AISC 341-22

Seismic Provisions for Structural Steel Buildings

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Supersedes the *Seismic Provisions for Structural Steel Buildings* dated July 12, 2016, and all previous versions

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
130 East Randolph Street, Suite 2000
Chicago, Illinois 60601-6204

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PREFACE

(This Preface is not a part of ANSI/AISC 341-22, *Seismic Provisions for Structural Steel Buildings*, but is included for informational purposes only.)

The *Specification for Structural Steel Buildings* (ANSI/AISC 360-22) 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 341-22) (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.

This standard adopts *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358-22) by reference. ANSI/AISC 358 specifies design, detailing, fabrication, and quality criteria for connections that are prequalified in accordance with the Provisions for use with special and intermediate moment frames. While the Provisions are modified every six years, ANSI/AISC 358 is processed more frequently, and newer editions of ANSI/AISC 358 may be recognized and enforced by the applicable building code.

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 2016 edition of the Provisions, including the following:

- New presentation of Table A3.1 clarifying allowable grades, strengths, and any other limitations on the material
- Addition of ASTM A709/A709M and ASTM A1066/A1066M to the list of permitted materials for use in seismic force-resisting systems
- Reorganization of items required in the structural design documents and specifications as coordinated with similar revisions to *Specification* Section A4
- Table D1.1 changes, including revised coefficients for all width-to-thickness ratios, clarification for when HSS design thickness is used instead of nominal thickness, and revised width-to-thickness limit equations for webs in I-shaped sections or channels, side plates of boxed I-shaped sections, and webs of box sections
- Provisions for ordinary truss moment frames
- Revisions to SMF continuity plate requirements, including width-to-thickness limits and reduced welding requirements
- Additional requirements and commentary for OCCS and SCCS column bases and column bracing
- Revised provisions for SCBF beams in V- and inverted V-braced frames to permit some limited yielding
- Revised SPSW angle of inclination in terms of its assumed value
- New requirements for coupling beam embedment and reinforcing in C-OSW and C-SSW
- Inclusion of a new system, coupled composite plate shear walls—Concrete Filled (CC-PSW/CF)
- Harmonization of Chapter J with *Specification* Chapter N
- Revised testing extrapolation limits for BRBF

- New Appendix 1, titled “Design Verification Using Nonlinear Response History Analysis”

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).

This specification was approved by the Committee on Specifications:

The Committee honors former members, vice-chair, Patrick J. Fortney, and emeritus member, Duane S. Ellifritt, who passed away during this cycle.

The Committee gratefully acknowledges AISC Board Oversight, Matt Smith; the advisory members Carlos Aguirre and Tiziano Perea for their contributions; and the following task committee and staff members for their involvement in the development of this document:

Abbas Aminmansour
Caroline R. Bennett
Eric Bolin
Mark Braekevelt
Michel Bruneau
Art Bustos
Joel A. Chandler
Shih-Ho Chao
Robert Chmielowski
Douglas Crampton
Mark D. Denavit
Richard M. Drake
Matthew R. Eatherton
Matthew F. Fadden
Larry A. Fahnestock
Shelley C. Finnigan
Timothy P. Fraser
Michael Gannon
Rupa Garai
Jeffrey Gasparott
Rodney D. Gibble
Subhash C. Goel
Arvind V. Goverdhan
Perry S. Green
Christina Harber
Alfred A. Herget
Stephen M. Herlache
Steven J. Herth
Devin Huber
Ronald B. Johnson
Kerry Kreitman
David W. Landis
Chad M. Larson
Dawn E. Lehman
Andres Lepage
Brent Leu
Carlo Lini
LeRoy A. Lutz
Andrew Lye
Bonnie E. Manley
Michael R. Marian
Jason P. McCormick
Patrick S. McManus
Austin A. Meier
Jared Moseley
J.R. Ubejd Mujagic
Kimberley T. Olson
Jeffrey A. Packer
Thomas D. Poulos
Max Puchtel
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Gian Andrea Rassati
Paul W. Richards
Charles W. Roeder
John A. Rolfes
Sougata Roy
Brandt Saxey
Thomas J. Schlafly
Jim Schoen
William Scott
Richard Scruton
Bahram M. Shahrooz
Thomas Sputo
Ryan Staudt
Andrea E. Surovek
James A. Swanson
Matthew Trammell
Robert Tremblay
Sriramulu Vinnakota
Robert Walter
Michael A. West

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SYMBOLS

Definitions for the symbols used in this standard are provided here and reflect the definitions provided in the body of this standard. Some symbols may be used multiple times throughout the document. The section or table number shown in the righthand column of the list identifies the first time the symbol is used in this document. Symbols without text definitions are omitted.

Symbol	Definition	Reference
A_f	Gross area of the flange of the special segment chord member, in.^{2} (mm^2)	E4.4b
A_g	Gross area, in.^{2} (mm^2)	Table D1.1
A_g	Gross area of column, in.^{2} (mm^2)	E3.4a
A_{lw}	Web area of link (excluding flanges), in.^{2} (mm^2)	F3.5b.2
A_s	Cross-sectional area of the structural steel core, in.^{2} (mm^2)	D1.4b.2
A_{sc}	Cross-sectional area of the yielding segment of steel core, in.^{2} (mm^2)	F4.5b.2
A_{sc}	Core area, in.^{2} (mm^2)	K3.2
A_{sh}	Minimum area of hoop reinforcement, in.^{2} (mm^2)	D1.4b.2
A_{sp}	Horizontal area of stiffened steel plate, in.^{2} (mm^2)	H6.3b
A_{sr}	Area of transverse reinforcement, in.^{2} (mm^2)	H4.5b.2
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.4
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)	H4.3
D	Dead load due to the weight of the structural elements and permanent features on the building, kips (N)	D1.4b.2
D	Outside diameter of round HSS, in. (mm)	H7.4b
D	Diameter of the holes, in. (mm)	F5.7a.1
E	Seismic load effect, kips (N)	F1.4a
E	Horizontal seismic load effect, kips (N)	E1.7a
E	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	Table D1.1
E	Modulus of elasticity of the steel beam, ksi (MPa)	G3.5a
E	Vertical and horizontal earthquake effect, kips (N)	F4.4a
E_{cl}	Capacity-limited horizontal seismic load effect, kips (N)	B2
E_{mh}	Horizontal seismic load effect including overstrength, kips (N) or kip-in. (N-mm)	B2
F_{ne}	Nominal stress calculated from *Specification* Chapter E using expected yield stress, ksi (MPa)	F1.6a
F_u	Specified minimum tensile strength, ksi (MPa)	A3.2
F_y	Specified minimum yield stress of the type of steel to be used in the member, 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_y	Specified minimum yield stress of the structural steel core, ksi (MPa)	D1.4b.2
F_y	Specified minimum yield stress of the gusset plate, ksi (MPa)	F2.6c.4
F_y	Specified minimum yield stress of the stiffener, ksi (MPa)	F3.5b.4
F_y	Specified minimum yield stress of the web plate, in.^{2} (mm^2)	F5.7b.1
F_y	Specified minimum yield stress of the steel beam, ksi (MPa)	G3.5a
F_y	Specified minimum yield stress of the plate, ksi (MPa)	H6.5b

