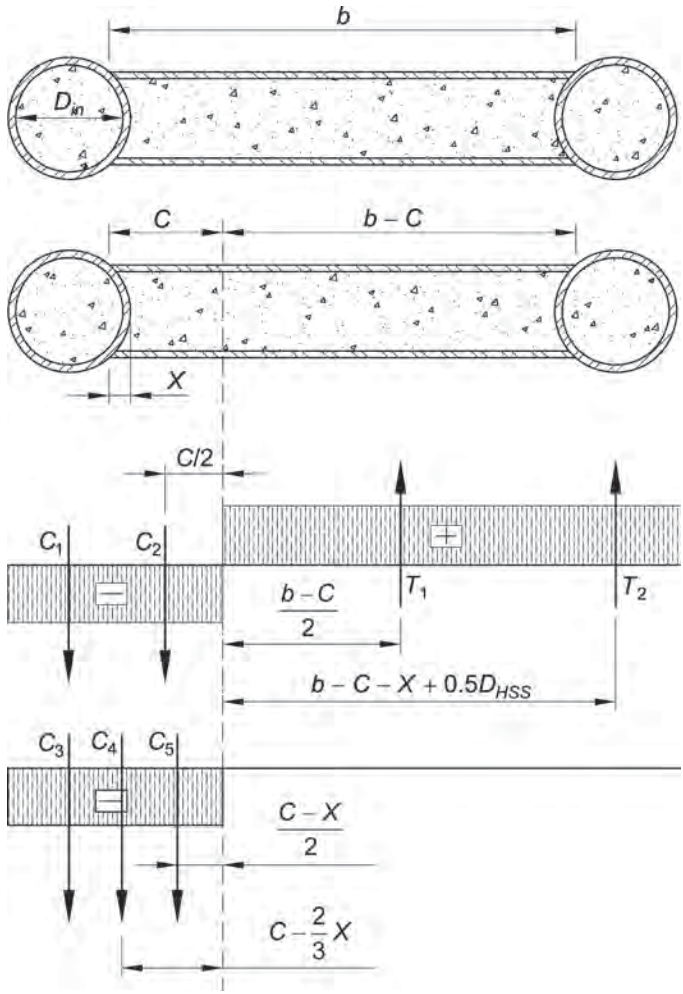


Fig. C-H7.6. Schematic diagram for stress distribution on C-PSW/CF cross section.

5b. Shear Strength

The shear strength of C-PSW/CF with boundary elements can be calculated using the composite contribution of the steel web plates and the cracked concrete. The shear strength of C-PSW/CF without boundary elements can be conservatively calculated as that provided by the steel plates alone without accounting for the contribution of the cracked concrete infill.

The in-plane shear behavior of the C-PSW/CF is governed by the plane stress behavior of the steel faceplates and the orthotropic elastic behavior of concrete cracked in principal tension. Ozaki et al. (2004) and Seo et al. (2016) developed the fundamental in-plane behavior mechanics-based model for such walls. The in-plane shear strength of concrete filled walls can be estimated as a tri-linear shear force-strain curve. The first part of the curve is before the concrete cracks. The second part is after concrete cracking, but before the steel faceplate yielding. The third part of the curve corresponds to the onset of steel yielding. The shear force corresponding to the onset point is the yield shear strength, S_{xy}^{Y}, of the section, given by

$$V_{ni} = S_{xy}^{Y} = \frac{K_s + K_{sc}}{\sqrt{3K_s^2 + K_{sc}^2}}\left(2t_p F_y\right) \quad \text{(C-H7-9)}$$

where

$$K_s = G2t_p \quad \text{(C-H7-10)}$$

$$K_{sc} = \frac{1}{\dfrac{4}{0.7E_c t_c} + \dfrac{2(1-\nu)}{E_s 2t_p}} \quad \text{(C-H7-11)}$$

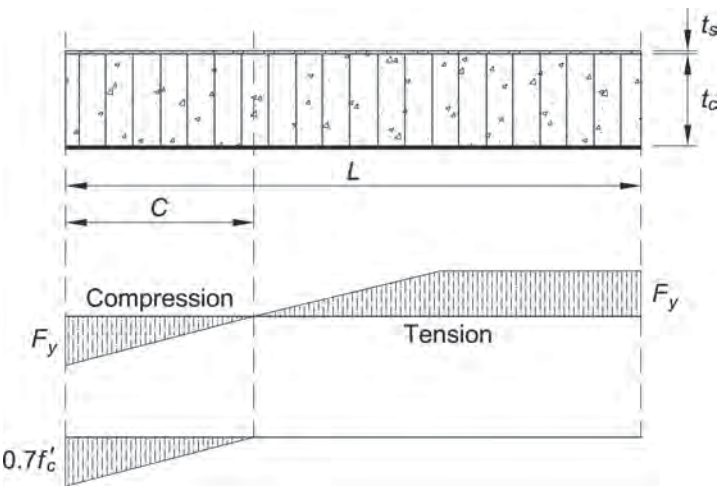


Fig. C-H7.7. Stress distribution for moment strength of C-PSW/CF without boundary elements (Kurt et al., 2016).

This equation was calibrated to the simplified form:

$$V_{ni} = \kappa A_s F_y \tag{C-H7-12}$$

where

$$\kappa = 1.11 - 5.16\bar{\rho} \tag{C-H7-13}$$

$$\bar{\rho} = \frac{A_{sw} F_{yw}}{\sqrt{1000 f_c'} A_{cw}} \tag{C-H7-14}$$

Varma et al. (2014) compared the in-plane shear strength of specimens predicted by the mechanics-based model with the experimental results. Figure C-H7.8 shows that the calculated and experimental values match closely, with the calculated (mechanics-based model) values being conservative.

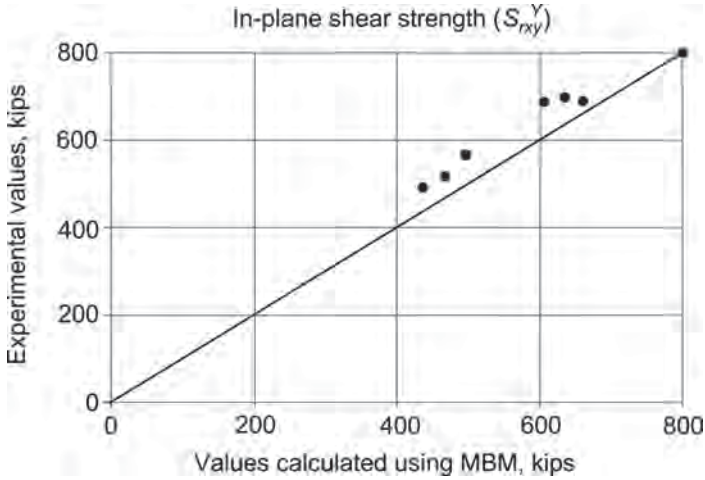


Fig. C-H7.8. Experimental versus calculated values of in-plane shear strength (Varma et al., 2014).

CHAPTER I
FABRICATION AND ERECTION

I1. SHOP AND ERECTION DRAWINGS

The *Code of Standard Practice for Steel Buildings and Bridges,* ANSI/AISC 303, Section 4.2.1(a) (AISC, 2016c) requires the transfer of information from the contract documents (design drawings and project specifications) into accurate and complete approval documents. Therefore, relevant items in the design drawings and project specifications that must be followed in fabrication and erection should be placed on the shop and erection drawings, or in typical notes issued for the project.

3. Shop and Erection Drawings for Composite Construction

For reinforced concrete and composite steel-concrete construction, it is recommended that the following provisions be satisfied: *Details and Detailing of Concrete Reinforcement,* ACI 315 (ACI, 1999), *Manual of Engineering and Placing Drawings for Reinforced Concrete Structures,* ACI 315R (ACI, 2004a), and *ACI Detailing Manual,* ACI SP-66 (ACI, 2004b), including modifications required by Chapter 18 of the *Building Code Requirements for Structural Concrete and Commentary,* ACI 318 (ACI, 2014) and *Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures,* ACI 352 (ACI, 2002).

I2. FABRICATION AND ERECTION

1. Protected Zone

Stress concentrations could lead to fracture in regions of high plastic strain; therefore there is a prohibition on placement of welded attachments in the protected zone. Arc spot welds (puddle welds) associated with the attachment of steel deck to structural steel do not produce high stress concentrations. The performance of full-scale moment connection specimens with arc spot welds in a pattern typical of deck attachment was unaffected by the arc spot welds (Toellner et al., 2015). In addition, a series of tests conducted on full-scale moment connection specimens with 0.177-in. (4.5mm)-diameter full-tip knurled shank power-actuated fasteners applied in a pattern typical of deck attachment or grid patterns with 1-in. (25 mm) edge distance and 2-in. (50 mm) spacing satisfied SMF qualification criteria (Toellner et al., 2015). Negligible differences were found in the cyclic load-displacement envelope (backbone), energy dissipation, and strength degradation prior to fracture as compared to specimens with no fasteners. For these reasons, arc spot welds and power-actuated fasteners up to 0.18-in. (4.6 mm) diameter are allowed for deck attachment.

While welds and power-actuated fasteners used to attach deck in typical patterns are permitted, such attachments are prohibited when used for other applications. In other applications the attachments could be installed by tradespersons who are not subject to the same quality control (QC) and quality assurance (QA) standards that

are required for structural steel. The prohibition reflects potential lack of control and inspection to ensure that attachments are provided consistent with the conditions of the testing cited above.

The exception permits the engineer of record to designate or approve attachments within the protected zone. Fastening or welding close to, or at, a component edge or with close spacing should not be allowed. Appropriate QC and QA should be required for any attachments within the protected zone.

Erection aids and attachments to meet safety requirements may be necessary in the protected zone. If erection aids or other attachments are required to be placed within the protected zone, good welding practices, including proper preheat, should be used. It may be necessary to remove the erection aid or attachment afterwards, and the surfaces of the protected zone may need to be further smoothed by grinding to remove any notch effects. In these and other such cases, the protected zone must be repaired. All such repairs must be approved by the engineer to ensure that severe stress concentrations would not cause a fracture during a seismic event.

2. Bolted Joints

The default installation requirement for high-strength bolts in the *Specification* is to the snug-tightened condition. In Section D2.2, the default condition for bolted connections in the SFRS is pretensioned bolts with faying surfaces of Class A slip coefficient or higher.

3. Welded Joints

As with the 2010 edition, these Provisions make reference to AWS D1.8/D1.8M for welded connection details, replacing such details stated in Appendix W of the 2005 edition.

Because the selection and proper use of welding filler metals is critical to achieving the necessary levels of strength, notch toughness, and quality, the review and approval of welding procedure specifications is required. The engineer of record may use outside consultants to review these documents, if needed.

Welds are sometimes specified for the full length of a connection. Weld tabs are used to permit the starts and stops of the weld passes to be placed outside the weld region itself, allowing for removal of the start and stop conditions and their associated discontinuities. Because the end of the weld, after tab removal, is an outside surface that needs to be notch-free, proper removal methods and subsequent finishing is necessary.

At continuity plates, the end of the continuity plate to column flange weld near the column flange tip permits the use of a full weld tab, and removal is generally efficient if properly detailed. It is permitted to allow ¼ in. (6 mm) of weld tab material to remain at the outboard end of the continuity plate-to-column weld ends because the strain demand placed on this weld is considerably less than that of a beam-to-column flange weld, and the probability of significant weld discontinuities with the distance

permitted is small. Also, complete weld tab removal at beam-to-column joints is required to facilitate magnetic particle testing required by Section J6.2f, but such testing is not required for continuity plate welds. At the opposite end of the continuity plate to column flange weld, near the column radius, weld tabs are not generally desirable and may not be practicable because of clip size and *k*-area concerns. Weld tabs at this location, if used, should not be removed because the removal process has the potential to cause more harm than good.

CHAPTER J

QUALITY CONTROL AND QUALITY ASSURANCE

J1. SCOPE

Specification Chapter N contains requirements for Quality Control (QC) and Quality Assurance (QA) for structural steel and composite construction. Users should also refer to the Commentary of *Specification* Chapter N for additional information regarding these QC and QA requirements, which are applicable to work addressed in the *Specification*, and are also applicable to the seismic force resisting system (SFRS). These Provisions add requirements that are applicable only to the SFRS.

To assure ductile seismic response, steel framing is required to meet the quality requirements as appropriate for the various components of the structure. The applicable building code may have specific quality assurance plan (QAP) requirements, also termed a statement of special inspections. The quality assurance plan should include the requirements of Chapter J.

Specification Section N6 permits waiver of QA when the fabricator or erector is approved by the authority having jurisdiction (AHJ) to do the work without QA. Under the scope of this edition of the Provisions, QC is a requirement whether or not invoked. QA is a requirement when invoked by the AHJ, applicable building code (ABC), purchaser, owner or engineer of record (EOR).

The Provisions, *Specification*, ANSI/AISC 303 *Code of Standard Practice for Steel Buildings and Bridges*, (AISC, 2016c), AWS D1.1/D1.1M, *Structural Welding Code—Steel* (AWS, 2015), and the RCSC *Specification for Structural Joints Using High-Strength Bolts* (RCSC, 2014) provide inspection and acceptance criteria for steel building structures.

The QAP is typically prepared by the engineer of record, and is a part of the contract documents. Chapter J provides the minimum acceptable requirements for a QAP that applies to the construction of welded joints, bolted joints and other details in the SFRS. The engineer of record should evaluate what is already a part of the contractor's quality control system in determining the quality assurance needs for each project. Where the fabricator's quality control system is considered adequate for the project, including compliance with the special needs for seismic applications, the QAP may be modified to reflect this. Similarly, where additional needs are identified, such as for innovative connection details or unfamiliar construction methods, supplementary requirements should be specified, as appropriate. The QAP as contained in this chapter is recommended for adoption without revision because consistent application of the same requirements is expected to improve reliability in the industry.

The QAP should be provided to the fabricator and erector as part of the bid documents, as any special quality control or quality assurance requirements may have substantial impact on the cost and scheduling of the work.

Structural observation at the site by the engineer of record or other design professional is an additional component of a QAP that is not addressed as part of this chapter, and should be developed based upon the specific needs of the project.

A QAP, similar to that required for all-steel structures, should be developed for composite structures and components. For the reinforced concrete portion of the work, in addition to the requirements in ACI 318 Section 26.13, attention is called to the ACI *Detailing Manual* (ACI, 2004b), with emphasis on the provisions of, ACI 121R *Guide for Concrete Construction Quality Systems in Conformance with ISO 9001* (ACI, 2008).

J2. FABRICATOR AND ERECTOR DOCUMENTS

1. Documents to be Submitted for Steel Construction

(a) through (d) and (f): The selection and proper use of welding filler metals is critical to achieving the necessary levels of strength, notch toughness and quality, and submittal to the engineer of welding filler metal documentation and welding procedure specifications (WPS) is required. Submittal allows a thorough review on the part of the engineer, and allows the engineer to use outside consultants to review these documents, if needed.

In the *Specification*, welding filler metal documentation and WPS are to be available for review. In the Provisions, these items must be submitted because the performance of the welded joints that transfer load in the SFRS may affect overall building performance in a seismic event. Also, the engineer's approval of the WPS is a requirement of the Provisions (see Section I2.3), but is not a requirement in the *Specification*.

(e) Bolt installation procedures include instructions for pre-installation verification testing by the fabricator's or erector's personnel, and instructions for installing the bolts using the method chosen for pretensioning (commonly turn-of-nut method, twist-off type tension control bolt method, direct tension indicator method, or calibrated wrench method). In the Provisions, these items must be submitted because the performance of the bolted joints that transfer load in the SFRS may affect overall building performance in a seismic event.

2. Documents to be Available for Review for Steel Construction

It is permitted to have some documents reviewed at the fabricator's or erector's facility by the engineer or designee, such as the QA Agency. The engineer may require submittal of these documents. The one year retention of the documents following substantial completion is to ensure their availability for further review until occupancy is permitted, and for a period following occupancy should issues arise, without placing an undue storage burden on the holder of the documents.

3. Documents to be Submitted for Concrete Construction

The items listed concern concrete and reinforcing steel embedded in the concrete, items that are outside the scope of the definition of structural steel as defined in

ANSI/AISC 303. Therefore, these documents are to be prepared and submitted by the contractor responsible for providing or installing these items.

4. Documents to be Available for Review for Composite Construction

It is permitted to have some documents reviewed at the responsible contractor's facility by the engineer or designee, such as the QA Agency. The engineer may require submittal of these documents. The one year retention of these documents following substantial completion is to ensure their availability for further review until occupancy is permitted, and for a period following occupancy should issues arise, without placing an undue storage burden on the holder of the documents.