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.2
F_{ysr}	Specified minimum yield stress of the transverse reinforcement, ksi (MPa)	D1.4b.2
F_{ysr}	Specified minimum yield stress of transverse reinforcement, ksi (MPa)	H4.5b.2
F_{ystr}	Specified minimum yield stress of transfer reinforcement, ksi (MPa)	H5.5c
G	Shear modulus of steel, ksi (MPa)	H4.3
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 of a chord member of the special segment, 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.1
I_y	Moment of inertia of the plate about the y-axis, in.^{4} (mm^4)	F5.7b.2
K	Effective length factor	F1.5b
L	Live load due to occupancy and moveable equipment, kips (N)	D1.4b.2
L	Length of column, in. (mm)	E3.4c.2
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_{bc}	Length between base and bracing point or between bracing points of a cantilever column where the bracing points are either braced against lateral displacement for both flanges or braced against twist of the cross section, in. (mm)	E6.4b
L_c	Effective length of brace = KL, in. (mm)	F1.5b
L_{cb}	Clear span length of the coupling beam, in. (mm)	H8.5c
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, considered to begin inside the first layer of confining reinforcement, nearest the edge of the wall, in the wall boundary member, in. (mm)	H5.5c
L_e	Embedment length of coupling beam measured from the face of the wall, in. (mm)	H4.5b.1
L_e	Minimum embedment length of coupling beam measured from the face of the wall that provides sufficient connection shear strength based on Equation H4-4 or H4-4M, in. (mm)	H4.5b.1
L_h	Distance between beam plastic hinge locations, as defined within the test report or ANSI/AISC 358, in. (mm)	E2.6d
L_h	Distance between beam plastic hinge locations, in. (mm)	G2.6d
L_s	Length of the special segment, in. (mm)	E4.5c
L_w	Composite shear wall length, in. (mm)	H7.1
M_f	Maximum probable moment at face of column, 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	Lesser plastic moment of the connected members, kip-in. (N-mm)	F2.6d
M_p	Plastic moment, kip-in. (N-mm)	E1.6b
M_p	Plastic moment of a link, kip-in. (N-mm)	F3.4a
M_p	Plastic 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_{pbe}	Expected flexural strength of the steel, concrete-encased, or composite beam, kip-in. (N-mm), determined in accordance with Section G2.6d	D1.2c.2
M_{pc}	Lesser plastic moment of the column sections for the direction in question, kip-in. (N-mm)	D2.5c
M_{pcc}	Plastic moment at the top and bottom ends of the composite column, kip-in. (N-mm)	G2.6f
M_{pcc}	Plastic moment of the smaller composite column, kip-in. (N-mm)	H2.6d
M_{pcc}	Plastic moment of a composite or reinforced concrete column, kip-in. (N-mm)	G3.4a
$M_{p,exp}$	Expected flexural capacity of composite coupling beam, kip-in. (N-mm)	H8.5c
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.1
M_r	Required strength of torsional bracing provided adjacent to plastic hinges, kip-in. (N-mm)	D1.2c.1
M_u	Required strength for torsional bracing provided adjacent to plastic hinges, kip-in. (N-mm)	D1.2c.2
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_1'	Effective moment at the end of the unbraced length opposite from M_2 as determined from *Specification* Appendix 1, kip-in. (N-mm)	E6.4b
M_2	Larger moment at end of unbraced length, kip-in. (N-mm) (shall be taken as positive in all cases)	E6.4b
M^*_{be}	Projection of the expected flexural strength of the beams at the plastic hinge locations to the column centerline, kip-in. (N-mm)	E3.4a
M^*_{pbe}	Projection of the expected flexural strength of the beam at the plastic hinge locations to the column centerline, kip-in. (N-mm)	G3.4a
M^*_{pc}	Projection of the nominal flexural strength 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)	E3.4a
M^*_{pcc}	Projection of the plastic moment of the column (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column, kip-in. (N-mm)	G3.4a
N_r	Number of horizontal rows of perforations	F5.7a.3
P_G	Axial force component of the gravity load, kips (N)	Table A-1.7.1
P_b	Axial design strength of wall at balanced condition, kips (N)	H5.4
P_n	Nominal axial compressive strength, kips (N)	F1.4a
P_n	Nominal axial compressive strength of the composite column calculated in accordance with the *Specification*, kips (N)	D1.4b.2
P_{nc}	Nominal axial compressive strength of the chord member at the ends, kips (N)	E4.4c

P_{nc}	Nominal axial compressive strength of a diagonal member 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 according to Section D1.4a, kips (N)	E3.4a
P_r	Required axial strength of the arching plate in tension resulting from web-plate tension in the absence of other forces, kips (N)	F5.7b.1
P_r	Required axial strength using LRFD or ASD load combinations, kips (N)	Table D1.1
P_r	Required strength of lateral bracing of each flange provided adjacent to plastic hinges, kips (N)	D1.2c.1
P_{rc}	Required axial strength, kips (N)	E3.4a
P_u	Required strength of lateral bracing provided adjacent to plastic hinges, kips (N)	D1.2c.2
P_y	Axial yield strength, kips (N)	F3.5b.2
P_{yc}	Available axial yield strength of column, kips (N)	E3.4a
P_{ysc}	Axial yield strength of steel core, ksi (MPa)	F4.2a
P_{ysc}	Measured yield force of the test specimen, kips (N)	F4.2b
P_{ye}	Expected axial yield strength, kips (N)	Table A-1.7.1
R_c	Factor to account for expected strength of concrete,	A3.2
R_n	Nominal strength, kips (N)	A3.2
R_n	Nominal shear strength, kips (N)	E3.6e.1
R_t	Ratio of the expected tensile strength to the specified minimum tensile strength, F_u, of that material	A3.2
R_y	Ratio of the expected yield stress to the specified minimum yield stress, F_y, of that material	A3.2
R_y	Ratio of the expected yield stress to the specified minimum yield stress of the gusset plate, F_y.	F2.6c.4
R_{yr}	Ratio of the expected yield stress of the transverse reinforcement material to the specified minimum yield stress, to be taken as the R_y value from Table A3.1 for the corresponding steel reinforcement material, F_{ysr}	H5.5d
S_{diag}	Shortest center-to-center distance between holes measured on the 45° diagonal, in. (mm)	F5.7a.1
T_{req}	Tension force, kips (N)	H7.4e
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, kips (N)	H7.4e
T_2	Tension force that develops to prevent splitting of the concrete element on a plane parallel to the steel plate, kips (N)	H7.4e
V_{be}	Expected shear strength of a steel coupling beam computed from Equation H5-2, kips (N)	H5.5c
V_{ce}	Limiting expected shear strength of an encased composite coupling beam, kips (N)	H5.5d
V_n	Nominal shear strength of link, kips (N)	F3.3
V_e	Expected vertical shear strength of the special segment, kips (N)	E4.5c
$V_{n,exp}$	Expected shear strength of composite coupling beam, kips (N)	H8.5c
V_p	Plastic shear strength of a link, kips (N)	F3.4a
V_r	Required shear strength of the connection, kips (N)	E1.6b
V_r	Required shear strength using LRFD or ASD load combinations, kips (N)	F3.5b.3
V_r	Required shear strength, kips (N)	F4.6d
V_u	Required shear strength of the connection, kips (N)	G3.6d
V_y	Shear yield strength, kips (N)	F3.5b.3
Y_{con}	Distance from the top of the steel beam to the top of the concrete, in. (mm)	G3.5a
Y_{PNA}	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.1
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)	E3.6g.5
a	Distance between connectors, in. (mm)	F2.5b
b	Width of compression element as shown in Table D1.1, in. (mm)	Table D1.1
b	Inside width of box section, in. (mm)	F3.5b.5
b	Largest unsupported length of plate between row of steel anchors or ties, in. (mm)	H7.5a
b_{bf}	Width of beam flange, in. (mm)	E3.6f.1
b_c	Clear width of coupling beam flange plate, in. (mm)	H8.5b
b_f	Width of flange of the smaller column connected, in. (mm)	D2.5b
b_f	Link flange width, in. (mm)	F3.5b.4
b_f	Width of beam flange, in. (mm)	E3.4c.1
b_w	Thickness of wall pier, in. (mm)	H4.5b.1
b_w	Width of wall, in. (mm)	H5.5c
b_{wc}	Width of concrete encasement, in. (mm)	H4.5b.2
d	Overall depth of the beam, in. (mm)	G3.5a
d	Overall depth of link, in. (mm)	F3.5b.2
d_c	Effective depth of concrete encasement, in. (mm)	H4.5b.2
d_{tie}	Diameter of tie bar, in. (mm)	H7.5b
d_z	$d - 2t_f$ of the deeper beam at the connection, in. (mm)	E3.6e.2
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)	F3.5b.1
f'_c	Specified compressive strength of concrete, ksi (MPa)	A3.2
g	Clear span of coupling beam, in. (mm)	H4.3
g	Clear span of coupling beam plus the wall concrete cover at each end of the beam, in. (mm)	H5.5c
h	Distance between horizontal boundary element centerlines, in. (mm)	F5.4a
h	Overall depth of composite section, in. (mm)	H4.3
h	Overall depth of the boundary member in the plane of the wall, in. (mm)	H5.5b
h	Width of compression element as shown in Table D1.1, in. (mm)	Table D1.1
h_c	Clear depth of coupling beam web plate, in. (mm)	H8.5b
h_{cc}	Cross-sectional dimension of the confined core measured center-to-center of the transverse reinforcement, in. (mm)	D1.4b.2
h_o	Distance between flange centroids, in. (mm)	D1.2c.1
h_w	Composite shear wall height, in. (mm)	H7.1
r	Governing radius of gyration, in. (mm)	E3.4c.2
r	Radius of the cut out, in. (mm)	F5.7b.1
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.1
r_y	Radius of gyration of individual components about their minor axis, in. (mm)	E4.5e
s	Spacing of transverse reinforcement, in. (mm)	H4.5b.2
s	Spacing of transverse reinforcement measured along the longitudinal axis of the structural member, in. (mm)	D1.4b.2
s_t	Largest center-to-center spacing of the tie bars, in. (mm)	H8.4c
t	Design wall thickness, in. (mm)	Table D1.1
t	Thickness of element as shown in Table D1.1, in. (mm)	Table D1.1
t	Thickness of column web or individual doubler plate, in. (mm)	E3.6e.2
t	Thickness of web plate, in. (mm)	F5.7a.3

t	Thickness of plate, in. (mm)	H7.4a
t	Thickness of the part subjected to through-thickness strain, in. (mm)	J7.2c
t	Thickness of HSS, in. (mm)	H7.4b
t_{bf}	Thickness of beam flange, in. (mm)	E3.4c.1
t_{eff}	Effective web-plate thickness, in. (mm)	F5.7a.3
t_f	Thickness of flange, in. (mm)	F3.5b.2
t_f	Thickness of flange of smaller column connected, in. (mm)	D2.5b
t_f	Thickness of coupling beam flange plate, in. (mm)	H8.5b
t_{lim}	Limiting column flange thickness, in. (mm)	E3.6f.1
t_p	Thickness of the gusset plate, in. (mm)	F2.6c.4
t_{sc}	Total thickness of composite plate shear wall, in. (mm)	H7.4c
t_w	Thickness of web, in. (mm)	F3.5b.2
t_w	Link web thickness, in. (mm)	F3.5b.4
t_w	Thickness of coupling beam web plate, in. (mm)	H8.5b
w_{min}	Minimum of w_1 and w_2, in. (mm)	H7.4c
w_1	Maximum spacing of tie bars in vertical and horizontal directions, in. (mm)	H7.4a
w_1, w_2	Vertical and horizontal spacing of tie bars, respectively, in. (mm)	H7.4c
w_z	Width of panel zone between column flanges, in. (mm)	E3.6e.2
Δ_{DE}	Frame drift corresponding to the design earthquake displacement, in. (mm)	F5.7b
Δ_b	Level of axial or rotational deformation imposed on the test specimen, in. (mm)	K3.4b
Δ_{bm}	Value of deformation quantity, Δ_b, at least equal to that corresponding to the design earthquake displacement, in. (mm)	K3.4c
Δ_{by}	Value of deformation quantity, Δ_b, at first yield of test specimen, in. (mm)	K3.4c
Δ_y	Yield elongation of a diagonal strip of web plate, in. (mm)	App. 1.5.6
Ω_o	Overstrength factor	B2
Ω_t	Overstrength factor for tension	E4.5b
Ω_v	Overstrength factor for shear	E3.6e.1
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.3
α_s	LRFD-ASD force level adjustment factor	Table D1.1
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.1
γ_{total}	Total link rotation angle, rad	K2.4c
δ_{DE}	Design earthquake displacement, in. (mm)	F3.4a
θ	Story drift angle, rad	K2.4b
λ_{hd}, λ_{md}	Limiting width-to-thickness ratio for highly and moderately ductile compression elements, respectively	D1.1b
ϕ_v	Resistance factor for shear	E3.6e.1
ϕ_t	Resistance factor for tension	E4.5b
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 earthquake displacement.

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†. 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 earthquake displacement.

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 of seismic force-resisting system that serves to transfer loads between diaphragms and the members of the vertical 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 displacement. Calculated displacement, taken at a specified point of interest, including the effect of expected inelastic action, due to design level earthquake forces as determined by the applicable building code.

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

Diaphragm plates. Stiffener plates at the top and bottom of the connection region of a filled composite column, either internal or external to the column, or extending through the column, which are used for load transfer in the composite connection.

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.

Frame drift. The story drift at the location of the frame or wall.

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 1-1/2 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 at the design earthquake displacement.

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 (RBS). 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 applicable, 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 mid-length 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)
AHJ (authority having jurisdiction)
AISC (American Institute of Steel Construction)
AISI (American Iron and Steel Institute)
ANSI (American National Standards Institute)
ASCE (American Society of Civil Engineers)
ASD (allowable strength design)
AWS (American Welding Society)
BRBF (buckling-restrained braced frame)
CJP (complete joint penetration)
CPRP (connection prequalification review panel)
C-EBF (composite eccentrically braced frame)
C-IMF (composite intermediate moment frame)
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)
C-PSW/CE (composite plate shear wall—concrete encased)
C-PSW/CF (composite plate shear wall—concrete filled)
C-SCBF (composite special concentrically braced frame)
C-SMF (composite special moment frame)
C-SSW (composite special shear wall)
CC-PSW/CF (coupled composite plate shear wall—concrete filled)
CVN (Charpy V-notch)
EBF (eccentrically braced frame)
EOR (engineer of record)
FCAW (flux cored arc welding)
FEMA (Federal Emergency Management Agency)
FR (fully restrained)
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)
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)
VT (visual testing)
WPQR (welder performance qualification records)
WPS (welding procedure specification)

CHAPTER A

GENERAL REQUIREMENTS

This chapter states the scope of these 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 Documents and Specifications Issued for Construction

A1. SCOPE

The *Seismic Provisions for Structural Steel Buildings*, hereafter referred to as these Provisions, shall apply to the design, fabrication, erection, and quality 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: As specified in ASCE/SEI 7, Section 14.1.2.2.1, buildings with structural steel systems in seismic design categories B and C do not need to meet the requirements of these Provisions provided that they are designed in accordance with the AISC *Specification for Structural Steel Buildings* and the seismic design coefficients and factors of ASCE/SEI 7, Table 12.2-1, Item H. These Provisions do not apply in seismic design category A. ASCE/SEI 7 specifically exempts some systems from the requirements of these Provisions. Further discussion is provided in the Commentary.