J3. QUALITY ASSURANCE AGENCY DOCUMENTS

QA Agencies should have internal procedures (written practices) that document how the Agency performs and documents inspection and testing. ASTM E329, *Standard Specification for Agencies Engaged in Construction Inspection, Testing, or Special Inspection* (ASTM, 2014), is commonly used as a guide in preparing and reviewing written practices. ASTM E329 defines the minimum requirements for inspection agency personnel or testing agency laboratory personnel, or both, and the minimum technical requirements for equipment and procedures utilized in the testing and inspection of construction and materials used in construction. Criteria are provided for evaluating the capability of an agency to properly perform designated tests on construction materials, and establish essential characteristics pertaining to the organization, personnel, facilities and quality systems of the agency. It can be used as a basis to evaluate an agency and is intended for use in qualifying and/or accrediting agencies, public or private, engaged in the testing and inspection of construction materials, including steel construction.

J4. INSPECTION AND NONDESTRUCTIVE TESTING PERSONNEL

Personnel performing welding inspection and nondestructive testing (NDT) should be qualified to perform their designated tasks, whether functioning in a role as QC or QA. Standards are available that provide guidance for determining suitable levels of training, experience, knowledge and skill for such personnel. These standards are typically included in a written practice used by QA agencies. They may be used as a part of a fabricator's or erector's QC program.

For personnel performing bolting inspection, no standard currently exists that provides guidance as to suitable levels of training, experience, knowledge or skill in performing such tasks. Therefore, the QA agency's written practice should contain the agency's criteria for determining their personnel qualifications to perform bolting inspection. Similarly, a fabricator's or erector's QC program should contain their criteria for bolting inspector qualification.

J5. INSPECTION TASKS

Chapter J defines two inspection levels for required inspection tasks and labels them as either observe or perform. This is in contrast to common building code terminology which use or have used the terms periodic or continuous. This change in terminology reflects the multi-task nature of welding and high strength bolting operations, and the required inspections during each specific phase.

1. Observe (O)

The *Specification* defines and uses the observe function in the same manner as used in the Provisions; however, to reflect the higher demand on and the consequence of failure of connections in the SFRS, these inspections are to be performed on a daily basis as a minimum.

2. Perform (P)

The *Specification* defines and uses the perform function in the same manner as used in the Provisions. There is no requirement to make perform inspections on a daily basis, as is required for observe functions, because the perform functions are specific tasks to be completed prior to final acceptance of the designated item, and need be performed at that time.

3. Document (D)

Inspection reports and nonconformance reports are required. The *Specification* contains limited requirements for documentation by QA of the types of inspections performed, including NDT. The Provisions require specific reporting of inspections in the same manner, but add requirements for both QC and QA reports for specific inspection tasks as described in the Document columns in the tables contained in Sections J6, J7, J8, J9 and J10.

J6. WELDING INSPECTION AND NONDESTRUCTIVE TESTING

1. Visual Welding Inspection

Visual inspection by a qualified inspector prior to, during, and after welding is emphasized 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 of bevels, gaps, alignment and other variables. During welding, adherence to the welding procedure specification (WPS) is maintained. After the joint is welded, it is then visually inspected to the requirements of AWS D1.1/D1.1M.

The commentary to *Specification* Section N5.4 on welding inspection contains extensive discussion regarding the observation of welding operations, including the determination of suitable intervals for performing such inspections. Welds in the SFRS should be considered for higher levels of observation, compared to welds not in the SFRS and addressed by *Specification* Chapter N. Welds designated demand

critical within the SFRS should be considered as warranting higher levels of observation, compared to other welds not designated demand critical within the SFRS.

2. NDT of Welded Joints

The use of nondestructive testing methods as required by this section is recommended to verify the soundness of welds that are subject to tensile loads as a part of the SFRS, or to verify that certain critical elements do not contain significant notches that could cause failure. Ultrasonic testing (UT) is capable of detecting serious embedded flaws in groove welds in all standard welded joint configurations. UT is not suitable for inspecting most fillet welds and smaller partial-joint-penetration (PJP) groove welds, nor should it be relied upon for the detection of surface or near-surface flaws. Magnetic particle testing (MT) is capable of detecting serious flaws on or near the surface of all types of welds, and is used for the surface examination of critical groove welds. The use of penetrant testing (PT) is not recommended for general weld inspection, but may be used for crack detection in specific locations such as weld access holes, or for the location of crack tips for cracks detected visually.

2a. CJP Groove Weld NDT

UT is used to detect serious embedded flaws in groove welds, but is not suitable for the detection of surface or near-surface flaws. MT is used to detect serious flaws on or near the surface of these welds. Because visual inspection is also implemented for all CJP groove welds, thus detecting the most serious surface defects, MT is performed at a rate of 25%.

2b. Column Splice and Column-to-Base Plate PJP Groove Weld NDT

Ultrasonic inspection (UT) of PJP groove welds is possible. However, interpretation of the results can be difficult. The *Specification* applies a 0.6 reduction factor to the available strength of PJP groove welds subjected to tension in lieu of UT inspection. However the prescriptive column splice detail utilizing PJP groove welds permitted for IMF, SMF, and STMF will subject the welds to demands in excess of what is permitted by the *Specification*, and the consequence of failure on the column splice weld would be essentially identical whether designated as a CJP or PJP groove weld. These PJP welds are also designated demand critical. Therefore, the same rate of UT for PJP groove welds is required as that for CJP groove welds.

It is also recognized that UT is usually not suitable for use with fillet welds and smaller partial-joint-penetration (PJP) groove welds. PJP groove welds used in column splices for IMF, SMF and STMF are assumed to have a weld size (throat) similar to that of a CJP groove weld, once consideration is made for the added welding to build out to the thicker lower flange.

To address the difficulties associated with UT of PJP groove welds, UT technicians should be qualified in accordance with AWS D1.8/D1.8M using weld joint mock-ups incorporating PJP groove welds.

The use of UT for PJP welds, for conditions other than the PJP groove welds permitted for column splices in IMF, SMF, and STMF, should generally be discouraged. Column-to-base plate welds are usually similar to that of a column splice as far as demand and consequence of failure. However, UT of a PJP groove weld at the column base T-joint will be more difficult than at a column splice butt joint, thus the PJP detail is not recommended at column bases.

2c. Base Metal NDT for Lamellar Tearing and Laminations

Lamellar tearing is the separation (tearing) of base metal along planes parallel to a rolled surface of a member. The tearing is the result of decohesion of "weak planes," usually associated with elongated "stringer" type inclusions, from the shrinkage of large weld metal deposits under conditions of high restraint, applying stress in the through-thickness direction of the base metal.

Lamellar tears rarely occur when the weld size is less than about ¾ to 1 in. (19 to 25 mm). Typically, inclusions located deeper from the surface than $t/4$ do not contribute to lamellar tearing susceptibility.

An appropriate criterion for laminations in SFRS connections does not exist in current standards. Although AWS D1.1/D1.1M Table 6.2 criteria has been written and is applicable to weld metal, not base metal, the use of Table 6.2 criteria has been deliberately selected as conservative acceptance criteria for laminations in these applications, immediately adjacent to and behind the weld.

2d. Beam Cope and Access Hole NDT

The stress flow near and around weld access holes is very complex, and the stress levels are very high. Notches serve as stress concentrations, locally amplifying this stress level which can lead to cracking. The surface of the weld access hole must be smooth, free from significant surface defects. Both PT and MT are capable of detecting unacceptable surface cracks.

2e. Reduced Beam Section Repair NDT

Because plastic straining and hinging, and potentially buckling, takes place in the thermally cut area of the reduced beam section (RBS), the area must be free of significant notches and cracks that would serve as stress concentrations and crack initiation sites. Inadvertent notches from thermal cutting, if sharp, may not be completely removed if relying solely upon visual inspection. If a welded repair is made, NDT is performed to verify that no surface or subsurface cracks have been caused by the repair.

2f. Weld Tab Removal Sites

Because weld tabs serve as locations for the starting and stopping of welds, and are therefore likely to contain a number of weld discontinuities, they are removed. To ensure that no significant discontinuities present in the tab extend into the finished weld itself, MT is performed. Any weld end discontinuities would be present at the

surface of the joint, and therefore would be more detrimental to performance than an embedded discontinuity.

J7. INSPECTION OF HIGH-STRENGTH BOLTING

The commentary to *Specification* Section N5.6 on bolting inspection contains extensive discussion regarding the observation of bolting operations. Bolts in the SFRS should be considered for higher levels of observation compared to bolts not in the SFRS and addressed by *Specification* Chapter N.

J8. OTHER STEEL STRUCTURE INSPECTIONS

Specification Section N5.8 provides for general inspection of the details of the steel frame, which would include those members in the SFRS, as well as anchor rods. Provisions Section J8 adds inspection of specific details unique to seismic construction.

J9. INSPECTION OF COMPOSITE STRUCTURES

J10. INSPECTION OF H-PILES

The *Specification* contains no inspection requirements for piling, as piling is not considered structural steel in ANSI/AISC 303. The Provisions address only steel H-piles that are part of the SFRS. The inspection is limited to verification of the protected zone. Piling materials, pile driving, embedment, etc. are not included. Where welded joints in piling occur, inspections should be performed as for welding of other structural steel as described in Section J6.

CHAPTER K

PREQUALIFICATION AND CYCLIC QUALIFICATION TESTING PROVISIONS

K1. PREQUALIFICATION OF BEAM-TO-COLUMN AND LINK-TO-COLUMN CONNECTIONS

1. Scope

Section K1 describes requirements for prequalification of beam-to-column connections in special moment frames (SMF), intermediate moment frames (IMF), composite special moment frames (C-SMF), and composite intermediate moment frames (C-IMF) and of link-to-column connections in eccentrically braced frames (EBF). The concept of prequalified beam-to-column connections for SMF and IMF, as used in the Provisions, was originally adopted from FEMA 350 (FEMA, 2000a), and was subsequently extended to include prequalified link-to-column connections for EBF. In the 2016 edition of the Provisions, the prequalification of beam-to-column connections was further extended to include C-SMF and C-IMF.

Following observations of moment connection damage in the 1994 Northridge earthquake, these Provisions adopted the philosophy that the performance of beam-to-column and link-to-column connections should be verified by realistic-scale cyclic testing. This philosophy is based on the view that the behavior of connections under severe cyclic loading, particularly in regard to the initiation and propagation of fracture, cannot be reliably predicted by analytical means alone. Consequently, the satisfactory performance of connections must be confirmed by laboratory testing conducted in accordance with Section K2. In order to meet this requirement, designers fundamentally have two options. The first option is to provide substantiating test data, either from project specific tests or from tests reported in the literature, on connections matching project conditions within the limits specified in Section K2. The second option available to designers is to use a prequalified connection.

The option to use prequalified connections in the Provisions does not alter the fundamental view that the performance of beam-to-column and link-to-column connections should be confirmed by testing. However, it is recognized that requiring designers to provide substantiating test data for each new project is unnecessarily burdensome, particularly when the same connections are used on a repeated basis that have already received extensive testing, evaluation and review.

It is the intent of the Provisions that designers be permitted to use prequalified connections without the need to present laboratory test data, as long as the connection design, detailing and quality assurance measures conform to the limits and requirements of the prequalification. The use of prequalified connections is intended to simplify the design and design approval process by removing the burden on designers to present test data, and by removing the burden on the authority having jurisdiction

to review and interpret test data. The use of prequalified connections is not intended as a guarantee against damage to, or failure of, connections in major earthquakes. The engineer of record in responsible charge of the building, based upon an understanding of and familiarity with the connection performance, behavior and limitations, is responsible for selecting appropriate connection types suited to the application and implementing designs, either directly or by delegated responsibility.

2. General Requirements

2a. Basis for Prequalification

In general terms, a prequalified connection is one that has undergone sufficient testing, analysis, evaluation and review so that a high level of confidence exists that the connection can fulfill the performance requirements specified in Section E3.6b for SMF, Section E2.6b for IMF, Section F3.6e for EBF, Section G3.6b for C-SMF, and Section G2.6b for C-IMF. Prequalification should be based primarily on laboratory test data, but supported by analytical studies of connection performance and by the development of detailed design criteria and design procedures. The behavior and expected performance of a prequalified connection should be well understood and predictable. Further, a sufficient body of test data should be available to ensure that a prequalified connection will perform as intended on a consistent and reliable basis.

Further guidance on prequalification of connections is provided by the commentary for FEMA 350, which indicates that the following four criteria should be satisfied for a prequalified connection:

There is sufficient experimental and analytical data on the connection performance to establish the likely yield mechanisms and failure modes for the connection.

Rational models for predicting the resistance associated with each mechanism and failure mode have been developed.

Given the material properties and geometry of the connection, a rational procedure can be used to estimate which mode and mechanism controls the behavior and deformation capacity (that is, story drift angle) that can be attained for the controlling conditions.

Given the models and procedures, the existing database is adequate to permit assessment of the statistical reliability of the connection.

2b. Authority for Prequalification

While the general basis for prequalification is outlined in Section K1.2a, it is not possible to provide highly detailed and specific criteria for prequalification, considering the wide variety of possible connection configurations, and considering the continually changing state-of-the-art in the understanding of connection performance. It is also recognized that decisions on whether or not a particular connection should be prequalified, and decisions on establishing limits on prequalification, will

ultimately entail a considerable degree of professional engineering judgment. Consequently, a fundamental premise of these Provisions is that prequalification can only be established based on an evaluation of the connection by a panel of knowledgeable individuals. Thus, the Provisions call for the establishment of a connection prequalification review panel (CPRP). Such a panel should consist of individuals with a high degree of experience, knowledge and expertise in connection behavior, design and construction. It is the responsibility of the CPRP to review all available data on a connection, and then determine if the connection warrants prequalification and determine the associated limits of prequalification, in accordance with Section K1. It is the intent of the Provisions that only a single, nationally recognized CPRP be established. To that end, AISC established the AISC connection prequalification review panel (CPRP) and developed, ANSI/AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC, 2010a).

Use of connections reviewed by connection review panels other than the AISC CPRP, as permitted in Section K1.2b, and determined suitable for prequalification status in accordance with the Provisions, is subject to approval of the authority having jurisdiction.

3. Testing Requirements

It is the intent of the Provisions that laboratory test data form the primary basis of prequalification, and that the connection testing conform to the requirements of Section K2. FEMA 350 specifies the minimum number of tests on non-identical specimens needed to establish prequalification of a connection, or subsequently to change the limits of prequalification. However, in the Provisions, the number of tests needed to support prequalification or to support changes in prequalification limits is not specified. The number of tests and range of testing variables needed to support prequalification decisions will be highly dependent on the particular features of the connection and on the availability of other supporting data. Consequently, this section requires that the CPRP determine whether the number and type of tests conducted on a connection are sufficient to warrant prequalification or to warrant a change in prequalification limits. Both FEMA 350 and the Provisions refer to "non-identical" test specimens, indicating that a broad range of variables potentially affecting connection performance should be investigated in a prequalification test program. It may also be desirable to test replicas of nominally identical specimens in order to investigate repeatability of performance prior to and after failure and to demonstrate consistency of the failure mechanism. Individuals planning a test program to support prequalification of a connection are encouraged to consult with the CPRP, in advance, for a preliminary assessment of the planned testing program.

Tests used to support prequalification are required to comply with Section K2. That section requires test specimens be loaded at least to a story drift angle as specified in Sections E3.6b and G3.6b for SMF and C-SMF, Sections E2.6b and G2.6b for IMF and C-IMF, or a link rotation angle as specified in Section F3.4a for EBF. These provisions do not include the additional requirement for connection rotation capacity at failure, as recommended in FEMA 350. For purposes of prequalification, however,

it is desirable to load specimens to larger deformation levels in order to reveal the ultimate controlling failure modes. Prequalification of a connection requires a clear understanding of the controlling failure modes for a connection; in other words, the failure modes that control the strength and deformation capacity of the connection. Consequently, test data must be available to support connection behavior models over the full range of loading, from the initial elastic response to the inelastic range of behavior, and finally through to the ultimate failure of the connection.

The story drift angle developed by a moment connection test specimen is the primary acceptance criterion for a beam-to-column moment connection in a moment frame. In an actual building, the story drift angle is determined as the story displacement divided by the story height, and includes both elastic and inelastic components of deformation. For a test specimen, story drift angle can usually be determined in a straightforward manner from displacement measurements on the test specimen. Guidelines for determining the story drift angle of a connection test specimen are provided by SAC (1997).

When a connection is being considered for prequalification by the CPRP, all test data for that connection must be available for review by the CPRP. This includes data on unsuccessful tests of connections that represent or are otherwise relevant to the final connection. Testing performed on a preliminary connection configuration that is not relevant to the final design need not be submitted. However, parametric studies on weak and strong panel zones of a connection that otherwise match the final connection are examples of developmental tests that should be submitted. Individuals seeking prequalification of a connection are obliged to present the entire known database of tests for the connection. Such data is essential for an assessment of the reliability of a connection. Note that unsuccessful tests do not necessarily preclude prequalification, particularly if the reasons for unsuccessful performance have been identified and addressed in the connection design procedures. For example, if 10 tests are conducted on varying sized members and one test is unsuccessful, the cause for the "failure" should be determined. If possible, the connection design procedure should be adjusted in such a way to preclude the failure and not invalidate the other nine tests. Subsequent tests should then be performed to validate the final proposed design procedure.