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 360-22 *Specification for Structural Steel Buildings*
ANSI/AISC 358-22 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*
ANSI/AISC 342-22 *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*

(b) American Welding Society (AWS)
AWS D1.8/D1.8M:2021 *Structural Welding Code—Seismic Supplement*
AWS B4.0:2016 *Standard Methods for Mechanical Testing of Welds* (U.S. Customary Units)
AWS B4.0M:2000(R2010) *Standard Methods for Mechanical Testing of Welds* (Metric Customary Units)
AWS D1.4/D1.4M:2018 *Structural Welding Code—Steel Reinforcing Bars*

(c) ASTM International (ASTM)
ASTM A615/615M-20 *Standard Specification for Deformed and Plain Carbon Steel Bars for Concrete Reinforcement*
ASTM A706/A706M-16 *Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement*
ASTM C31/C31M-19a *Standard Practice for Making and Curing Concrete Test Specimens in the Field*
ASTM C39/C39M-20 *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*
ASTM E8/E8M-21 *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. Unless a material is determined suitable by testing or other rational criteria to exceed the specified yield stresses described herein, 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, with the following exceptions:

(a) For systems defined in Sections E1 (Ordinary Moment Frames), F1 (Ordinary Concentrically Braced Frames), G1 (Composite Ordinary Moment Frames), H1 (Composite Ordinary Braced Frames), and H4 (Composite Ordinary Shear Walls), this limit shall not exceed 55 ksi (380 MPa), except for columns in the system defined by Section H1, for which the limit of 70 ksi (485 MPa) applies, and except as allowed in exception (b).

(b) For columns in systems defined in Chapter F and Section E3 (Special Moment Frames), E4 (Special Truss Moment Frames), G3 (Composite Special Moment Frames), H2 (Composite Special Concentrically Braced Frames), and H3

(Composite Eccentrically Braced Frames), this limit shall not exceed 70 ksi (485 MPa).

The ASTM materials shown in Table A3.1 are permitted to be used in the SFRS described in Chapters E, F, G, and H.

Table A3.1
Listed Materials Permitted for use in SFRS Described in Chapters E, F, G, and H

Standard Designation	Permissible Grades/Strengths	Other Limitations
(a) Hot-Rolled Shapes		
ASTM A36/A36M	–	–
ASTM A529/A529M	Gr. 50 (345) or Gr. 55 (380)	–
ASTM A572/A572M	Gr. 42 (290), Gr. 50 (345), or Gr. 55 (380)	Type 1, 2, or 3
ASTM A588/A588M	–	–
ASTM A709/A709M,	Gr. 36 (250), Gr. 50 (345), Gr. 50S (345S), Gr. 50W (345W), QST 50 (QST345), QST 50S (QST345S), QST 65 (QST450), or QST 70 (QST485)	–
ASTM A913/A913M	Gr. 50 (345), Gr. 60 (415), Gr. 65 (450), or Gr. 70 (485)	–
ASTM A992/A992M	–	–
ASTM A1043/A1043M	Gr. 36 (250) or Gr. 50 (345)	Gr. 36 (250) or 50 (345) ≤ 2 in. (50 mm); Gr. 50 (345) > 2 in. (50 mm)
(b) Hollow Structural Sections (HSS)		
ASTM A53/A53M	Gr. B	–
ASTM A500/A500M	Gr. B, Gr. C, or Gr. D	–
ASTM A501/A501M	Gr. B	ERW or Seamless
ASTM A1085/A1085M[a]	Gr. A	–
(c) Plates		
ASTM A36/A36M	–	–
ASTM A529/A529M	Gr. 50 (345) or Gr. 55 (380)	–
ASTM A572/A572M	Gr. 42 (290), Gr. 50 (345), or Gr. 55 (380)	Type 1, 2, or 3 ≤ 4 in. (100 mm)
ASTM A588/A588M	–	–
ASTM A709/A709M	Gr. 36 (250), Gr. 50 (345), Gr. 50W (345W),	–
ASTM A1011/A1011M	Gr. 55 (380)	HSLAS
ASTM A1043/A1043M	Gr. 36 (250) or Gr. 50 (345)	Gr. 36 (250) ≤ 2 in. (50 mm)
(d) Bars		
ASTM A36/A36M	–	–
ASTM A529/A529M	Gr 50 (345) or 55 (380)	–
ASTM A572/A572M	Gr 42 (290), Gr. 50 (345), or Gr. 55 (380)	Type 1, 2, or 3
ASTM A709/A709M	Gr. 36 (250), Gr. 50 (345), , Gr. 50W (345W),	–
(e) Sheet		
ASTM A1011/A1011M	Gr. 55 (380)	HSLAS
(f) Steel Reinforcement		
ASTM A615/A615M	Gr. 60 (420) and Gr. 80 (550)	
ASTM A706/A706M	Gr. 60 (420) and Gr. 80 (550)	

– Indicates no restriction applicable on grades/strengths or there are no limitations, as applicable.
[a] ASTM A1085/A1085M material is only available in Grade A, therefore it is permitted to specify ASTM A1085/A1085M without any grade designation.

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. For composite members or adjoining members, as applicable, whose nominal strength is a function of the specified concrete compressive strength, f'_c, the expected strength of an element shall be determined from the expected concrete compressive strength, $R_cf'_c$. R_c is the factor to account for the expected strength of concrete. The value of R_c shall be taken as 1.3.

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. When R_n is a function of f'_c, $R_cf'_c$ is permitted to be used in lieu of f'_c.

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, Rn, 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 A3.2. 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.

TABLE A3.2
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 A709/A709M Gr. 36 (250)	1.5	1.2
• ASTM A1043/A1043M Gr. 36 (250)	1.3	1.1
• ASTM A992/A992M	1.1	1.1
• ASTM A709 Gr. 50S (345S)	1.1	1.1
• ASTM A572/A572M Gr. 50 (345), or 55 (380)	1.1	1.1
• ASTM A709/A709M Gr. 50 (345)	1.1	1.1
• ASTM A913/A913M Gr. 50 (345), 60 (415), 65 (450), or 70 (485)	1.1	1.1
• ASTM A709/A709M QST 50 (QST345), A709/A709M QST 50S (QST345S), A709/A709M QST 65 (QST450), or A709/A709M QST 70 (QST485)	1.1	1.1
• ASTM A588/A588M	1.1	1.1
• ASTM A709/A709M Gr. 50W (345W)	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 Gr. A[a]	1.25	1.15
Plates, Strips, and Sheets:		
• ASTM A36/A36M	1.3	1.2
• ASTM 709/A709M Gr. 36 (250)	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
• A709/A709M Gr. 50 (345)	1.1	1.2
• ASTM A588/A588M	1.1	1.2
• ASTM A709/A709M Gr. 50W (345W)	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. 80 (550)	1.1	1.2
• ASTM A706/A706M Gr. 60 (420) and Gr. 80 (550)	1.2	1.2
[a] ASTM A1085/A1085M material is only available in Grade A, therefore it is permitted to specify ASTM A1085/A1085M without any grade designation.		