4. Prequalification Variables

This section provides a list of variables that can affect connection performance, and that should be considered in the prequalification of connections. The CPRP should consider the possible effects of each variable on connection performance, and establish limits of application for each variable. Laboratory tests or analytical studies investigating the full range of all variables listed in this section are not required and would not be practical. Connection testing and/or analytical studies investigating the effects of these variables are only required where deemed necessary by the CPRP. However, regardless of which variables are explicitly considered in testing or analytical studies, the CPRP should still consider the possible effects of all variables listed in this section, and assign appropriate limits.

5. Design Procedure

In order to prequalify a connection, a detailed and comprehensive design procedure consistent with the test results and addressing all pertinent limit states must be available for the connection. This design procedure must be included as part of the prequalification record, as required in Section K1.6. Examples of the format and typical content of such design procedures can be found in FEMA 350.

6. Prequalification Record

A written prequalification record is required for a prequalified connection. As a minimum, the prequalification record must include the information listed in Section K1.6. The prequalification record should provide a comprehensive listing of all information needed by a designer to determine the applicability and limitations of the connection, and information needed to design the connection. The prequalification record need not include detailed records of laboratory tests or analytical studies. However, a list of references should be included for all test reports, research reports, and other publications used as a basis of prequalification. These references should, to the extent possible, be available in the public domain to permit independent review of the data and to maintain the integrity and credibility of the prequalification process. FEMA 350 (FEMA, 2000a) provides an example of the type and formatting of information needed for a prequalified connection.

For connections prequalified by CPRP, ANSI/AISC 358 serves as the prequalification record.

K2. CYCLIC TESTS FOR QUALIFICATION OF BEAM-TO-COLUMN AND LINK-TO-COLUMN CONNECTIONS

1. Scope

The development of testing requirements for beam-to-column moment connections was motivated by the widespread occurrence of fractures in such connections in the 1994 Northridge earthquake. To improve performance of connections in future earthquakes, laboratory testing is required 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 section has been written with the intent of providing the simplest testing requirements possible, while still providing reasonable assurance that connections tested in accordance with these Provisions will perform satisfactorily in an earthquake. Where conditions in the actual building differ significantly from the test conditions specified in this section, additional testing beyond the requirements herein may be needed to ensure 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 the intent of these Provisions that project-specific connection tests be conducted on a routine basis for building construction projects. Rather, it is anticipated that most projects would use connection details that have been previously prequalified in accordance with Section K1. If connections are being used that have not been prequalified, then connection performance must be verified by testing in accordance with Section K2. However, even in such cases, tests reported in the literature can be used to demonstrate that a connection satisfies the strength and rotation requirements of the Provisions, so long as the reported tests satisfy the requirements of this section. Consequently, it is expected that project-specific connection tests would be conducted for only a very small number of construction projects.

Although the provisions in this section predominantly address 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 that 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. Further, recent research on link-to-column connections (Okazaki et al., 2004b; Okazaki, 2004) has demonstrated that such connections, designed and constructed using pre-Northridge practices, show poor performance in laboratory testing. Consequently, in these Provisions, the same testing requirements are applied to both moment connections and to link-to-column connections. In the 2016 edition of the Provisions, requirements were added for testing beam-to-column connections in C-SMF and C-IMF.

When developing a test program, the designer should be aware that the authority having jurisdiction may impose additional testing and reporting requirements not covered in this section. Examples of testing guidelines or requirements developed by other organizations or agencies include those published by SAC (FEMA, 2000a; SAC, 1997), by the ICC Evaluation Service (ICC, 2008), and by the County of Los Angeles (County of Los Angeles Department of Public Works, 1996). Prior to developing a test program, the appropriate authority having jurisdiction should be consulted to ensure the test program meets all applicable requirements. Even when not required, the designer may find the information contained in the foregoing references to be useful resources in developing a test program.

2. 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 the 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.

Examples of subassemblages used to test link-to-column connections can be found in Hjelmstad and Popov (1983); Kasai and Popov (1986c); Ricles and Popov (1987b); Engelhardt and Popov (1989a); Dusicka and Itani (2002); McDaniel et al. (2002); Arce (2002); and Okazaki et al. (2004b).

3. Essential Test Variables

3a. Sources of Inelastic Rotation

This section is intended to ensure 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 moment 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. An allowance is made for such variations by permitting a 25% variation in the percentage of the total inelastic rotation supplied by a member or connecting element in a test specimen as compared with the design intent of the prototype. Thus, for the example above where 100% of the inelastic rotation in the prototype is expected to be developed by flexural yielding of the beam, at least 75% 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 link-to-column connections in EBF, the type of yielding (shear yielding, flexural yielding, or a combination of shear and flexural yielding) expected in the test

specimen link should be substantially the same as for the prototype link. For example, a link-to-column connection detail which performs satisfactorily for a shear-yielding link $\left(e \le 1.6M_p/V_p\right)$ may not necessarily perform well for a flexural-yielding link $\left(e \ge 2.6M_p/V_p\right)$. The load and deformation demands at the link-to-column connection will differ significantly for these cases.

Satisfying the requirements of this section will require the designer to have a clear understanding of the manner in which inelastic rotation is developed in the prototype and in the test specimen.

One of the key parameters measured in a connection test is the inelastic rotation that can be developed in the specimen. The acceptance criterion in the Provisions is based on story drift angle, which includes both elastic and inelastic rotations. However, inelastic rotation provides an important indication of connection performance in earthquakes and should still be measured and reported in connection tests. Researchers have used a variety of different definitions for inelastic rotation of moment connection test specimens in the past, making comparison among tests difficult. In order to promote consistency in how test results are reported, these Provisions require that inelastic rotation for moment connection test specimens be determined based on the assumption that all inelastic deformation of a test specimen is concentrated at a single point at the intersection of the centerline of the beam with the centerline of the column. With this definition, inelastic rotation is equal to the inelastic portion of the story drift angle. Previously the Provisions defined inelastic rotation of moment connection specimens with respect to the face of the column. The definition has been changed to the centerline of the column to be consistent with recommendations of SAC (SAC, 1997; FEMA, 2000a).

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 determined based upon an analysis of test specimen deformations, and can normally be determined 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 (1987a); Engelhardt and Popov (1989a); and Arce (2002).

3b. 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 ensure 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, 2001). 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. For C-SMF and C-IMF systems, the weight per foot of the structural steel member that forms part of the test beam must adhere to the specified limits. However, there is no limit on the total weight per foot of the beam in the test specimen.

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 moment connection test specimen constructed with a W36×150 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 lb/ft (= 150/0.75). The limits specified in this section have been chosen somewhat arbitrarily based on judgment, as no quantitative research results are available on scale effects.

When choosing a beam size for a test specimen, several other factors should be considered in addition to the depth and weight of the section. One of these factors is the width-to-thickness ratio, b/t, 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 load 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 locations of flange and web buckling, which may in turn initiate a fracture. Consequently, it is desirable to test beams over a range of b/t ratios in order to evaluate these effects. For C-SMF and C-IMF systems, b/t ratios are pertinent to steel members that form part of the composite system. For some composite systems, local buckling of steel members may be restrained by concrete elements. For example, filling a steel tube with concrete or encasing a steel member in concrete may delay the onset and reduce the severity of local buckling. These effects should be considered when designing a test specimen and when considering how test results can be extrapolated to the prototype.

These provisions also require that the depth of the test column be at least 90% of the depth of the prototype column. Tests conducted as part of the SAC program indicated that performance of connections with deep columns may differ from the performance

with W12 and W14 columns (Chi and Uang, 2002). Additional recent research on moment connections with deep columns is reported by Ricles et al. (2004b). For C-SMF and C-IMF systems, this limitation only applies to the depth of the structural steel member that forms part of a composite column, not to the overall depth of the composite column.

In addition to adhering separately to the size restrictions for beams and to the size restrictions for columns, the combination of beam and column sizes used in a test specimen should reasonably reflect the pairing of beam and column sizes used in the prototype. For example, consider a building design that calls for the use of a W36 beam attached to a W36 column. For the connection type proposed for this building, successful tests have been run on specimens using a W36 beam attached to a W14 column, and on other specimens using a W24 beam attached to a W36 column. Thus, test data is available for this connection on specimens meeting the beam size limitations of Section K2.3b, and separately on specimens meeting the column size restrictions of Section K2.3b. Nonetheless, these tests would not be suitable for qualifying this connection for the case of a W36 beam attached to a W36 column, since the combination of beam and column sizes used in the test specimens does not match the combination of beam and column sizes in the prototype, within the limits of Section K2.3b.