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-lbf (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-lbf (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.

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-lbf (J) [a]	20 (27) min. @ 0°F (–18°C)[a]		25 (34) min. @ –20°F (–30°C)[b]

[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.
[b] Filler metals classified as meeting 25 ft-lbf (34 J) min. at a temperature lower than –20°F (–30°C) also meet this requirement.

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: In addition to the requirements in Section A3.4a, 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-lbf (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-lbf (54 J) min. at a temperature lower than +70°F (+20°C) also meet this requirement.

5. Concrete and Steel Reinforcement

Concrete and steel reinforcement used in composite components in composite intermediate or special SFRS of Sections G2, G3, G4, H2, H3, H5, H6, and H7 shall satisfy the requirements of ACI 318, Chapters 18 and 20. 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 and Chapter 20.

A4. STRUCTURAL DESIGN DOCUMENTS AND SPECIFICATIONS ISSUED FOR CONSTRUCTION

Structural design documents 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 this section, as applicable.

1. General

The structural design documents and specifications shall indicate the following general items, as applicable:

(a) Designation of the SFRS

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

(c) Connection details between concrete floor diaphragms and the structural steel elements of the SFRS

(d) Fabrication documents and erection document requirements not addressed in Section I1

2. Steel Construction

The structural design documents and specifications shall include the following items pertaining to steel construction, as applicable:

(a) Configuration of the connections

(b) Connection material specifications and sizes

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

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

(e) Locations of stability bracing members

(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 and dimensions of protected zones

(h) Connection detailing

The structural design documents and specifications shall include the following items pertaining to elements of connection details, as applicable:

(1) Locations of demand critical welds

(2) Locations where weld backing is required to be removed, and where fillet welds are required after backing is removed

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

(4) Locations where weld tabs are required to be removed

(5) Locations where tapered splices are required

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

User Note: These Provisions and ANSI/AISC 358 include requirements related to protected zones, demand critical welds, removal of weld backing and repair after backing is removed, fillet welding of weld backing, weld tab removal, tapered transitions at splices, and special weld access hole geometry. These explicit requirements are considered adequate and effective for the great majority of steel structures and are strongly encouraged to be used without modification. There may be special or unique conditions where supplemental requirements are deemed to be necessary by the engineer of record. In such cases, these project-specific requirements must also be clearly delineated in the structural design documents.

3. Composite Construction

For the steel components of reinforced concrete or composite elements, structural design documents 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 these Provisions.

This chapter is organized as follows:

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 the horizontal seismic load effect including overstrength, 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, 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, E_{mh}, is determined using Equation 12.4-7: $E_{mh} = \Omega_o Q_E$. E_{mh} need not be taken larger than E_{cl}. Therefore, where these Provisions refer to capacity-limited seismic load, 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. However, where these Provisions refer to the overstrength seismic load, E_{mh} is permitted to be taken as calculated in ASCE/SEI 7, Equation 12.4-7, with a maximum value of E_{mh} equal to E_{cl}.

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

Chords, collectors, truss diaphragms, and their connections are part of the seismic force-resisting system and are subject to the requirements of Sections A3, D2.2, and D2.3. 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. Diaphragm, chord, and collector forces resulting from transfer of lateral forces associated with horizontal offsets in the lateral force-resisting system shall use the load combinations in the applicable building code, including capacity-limited seismic forces associated with the transfer.

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.

Exception: 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, ordinary cantilever column systems, special cantilever column systems, or combinations thereof, and the truss diagonal members conform to Sections F1.4b and F1.5, and the 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 design is based on elastic analysis, the stiffness properties of the members and components of steel systems shall be based on elastic section properties 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 and Appendix 1.

User Note: ASCE/SEI 7, Chapter 16, includes requirements for using nonlinear response history analysis procedures for seismic design. ASCE/SEI 7 provides requirements for calculating seismic demands under maximum considered earthquake (MCE) ground motion shaking intensities and acceptance criteria for story drifts, required strengths for force-controlled actions, and inelastic deformations in deformation-controlled components.

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

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.

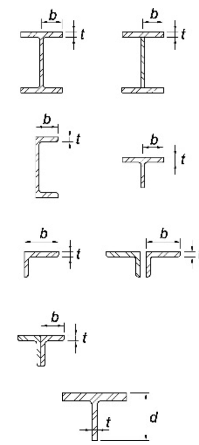
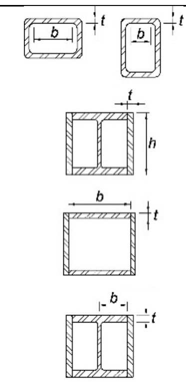
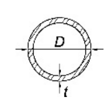
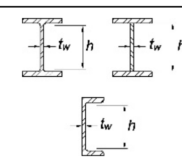
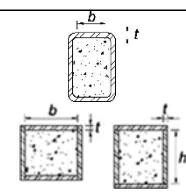
Concrete sections shall comply with the requirements of ACI 318, Section 18.4, for moderately ductile members and ACI 318, Sections 18.6, 18.7, and 18.8, for highly ductile members.

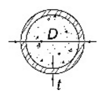
1b. Width-to-Thickness Limitations of Steel and Composite Sections

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

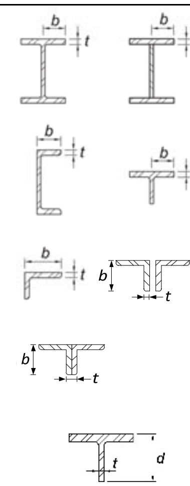
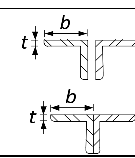
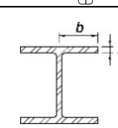
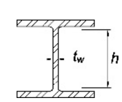
TABLE D1.1a
Width-to-Thickness Ratios: Compression Elements
Diagonal Braces

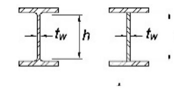
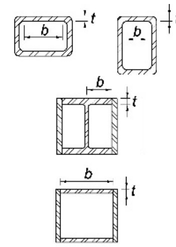
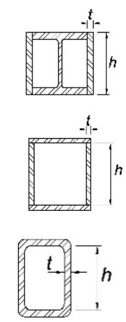
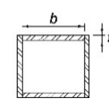
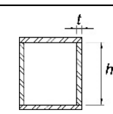
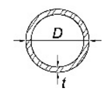
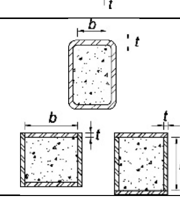
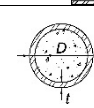
	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio λ_{md} Moderately Ductile Members	Example
	1	1) Flanges of rolled or built-up I-shaped sections 2) Flange and stem of rolled or built-up tees 3) Flanges of rolled or built-up channels 4) Legs of single angles or double-angle members with separators 5) Outstanding legs of pairs of angles in continuous contact	b/t d/t	$0.30\sqrt{\frac{E}{R_yF_y}}$	$0.38\sqrt{\frac{E}{R_yF_y}}$	
Stiffened Elements	2	1) Walls of rectangular HSS[a] 2) Flanges and side plates of boxed I-shaped sections 3) Walls of box sections	b/t h/t	$0.55\sqrt{\frac{E}{R_yF_y}}$	$0.64\sqrt{\frac{E}{R_yF_y}}$	
	3	Walls of round HSS[a]	D/t	$0.038\frac{E}{R_yF_y}$	$0.044\frac{E}{R_yF_y}$	
	4	Webs of rolled or built-up I-shaped sections and channels	h/t_w	$1.49\sqrt{\frac{E}{R_yF_y}}$	$1.49\sqrt{\frac{E}{R_yF_y}}$	
	5	Walls of filled rectangular HSS and box sections.[a]	b/t h/t	$1.4\sqrt{\frac{E}{R_yF_y}}$	$2.26\sqrt{\frac{E}{R_yF_y}}$	

	6	Walls of filled round HSS sections[a]	D/t	$0.076\dfrac{E}{R_yF_y}$	$0.15\dfrac{E}{R_yF_y}$	

[a] The design wall thickness, $0.93t$, shall be used in the calculations involving the wall thickness of hollow structural sections (HSS), as defined in *Specification* Section B4.2.

TABLE D1.1b
Width-to-Thickness Ratios: Compression Elements
All Members Except Diagonal Braces

	Case	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
Unstiffened Elements	7	1) Flanges of rolled or built-up I-shaped sections 2) Flange and stem of rolled or built-up tees 3) Flanges of rolled or built-up channels 4) Legs of single angles or double-angle members with separators 5) Outstanding legs of pairs of angles in continuous contact	b/t d/t	$0.30\sqrt{\dfrac{E}{R_yF_y}}$	$0.38\sqrt{\dfrac{E}{R_yF_y}}$	
Unstiffened Elements	8	Horizontal legs of double-angle members with separators or in continuous contact	b/t	$0.47\sqrt{\dfrac{E}{R_yF_y}}$	$0.54\sqrt{\dfrac{E}{R_yF_y}}$	
Unstiffened Elements	9	Flanges of H-pile sections per Section D4	b/t	not applicable	$0.45\sqrt{\dfrac{E}{R_yF_y}}$	
Stiffened	10	Webs of H-pile sections	h/t_w	not applicable	$1.50\sqrt{\dfrac{E}{R_yF_y}}$	

	11	Where used in beams, columns, or links, as webs in flexure, or combined axial and flexure: Webs of rolled or built-up I-shaped sections and channels	h/t_w	$2.5(1-C_a)^{2.3}\sqrt{\frac{E}{R_yF_y}}$ [b]	$5.4(1-C_a)^{2.3}\sqrt{\frac{E}{R_yF_y}}$ [b]	
	12	Where used in beams or columns as flanges in uniform compression due to flexure or combined axial and flexure: 1) Flanges of rectangular HSS[a] 2) Flanges of boxed I-shaped sections 3) Flanges of box sections	b/t	$0.55\sqrt{\frac{E}{R_yF_y}}$	$1.00\sqrt{\frac{E}{R_yF_y}}$	
	13	Where used in beams, columns, or links, as webs in flexure, or combined axial and flexure: 1) Side plates of boxed I-shaped sections 2) Webs of rectangular HSS[a] 3) Webs of box sections	h/t	For $C_a \le 0.125$[b] $2.45(1-0.93C_a)\sqrt{E/R_yF_y}$ For $C_a > 0.125$ $2.26(1-0.34C_a)\sqrt{E/R_yF_y}$ $\ge 1.49\sqrt{E/R_yF_y}$	For $C_a \le 0.125$[b] $3.76(1-2.75C_a)\sqrt{E/R_yF_y}$ For $C_a > 0.125$ $2.61(1-0.43C_a)\sqrt{E/R_yF_y}$ $\ge 1.49\sqrt{E/R_yF_y}$	
	14	Flanges of box sections used as link beams	b/t	$0.55\sqrt{\frac{E}{R_yF_y}}$	$0.64\sqrt{\frac{E}{R_yF_y}}$	
	15	Webs of box sections used as EBF links	h/t	$0.64\sqrt{\frac{E}{R_yF_y}}$	$1.67\sqrt{\frac{E}{R_yF_y}}$	
	16	Walls of round HSS[a]	D/t	$0.038\frac{E}{R_yF_y}$	$0.07\frac{E}{R_yF_y}$	
Composite	17	Flanges and webs of filled rectangular HSS and box sections.[a]	b/t h/t	$1.4\sqrt{\frac{E}{R_yF_y}}$	$2.26\sqrt{\frac{E}{R_yF_y}}$	
	18	Walls of filled round HSS sections[a]	D/t	$0.076\frac{E}{R_yF_y}$	$0.15\frac{E}{R_yF_y}$	

[a] The design wall thickness, 0.93t, shall be used in the calculations involving the wall thickness of hollow structural sections (HSS), as defined in *Specification* Section B4.2.

[b] $C_a = \frac{\alpha_s P_r}{R_y F_y A_g}$

where

A_g = gross area, in.^{2} (mm^2)
E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_y = specified minimum yield stress, ksi (MPa)
P_r = required axial strength using LRFD or ASD load combinations, kips (N)
R_y = ratio of the expected yield stress to the specified minimum yield stress
α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD

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

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

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

$$L_b = 0.17 r_y E / (R_y F_y) \qquad \text{(D1-3)}$$

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 $L_b = 0.086 r_y E / (R_y F_y)$. 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:

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

where

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

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

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

(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:

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

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

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

of the beam, where

M_{pbe} = 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 $M_u = 0.06M_{pbe}$ 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 $M_r = M_u = M_{pbe}$ 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 region 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 transverse-bar diameters

(4) 12 in. (300 mm)

(b) The maximum spacing of transverse reinforcement at the top and bottom 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) Spacing of transverse reinforcement 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 hoop reinforcement, A_{sh}, shall be:

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

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 transverse reinforcement, 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 transverse 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 structural 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 a minimum length required to fully develop the concrete-encased structural 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.

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 slab 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 is permitted to be calculated according to AISI S310 or 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. Refer to 1926.756(e)(1) of OSHA *Safety and Health Regulations for Construction*, Standards—29 CFR 1926, Subpart R—Steel Erection.

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 moment 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 18.7.4.4.

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 longitudinal reinforcement 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 steel reinforcement as part of the anchorage embedment design, it is important to consider the anchor failure modes and provide reinforcement that meets the development length requirements 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 $0.7F_yZ/(\alpha_sH)$ 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 0.025*H* 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 section 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 meet the development length requirements in ACI 318 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 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 transverse reinforcement when confinement of the lap 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 3/4 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 corresponding to the design earthquake displacement 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.

For H-piles, the connection between the pile cap and piles shall be designed for a tensile force not less than 10% of the pile compression capacity.

Exception: Connection tensile capacity need not exceed the strength required to resist seismic load effects including overstrength. Connections need not be designed for tension where the foundation or supported structure does not rely on the tensile capacity of the piles for stability under design seismic forces including the effects of overstrength.

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 for OMF composed of structural steel beams and columns. OMF composed of structural steel trusses and columns shall satisfy the requirements of Section E1.7.