3f. Steel Strength for Steel Members and Connection Elements

The actual yield stress 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 moment connection design in which inelastic rotation is developed by yielding of the beam, and the beam has been specified to be of ASTM A36/A36M steel. If the beam has an actual yield stress of 55 ksi (380 MPa), the connection is required to resist a moment that is 50% higher than if the beam had an actual yield stress of 36 ksi (250 MPa). 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 A992/A992M beams, the test beam is required to have a yield stress of at least 47 ksi (324 MPa) ($= 0.85R_yF_y$ for ASTM A992/A992M). This minimum yield stress is required to be exhibited by both the web and flanges of the test beam.

The requirements of this section are applicable only to members or connecting elements of the test specimen that are intended to contribute to the inelastic rotation of the specimen through yielding. The requirements of this section are not applicable to members or connecting elements that are intended to remain essentially elastic.

3i. Welded Joints

The intent of the 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 variables, such as current and voltage, be within the range established by the weld 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/D1.1M. It is not the intent of this section that the electrodes used to make welds in a test specimen must necessarily be the same AWS classification, diameter or brand as the electrodes to be used on the prototype.

4. Loading History

For biaxial loading of columns, the intent is to require that both axes are loaded using a pseudo-statically applied load (variable load) as specified in Section K2.4b. The option to apply simultaneous varying loads using Section K2.4b is not prohibited, although the coordination of the two loading sequences would require judgment, presumably supplied by the CPRP. It does not appear reasonable to try to explain how the loads would be coordinated in the Provisions since different connections might suggest different phasing of the loads. Proponents and reviewers are reminded that coordination of loading must be considered. Although not stated explicitly, biaxially symmetric columns would not require duplicate testing about both axes.

The Provisions require that testing include the most demanding combination of beams for which prequalification is sought. For some systems, particularly composite systems, the "largest beam" might not always represent the most demanding situation.

The Provisions provide an option to apply a variable load about at least one axis while a constant (static) load, equal to the expected demand from the beam in the orthogonal direction, may be applied about the orthogonal axis. The use of a static load, equal to the expected strength of the orthogonal beam, is intended to address the lack of test data demonstrating how and at what magnitude simultaneously variable loads should be applied. The Provisions allow for other loading sequences should alternate loading be deemed more appropriate by the proponent and reviewers.

The loading sequence prescribed in Section K2.4b for beam-to-column moment connections is taken from SAC/BD-97/02, *Protocol for Fabrication, Inspection, Testing, and Documentation of Beam Column Connection Tests and Other Experimental Specimens* (SAC, 1997). This document should be consulted for further details of the loading sequence, as well as for further useful information on testing procedures. The prescribed loading sequence is not intended to represent the demands presented by a particular earthquake ground motion. This loading sequence was developed based on a series of nonlinear time history analyses of steel moment frame structures subjected to a range of seismic inputs. The maximum deformation, as well as the cumulative deformation and dissipated energy sustained by beam-to-column connections in these analyses, were considered when establishing the prescribed loading sequence and the connection acceptance criteria. If a designer conducts a nonlinear time history analysis of a moment frame structure in order to evaluate demands on the beam-to-column

connections, considerable judgment will be needed when comparing the demands on the connection predicted by the analysis with the demands placed on a connection test specimen using the prescribed loading sequence. In general, however, a connection can be expected to provide satisfactory performance if the cumulative plastic deformation, and the total dissipated energy sustained by the test specimen prior to failure are equal to or greater than the same quantities predicted by a nonlinear time-history analysis. When evaluating the cumulative plastic deformation, both total rotation (elastic plus inelastic) as well as inelastic rotation at the connection should be considered. SAC/BD-00/10 (SAC, 2000) can be consulted for further information on this topic.

Section K2.4c specifies the loading sequence for qualifying tests on link-to-column connections and is based on work by Richards and Uang (2003) and Richards (2004).

The loading sequence specified in ATC-24, *Guidelines for Cyclic Seismic Testing of Components of Steel Structures* (ATC, 1992) is considered as an acceptable alternative to those prescribed in Sections K2.4b and K2.4c. Further, any other loading sequence may be used for beam-to-column moment connections or link-to-column connections, as long as the loading sequence is equivalent to or more severe than those prescribed in Sections K2.4b and K2.4c. To be considered as equivalent or more severe, alternative loading sequences should meet the following requirements: (1) the number of inelastic loading cycles should be at least as large as the number of inelastic loading cycles resulting from the prescribed loading sequence; and (2) the cumulative plastic deformation should be at least as large as the cumulative plastic deformation resulting from the prescribed loading sequence.

Dynamically applied loads are not required by the Provisions. Slowly applied cyclic loads, as typically reported in the literature for connection tests, are acceptable for the purposes of the 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 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 practical.

6. Testing Requirements for Material Specimens

Tension testing is required for steel members and connection elements of the test specimen that contribute to the inelastic rotation of the specimen by yielding. These tests are required to demonstrate conformance with the requirements of Section K2.3f, 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 reporting of the upper yield point. Yield strength may be reported using either the 0.2% offset or 0.5% elongation under load. 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% strain offset method based on independent testing using common specimen size for all members is required in this section. This follows the protocol used during the SAC investigation.

Since this tension testing utilizes potentially different specimen geometry, testing protocol, and specimen location, differences from the material test report are to be expected. Appendix X2 of ASTM A6 discusses the variation of tensile properties within a heat of steel for a variety of reasons. Based on previous work, this appendix reports the value of one standard deviation of this variance to be 8% of the yield strength using ASTM standards.

This special testing is not required for project materials as the strength ratios in Table A3.1 were developed using standard producer material test report data. Therefore, supplemental testing of project material should only be required if the identity of the material is in question prior to fabrication.

Only tension tests for steel members and connection elements 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.

For C-SMF and C-IMF specimens, material testing is also required for reinforcing steel and concrete. Because of potentially significant differences in specified concrete compressive strength compared to the actual compressive strength, limits are placed on the degree to which the actual tested compressive strength of concrete in a specimen is allowed to differ from the specified value. An exception to these limits is provided if it can be demonstrated that differences in concrete beyond these limits will not result in unacceptable differences in connection performance between the test specimen and the prototype.

K3. CYCLIC TESTS FOR QUALIFICATION OF BUCKLING-RESTRAINED BRACES

The provisions of this section require the introduction of several new variables. The quantity Δ_{bm} represents both an axial displacement and a rotational quantity. Both quantities are determined by examining the profile of the building at the design story drift, Δ_m, and extracting joint lateral and rotational deformation demands.

Determining the maximum rotation imposed on the braces used in the building may require significant effort. The engineer may prefer to select a reasonable value (in other words, story drift), which can be simply demonstrated to be conservative for

each brace type, and is expected to be within the performance envelope of the braces selected for use on the project.

Two types of testing are referred to in this section. The first type is subassemblage testing, described in Section K3.2, an example of which is illustrated in Figure C-K3.1.

The second type of testing, described in Section K3.3 as brace specimen testing, is permitted to be uniaxial testing.

1. Scope

The development of the testing requirements in the Provisions was motivated by the relatively small amount of test data on buckling-restrained braced frame (BRBF) systems available to structural engineers. In addition, no data on the response of BRBF to severe ground motion is available. Therefore, the seismic performance of these systems is relatively unknown compared to more conventional steel-framed structures.

The behavior of a BRBF differs markedly from conventional braced frames and other structural steel seismic force-resisting systems. Various factors affecting brace performance under earthquake loading are not well understood and the requirement for testing is intended to provide assurance that the braces will perform as required, and also to enhance the overall state of knowledge of these systems.

It is recognized that testing of brace specimens and subassemblages can be costly and time-consuming. Consequently, this section has been written with the intent of providing the simplest testing requirements possible, while still providing reasonable assurance that prototype BRBF based on brace specimens and subassemblages tested in accordance with these provisions will perform satisfactorily in an actual earthquake.

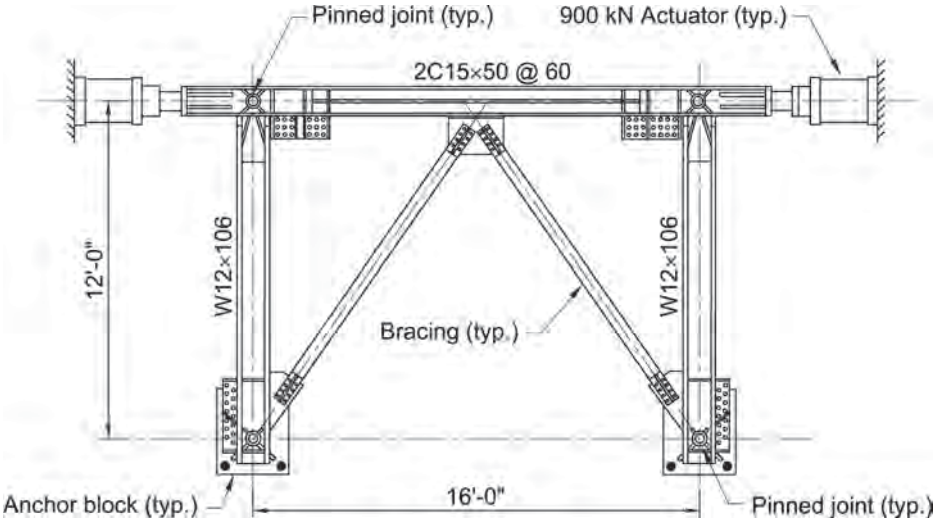


Fig. C-K3.1. Example of test subassemblage.

It is not intended that the Provisions drive project-specific tests on a routine basis for building construction projects. In most cases, tests reported in the literature or supplied by the brace manufacturer can be used to demonstrate that a brace and subassemblage configuration satisfies the strength and inelastic rotation requirements of these Provisions. Such tests, however, should satisfy the requirements of this section.

The Provisions of this section have been written allowing submission of data on previous testing, based on similar conditions. As the body of test data for each brace type grows, the need for additional testing is expected to diminish. The Provisions allow for manufacturer-designed braces, through the use of a documented design methodology.

Most testing programs developed for primarily axial-load-carrying components focus largely on uniaxial testing. However, the Provisions are intended to direct the primary focus of the program toward testing of a subassemblage that imposes combined axial and rotational deformations on the brace specimen. This reflects the view that the ability of the brace to accommodate the necessary rotational deformations cannot be reliably predicted by analytical means alone. Subassemblage test requirements are discussed more completely in Commentary Section K3.2.

Where conditions in the actual building differ significantly from the test conditions specified in this section, additional testing beyond the requirements described herein may be needed to ensure satisfactory brace performance. Prior to developing a test program, the appropriate regulatory agencies should be consulted to ensure the test program meets all applicable requirements.

The brace deformation at first significant yield is used in developing the test sequence described in Section K3.4c. The quantity is required to determine the actual cumulative inelastic deformation demands on the brace. If the nominal yield stress of the steel core were used to determine the test sequence, and significant material overstrength were to exist, the total inelastic deformation demand imposed during the test sequence would be overestimated.

2. Subassemblage Test Specimen

The objective of subassemblage testing is to verify the ability of the brace, and in particular its steel core extension and buckling restraining mechanism, to accommodate the combined axial and rotational deformation demands without failure.

It is recognized that subassemblage testing is more difficult and expensive than uniaxial testing of brace specimens. However, the complexity of the brace behavior due to the combined rotational and axial demands, and the relative lack of test data on the performance of these systems, indicates that subassemblage testing should be performed.

Subassemblage testing is not intended to be required for each project. Rather, it is expected that brace manufacturers will perform the tests for a reasonable range of axial loads, steel core configurations, and other parameters as required by the Provisions. It is expected that this data will subsequently be available to engineers on

other projects. Manufacturers are therefore encouraged to conduct tests that establish the device performance limits to minimize the need for subassemblage testing on projects.

Similar requirements are given in terms of measured axial yield strength of both the prototype and the test specimen braces. This is better suited to manufacturer's product testing than to project-specific testing. Comparison of coupon test results is a way to establish a similarity between the subassemblage test specimen brace and the prototype braces. Once similarity is established, it is acceptable to fabricate test specimens and prototype braces from different heats of steel.

A variety of subassemblage configurations are possible for imposing combined axial and rotational deformation demands on a test specimen. Some potential subassemblages are shown in Figure C-K3.2. The subassemblage need not include connecting beams and columns provided that the test apparatus duplicates, to a reasonable degree, the combined axial and rotational deformations expected at each end of the brace.

Rotational demands may be concentrated in the steel core extension in the region just outside the buckling restraining mechanism. Depending on the magnitude of the rotational demands, limited flexural yielding of the steel core extension may occur. Rotational demands can also be accommodated by other means, such as tolerance in the buckling restraint layer or mechanism, elastic flexibility of the brace and steel core extension, or through the use of pins or spherical bearing assemblies. It is in the engineer's best interest to include in subassemblage testing all components that contribute significantly to accommodating rotational demands.

While the upward extrapolation permitted for brace test specimens in accordance with Section K3.3c(b) is considerable, the subassemblage is not permitted to be much smaller than the prototype. It is expected that the subassemblage test will be reasonably similar to the prototype and thus will provide confirmation of the ability of the design to provide the required performance.

It is intended that the subassemblage test specimen be larger in axial-force capacity than the prototype. However, the possibility exists for braces to be designed with very large axial forces. Should the brace yield force be so large as to make subassemblage testing impractical, the engineer is expected to make use of the Provisions that allow for alternate testing programs, based on building official approval and qualified peer review. Such programs may include, but are not limited to, nonlinear finite element analysis, partial specimen testing, and reduced-scale testing, in combination with full-scale uniaxial testing where applicable or required.

The steel core material was not included in the list of requirements. The more critical parameter, calculated margin of safety for the steel core projection stability, is required to meet or exceed the value used in the prototype. The method of calculating the steel core projection stability should be included in the design methodology.

It is recognized that both test specimens required for brace qualification may have been performed as subassemblage tests given that subassemblage tests are generally considered more demanding than brace specimen tests. In this case there would be

two tests available to determine the factor of safety against overall brace buckling. It is not intended that the more conservative of these must be used in design. Testing facilities often are not large enough to test braces of sufficient length to determine accurate factors of safety for large capacity braces resulting in very conservative factors of safety for overall casing buckling. It is not intended that the more conservative factors of safety dictate design when a more representative subassemblage test is also available.

The subassemblage test specimen is required to undergo combined axial and rotational deformations similar to those in the prototype. It is recognized that identical braces, in different locations in the building, will undergo different maximum axial and rotational deformation demands. In addition, the maximum rotational and axial deformation demands may be different at each end of the brace. The engineer is

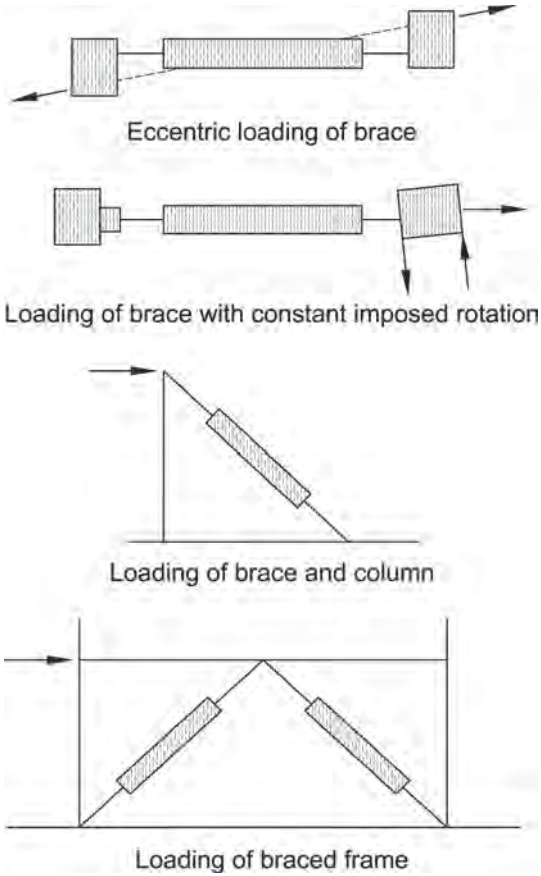


Fig. C-K3.2. Possible test subassemblages.

expected to make simplifying assumptions to determine the most appropriate combination of rotational and axial deformation demands for the testing program.

Some subassemblage configurations will require that one deformation quantity be fixed while the other is varied as described in the test conditions discussed above. In such a case, the rotational quantity may be applied and maintained at the maximum value, and the axial deformation applied according to the required loading sequence. The engineer may wish to perform subsequent tests on the same subassemblage specimen to bound the brace performance.

3. Brace Test Specimen

The objective of brace test specimen testing is to establish basic design parameters for the BRBF system.

The allowance of previous test data (similarity) to satisfy these provisions is less restrictive for uniaxial testing than for subassemblage testing. Subassemblage test specimen requirements are discussed in Commentary Section K3.2.

A considerable number of uniaxial tests have been performed on some brace systems and the engineer is encouraged, wherever possible, to submit previous test data to meet these provisions. Relatively few subassemblage tests have been performed. This type of testing is considered a more demanding test of the overall brace performance.