4. System Requirements

There are no requirements specific to this system for OMF composed of structural steel beams and columns. OMF composed of structural steel trusses and columns shall satisfy the requirements of Section E1.7.

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. OMF composed of structural steel trusses and columns shall satisfy the additional requirements of Section E1.7.

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. OMF truss to column connections shall satisfy the requirements of Section E1.7.

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 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_yM_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_r, shall be determined using the capacity-limited seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, shall be determined as follows:

$$E_{cl} = 2\left(1.1R_yM_p\right)/L_{cf} \tag{E1-1}$$

where

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

Continuity plates shall be provided as required by *Specification* Section J10.

User Note: The permitted welds for the welded joints of the continuity plates to the column flanges include CJP groove welds, two-sided partial-joint-penetration (PJP) groove welds with reinforcing fillet welds, two-sided fillet welds, and combinations of PJP groove welds and fillet welds.

(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.

The continuity plate requirements in Section E1.6b(a) shall apply, except that the required flexural strength used to check for continuity plates shall be the maximum moment that can be transferred to the connection by the system.

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 CJP 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) The required strength of the welded joint of the continuity plate to the column flange shall not be less than the available strength of the contact area of the plate with the column flange. Alternatively, continuity plates shall satisfy the requirements of Section E3.6f.2.

User Note: The permitted welds for these welded joints include CJP groove welds, two-sided partial-joint-penetration (PJP) groove welds with reinforcing fillet welds, two-sided fillet welds, and combinations of PJP groove welds and fillet welds.

(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).

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 between beams and columns 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 PR 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_r shall be determined in accordance with Section E1.6b(a) with M_p in Equation E1-1 taken as $M_{n,PR}$.

7. OMF Composed of Structural Steel Trusses and Structural Steel Columns

A structural steel truss is permitted to be used as the beam in an ordinary moment frame. The truss and connections from the truss to the column shall be designed for end moments consistent with axial yielding in the truss chords, flexural yielding in the column, or yielding of the column in shear in the region of high shear demand between the top and bottom chords of the truss.

7a. Analysis Requirements

The truss members and their connections shall be designed for the capacity-limited horizontal seismic load effect, E_{cl}, defined by the truss end connection forces prescribed in Section E1.7d.

7b. Basis of Design

OMF composed of structural steel trusses and columns are limited to one story structures.

7c. System Requirements

Columns shall be braced out-of-plane at both the top and bottom chord elevations of the moment-connected truss. Stability forces shall be based on the column bracing requirements for panel bracing from *Specification* Appendix 6, Section 6.2.1. Bracing shall be designed to meet beam-column bracing requirements of *Specification* Appendix 6, Section 6.4, using column axial loads and bending moments consistent with connection design forces prescribed in Section E1.7d.

7d. Truss-to-Column Connections

Truss-to-column connections shall be designed to transfer top and bottom chord axial forces equal to or greater than the minimum of:

(a) The expected yield strength in tension of the truss chord section, determined as $R_y F_y A_g / \alpha_s$

(b) The chord force associated with the expected flexural strength of the column, determined as $1.1 R_y M_p / \alpha_s$

(c) The chord force associated with the moment based on the expected shear strength of the column between the top and bottom chord of the connected truss

OMF truss-to-column connections shall be designed to transfer the vertical shear force generated from end moments consistent with these chord forces.

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) CJP 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:

$$E_{cl} = 2\left(1.1R_y M_p\right)/L_h \qquad \text{(E2-1)}$$

where

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

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

where

ΣM^*_{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 ΣM^*_{pc} as follows:

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

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

ΣM^*_{be} = 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 ΣM^*_{be} as follows:

$$\Sigma M^*_{be} = \Sigma\left(M_{pr} + \alpha_s M_v\right) \tag{E3-3}$$

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 $\alpha_s P_{rc} < 0.3P_{yc}$ 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 available axial yield strength of column, P_{yc}, shall be determined as follows:

$$P_{yc} = F_{yc} A_g \tag{E3-4}$$

and the required axial strength is P_{rc} using LRFD or ASD load combinations 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 beam flange, in. (mm)

t_{bf} = thickness of beam flange, in. (mm)

2. Unbraced Connections

Columns that do not have bracing transverse to the seismic frame at the beam-to-column connection 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 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, in accordance with 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) CJP 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 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:

$$E_{cl} = 2M_{pr}/L_h \tag{E3-5}$$

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

M_{pr} = maximum probable moment at the location of the plastic hinge, as defined in Section E3.4a, kip-in. (N-mm)

When E_{cl} as defined in Equation E3-5 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 satisfy the following requirement:

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

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 E3-6. Additionally, the individual thicknesses of the column web and doubler plate shall satisfy Equation E3-6, 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 E3-6.

3. Panel-Zone Doubler Plates

The thickness of doubler plates, if used, shall not be less than 1/4 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 E3-6, 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 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 CJP groove welds, PJP groove welds, or fillet welds. The required strength of PJP 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 E3-6, 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 E3-6, 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:

$$t_{lim} = \frac{b_{bf}}{6} \tag{E3-7}$$

where

b_{bf} = width of beam flange, in. (mm)

(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:

$$t_{lim} = \frac{b_{bf}}{12} \tag{E3-8}$$

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 according to Sections E2.6f or E3.6f.1, or where they are listed as an alternative in Section E1.6b(c)(4), 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.

(3) The continuity plate width-to-thickness ratio shall be limited by

$$b/t \le 0.56\sqrt{\frac{E}{R_y F_y}} \tag{E3-9}$$

(c) Continuity-Plate Welds

Continuity plates shall be welded to column flanges using CJP groove welds or fillet welds on each side of the continuity plate with weld size of each fillet weld equal to at least 75% of the thickness of the continuity plate.

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 2-1/2 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 2-1/2 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 $\Sigma M_{pc} / (\alpha_s H_c)$, where ΣM_{pc} is the sum of the plastic moments 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 the design earthquake displacement 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 $0.03F_y A_g / \alpha_s$.

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:

$$P_r = 0.06 R_y F_y A_f / \alpha_s \qquad \text{(E4-1)}$$

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:

$$P_r = 0.02 R_y P_{nc} / \alpha_s \qquad \text{(E4-2)}$$

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 of chord members, $\phi_t P_n$ (LRFD) and P_n/Ω_t (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

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

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

5c. Expected Vertical Shear Strength of Special Segment

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

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

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 $R_y F_y A_g / \alpha_s$.

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 yield strength, P_{yc}, 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

No demand critical welds are required for this system.

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 axial strength, P_{rc}, shall not exceed 15% of the available axial yield strength, P_{yc}, for these load combinations only.

4b. Stability Bracing of Columns

Bracing required in this section shall restrain lateral-torsional buckling of the cantilever column to develop flexural yielding at the column base. When columns are bent about their major axis, bracing shall be provided at the top of the column and at intermediate locations if necessary, satisfying the following requirements:

(a) Both flanges of the column shall be laterally braced for lateral-torsional buckling or the column cross section shall be braced for lateral-torsional buckling with point torsional bracing.

(b) Bracing shall meet the requirements of *Specification* Appendix 6 for lateral or point torsional bracing of beams, where C_d is 1.0 and the required flexural strength of the member shall be determined in accordance with Equation D1-1 in Section D1.2a.1(b).