It is recognized that the fabrication tolerances used by brace manufacturers to achieve the required brace performance may be tighter than those used for other fabricated structural steel members. The engineer is cautioned against including excessively prescriptive brace specifications, as the intent of the Provisions is that the fabrication and supply of the braces is achieved through a performance-based specification process. It is considered sufficient that the manufacture of the test specimen and the prototype braces be conducted using the same quality control and assurance procedures, and the braces be designed using the same design methodology.

The engineer should also recognize that manufacturer process improvements over time may result in some manufacturing and quality control and assurance procedures changing between the time of manufacture of the brace test specimen and of the prototype. In such cases reasonable judgment is required.

During the planning stages of either a subassemblage or uniaxial brace test, certain conditions may exist that cause the test specimen to deviate from the parameters established in the testing section. These conditions may include:

- Lack of availability of beam, column, and brace sizes that reasonably match those to be used in the actual building frame
- Test set-up limitations in the laboratory
- Transportation and field-erection constraints
- Actuator-to-subassemblage connection conditions that require reinforcement of test specimen elements not reinforced in the actual building frame

In certain cases, both the authority having jurisdiction and the peer reviewer may deem such deviations acceptable. The cases in which such deviations are acceptable are project-specific by nature and, therefore, do not lend themselves to further description in this Commentary. For these specific cases, it is recommended that the engineer of record demonstrate that the following objectives are met:

- Reasonable relationship of scale
- Similar design methodology
- Adequate system strength
- Stable buckling-restraint of the steel core in the prototype
- Adequate rotation capacity in the prototype
- Adequate cumulative strain capacity in the prototype

In many cases it will not be practical or reasonable to test the exact brace connections present in the prototype. These provisions are not intended to require such testing. In general, the demands on the steel core extension-to-gusset plate connection are well defined due to the known axial capacity of the brace and the limited flexural capacity of the steel core extension. While the subsequent design of the bolted or welded gusset plate connection is itself a complicated issue and the subject of continuing investigation, it is not intended that these connections become the focus of the testing program.

For the purposes of utilizing previous test data to meet the requirements of this section, the requirements for similarity between the brace and subassemblage brace test specimen can be considered to exclude the steel core extension connection to the frame.

The intent is to allow test data from previous test programs to be presented where possible. See Commentary Section K3.2.

The intent of this provision is to ensure that the end connections of the brace test specimen reasonably represent those of the prototype. It is possible that due to fabrication or assembly constraints, variations in fit-up, faying-surface preparation, or bolt or pin hole fabrication and size may occur. In certain cases, such variations may not be detrimental to the qualification of a successful cyclic test. The final acceptability of variations in brace-end connections rests on the opinion of the building official.

4. Loading History

The loading sequence requires each tested brace to achieve ductilities corresponding to 2.0 times the design story drift and a cumulative inelastic axial ductility capacity of 200 times the yield displacement. Both of these requirements are based on a study in which a series of nonlinear dynamic analyses was conducted on model buildings in order to investigate the performance of this system. The ductility capacity requirement represents a mean of response values (Sabelli et al., 2003). The cumulative ductility requirement is significantly higher than expected for the design basis earthquake, but testing of braces has shown this value to be easily achieved. It is expected

TABLE C-K3.1
Example Brace Testing Protocol

Cycle	Deformation	Inelastic Deformation	Cumulative Inelastic Deformation
2 @ Δ_{by}		$= 2*4*(\Delta_{by} - \Delta_{by}) = 0\Delta_{by}$	$0\Delta_{by} = 0\Delta_{by}$
2 @ ½Δ_{bm}	$= 4$ @ $2.0\Delta_{by}$	$= 2*4*(2.0\Delta_{by} - \Delta_{by}) = 8\Delta_{by}$	$0\Delta_{by} + 8\Delta_{by} = 8\Delta_{by}$
2 @ Δ_{bm}	$= 4$ @ $4.0\Delta_{by}$	$= 2*4*(4.0\Delta_{by} - \Delta_{by}) = 24\Delta_{by}$	$8\Delta_{by} + 24\Delta_{by} = 32\Delta_{by}$
2 @ 1½Δ_{bm}	$= 2$ @ $6.0\Delta_{by}$	$= 2*4*(6.0\Delta_{by} - \Delta_{by}) = 40\Delta_{by}$	$32\Delta_{by} + 40\Delta_{by} = 72\Delta_{by}$
2 @ 2Δ_{bm}	$= 2$ @ $8.0\Delta_{by}$	$= 2*4*(8.0\Delta_{by} - \Delta_{by}) = 56\Delta_{by}$	$72\Delta_{by} + 56\Delta_{by} = 128\Delta_{by}$
4 @ 1½Δ_{bm}	$= 2$ @ $6.0\Delta_{by}$	$= 4*4*(6.0\Delta_{by} - \Delta_{by}) = 80\Delta_{by}$	$128\Delta_{by} + 80\Delta_{by} = 208\Delta_{by}$
Cumulative inelastic deformation at end of protocol = $208\Delta_{by}$			

that as more test data and building analysis results become available these requirements may be revisited.

The ratio of brace yield deformation, Δ_{by}, to the brace deformation corresponding to the design story drift, Δ_{bm}, must be calculated in order to define the testing protocol. This ratio is typically the same as the ratio of the displacement amplification factor (as defined in the applicable building code) to the actual overstrength of the brace; the minimum overstrength is determined by the resistance factor (LRFD) or the safety factor (ASD) in Section F4.5b.2.

Engineers should note that there is a minimum brace deformation demand, Δ_{bm}, corresponding to 1% story drift. Providing overstrength beyond that required to so limit the design story drift may not be used as a basis to reduce the testing protocol requirements. Testing to at least twice this minimum (in other words, to 2% drift) is required.

Table C-K3.1 shows an example brace test protocol. For this example, it is assumed that the brace deformation corresponding to the design story drift is four times the yield deformation; it is also assumed that the design story drift is larger than the 1% minimum. The test protocol is then constructed in accordance with Section K3.4c. In order to calculate the cumulative inelastic deformation, the cycles are converted from multiples of brace deformation at the design story drift, Δ_{bm}, to multiples of brace yield deformation, Δ_{by}. Since the cumulative inelastic drift at the end of the $2.0\Delta_{bm}$ cycles is less than the minimum of $200\Delta_{by}$ required for brace tests, additional cycles to $1.5\Delta_{bm}$ are required. At the end of four such cycles, the required cumulative inelastic deformation has been reached.

Dynamically applied loads are not required by the Provisions. The use of slowly applied cyclic loads, widely described in the literature for brace specimen tests, is 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 apply dynamic loads to large-scale test specimens. Furthermore, the

available research on dynamic loading effects on steel test specimens has not demonstrated a compelling need for such testing.

If rate-of-loading effects are thought to be potentially significant for the steel core material used in the prototype, it may be possible to estimate the expected change in behavior by performing coupon tests at low (test cyclic) and high (dynamic earthquake) load rates. The results from brace tests would then be factored accordingly.

5. Instrumentation

Minimum instrumentation requirements are specified to permit determination of necessary data. It is expected that alternative instrumentation adequate for these purposes will be used in some cases.

6. Materials Testing Requirements

Tension testing of the steel core material used in the manufacture of the test specimens is required. In general, there has been good agreement between coupon test results and observed tensile yield strengths in full-scale uniaxial tests. Material testing required by this section is consistent with that required for testing of beam-to-column moment connections. For further information on this topic, refer to Commentary Section K2.6.

7. Test Reporting Requirements

The results reported are necessary for conformance demonstration and for determination of strain-hardening and compression-overstrength requirements. As nonlinear modeling becomes more common, the production of test data to calibrate nonlinear elements is becoming an important secondary function. Little data exists on the behavior of braces beyond their design range; such information can be useful in verifying the reliability of the system.

8. Acceptance Criteria

The acceptance criteria are written so that the minimum testing data that must be submitted is at least one subassemblage test and at least one uniaxial test. In many cases the subassemblage test specimen also qualifies as a brace test specimen provided the requirements of Section K3.3 are met. If project specific subassemblage testing is to be performed it may be simplest to perform two subassemblage tests to meet the requirements of this section. For the purposes of these requirements a single subassemblage test incorporating two braces in a chevron or other configuration is also considered acceptable.

Depending on the means used to connect the test specimen to the subassemblage or test apparatus, and the instrumentation system used, bolt slip may appear in the load versus displacement history for some tests. This may appear as a series of downward spikes in the load versus displacement plot and is not generally a cause for concern, provided the behavior does not adversely affect the performance of the brace or brace connection.

These acceptance criteria are intended to be minimum requirements. The 1.5 limit in Section K3.8, requirement (d), is essentially a limitation on β based on available test data, where β is the compression strength adjustment factor. Currently available braces should be able to satisfy this requirement.

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