(c) For doubly symmetric I-shaped members, the bracing shall have a maximum spacing of:

$$L_{bc} = \left[0.12 - 0.076\left(M_1'/M_2\right)\right]\frac{r_y E}{R_y F_y} \qquad \text{(E6-1)}$$

where

L_{bc} = length between base and bracing point or between bracing points of a cantilever column where the bracing points are either braced against lateral displacement for both flanges or braced against twist of the cross section, in. (mm)

M_1' = effective moment at the end of the unbraced length opposite from M_2 as determined from *Specification* Appendix 1, kip-in. (N-mm)

M_2 = larger moment at end of unbraced length, kip-in. (N/mm) (shall be taken as positive in all cases)

(d) For rectangular HSS and box sections, the bracing shall have a maximum spacing of:

$$L_{bc} = \left[0.17 - 0.10\left(M_1'/M_2\right)\right]\frac{r_y E}{R_y F_y} \geq 0.10\,\frac{r_y E}{R_y F_y} \qquad \text{(E6-2)}$$

Exceptions:

(a) Bracing may be omitted for square or round HSS and for square box sections.

(b) Bracing may be omitted for any column section acting as a cantilever only about its minor axis.

(c) Bracing may be omitted for cantilever columns bent about their major axis when the column cantilever length from the base to the top does not exceed half the maximum spacing calculated in accordance with Equation E6-1 or E6-2 as applicable.

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

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

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 overstrength 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- and inverted V-braced frames 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 taken as 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 taken as a maximum of $0.3P_n$

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-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 ordinary concentrically 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 horizontal seismic load effect including overstrength, E_{mh}, 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 horizontal seismic load effect including overstrength, E_{mh}, 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 horizontal seismic load effect including overstrength, E_{mh}, 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, geometric 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 resulting in compression in the column and contributed by the compression or tension 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 $R_yF_yA_g$,

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

$$\frac{L_c}{r} \le 4\sqrt{E/F_y} \tag{F1-1}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
L_c = effective length of brace = KL, in. (mm)
K = effective length factor
L = length of brace, in. (mm)
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 $1.1 F_{ne} A_g / \alpha_s$, where F_{ne} is the nominal stress calculated from *Specification* Chapter E using expected yield stress, $R_y F_y$, in lieu of F_y. The brace length used for the determination of F_{ne} 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.

F2. SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF)

1. Scope

Special concentrically braced frames (SCBF) 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 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.

For the purpose of designating a brace as acting in tension or in compression, in order to establish the expected brace strength the horizontal component of the design earthquake loads shall be applied in one direction per analysis. Analyses shall be performed for each direction of frame loading. For systems that include columns that form part of two intersecting frames in orthogonal or multi-axial directions, the analysis shall consider the potential for brace yielding in both directions simultaneously.

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 $(1/0.877)F_{ne}A_g$, where F_{ne} is the nominal stress calculated from *Specification* Chapter E using expected yield stress, R_yF_y, in lieu of F_y. The brace length used for the determination of F_{ne} 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 $0.3F_n$ 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.

(d) To compute the required strength of beams in V- and inverted V-braced frames, the expected brace strength in tension need not exceed the magnitude of the expected brace strength in compression.

User Note: When computing F_{ne} for analyses in this section, including Exception (d), the brace length is defined as the distance from brace end to brace end. This length depends upon the final brace-to-gusset connection configuration and iteration may be required.

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. 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.

Exception: Lines of bracing may be exempted from the lateral-force distribution requirement for buildings meeting the following requirements:

1. The required strength of each brace in compression along the exempted line is the overstrength seismic load.
2. Removal of noncompliant lines of bracing, singly or in combination, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity in accordance with ASCE/SEI 7.

User Note: Compliance with Exception 2 may be performed similar to the ASCE/SEI 7 redundancy determination in accordance with Table 12.3-3, with all braces on the noncompliant line(s) removed. In some cases, the removal of one noncompliant line may be more severe for torsion than the removal of two.

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- or inverted V-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-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 special concentrically 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 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, geometric 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 resulting in compression in the column and contributed by the compression or tension 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

(c) For beams of V- and inverted V-braced frames designed using Exception (d) of Section F2.3, a zone adjacent to each gusset plate edge equal to the beam 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

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 satisfy one of the following requirements:

(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 SCBF 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* Equations 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_yM_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 earthquake displacement. 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/A572M Grade 50 plate and $0.62t_p$ for ASTM A36/A36M 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 (CJP) groove welds.

Column splices shall be designed to develop at least 50% of the lesser plastic moment of the connected members, M_p, divided by α_s.

The required shear strength shall be $\left(\Sigma M_p/\alpha_s\right)/H_c$,

where

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

ΣM_p = sum of the plastic moments, 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 capacity-limited horizontal seismic load effect, 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) The forces corresponding to the resistance of the foundation to overturning uplift.

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

Analyses shall be performed for each direction of frame loading. For systems that include columns that form part of two intersecting frames in orthogonal or multi-axial directions, the analysis shall consider the potential for link yielding in both directions simultaneously.

The inelastic link rotation angle shall be determined from the inelastic portion of the design earthquake displacement. 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 at the design earthquake displacement, δ_{DE}. 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 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 by Section D1.2c for expected plastic hinge locations.

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 $e \leq 1.6M_p/V_p$ are permitted to satisfy the requirements for moderately ductile members, where e is the length of link, defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face. Webs of links with box sections with link lengths, $e \leq 1.6M_p/V_p$, 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, CJP 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

$$V_p = 0.6F_y A_{lw} \text{ for } \alpha_s P_r/P_y \leq 0.15 \tag{F3-2}$$

$$V_p = 0.6F_y A_{lw}\sqrt{1-\left(\alpha_s P_r/P_y\right)^2} \text{ for } \alpha_s P_r/P_y > 0.15 \tag{F3-3}$$

A_{lw} = web area of link (excluding flanges), in.^{2} (mm^2)

$= \left(d - 2t_f\right)t_w$ for I-shaped link sections (F3-4)

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

P_r = required axial strength using LRFD or ASD load combinations, kips (N)
P_y = axial yield strength = F_yA_g (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 for $\alpha_s P_r/P_y \le 0.15$ (F3-8)

$$M_p = F_yZ\left(\frac{1-\alpha_s P_r/P_y}{0.85}\right) \text{ for } \alpha_s P_r/P_y > 0.15 \tag{F3-9}$$

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 $\alpha_s P_r/P_y > 0.15$, 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 $\rho' > 0.5$

$$e \le \frac{1.6M_p}{V_p}(1.15 - 0.3\rho') \tag{F3-11}$$

where

$$\rho' = \frac{P_r/P_y}{V_r/V_y} \tag{F3-12}$$

V_r = required shear strength using LRFD or ASD load combinations, kips (N)
V_y = shear yield strength, kips (N)
$= 0.6F_yA_{lw}$ (F3-13)

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

4. Link Stiffeners for I-Shaped Cross Sections

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $\left(b_f - 2t_w\right)$ and a thickness not less than the larger of $0.75t_w$ or 3/8 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 3/8 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 $F_y A_{st}/\alpha_s$, 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 $F_y A_{st}/(4\alpha_s)$.

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 1/2 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 $0.67\sqrt{\dfrac{E}{R_y F_y}}$, full-depth web stiffeners shall be provided on one side of each link web, spaced at intervals not exceeding $20t_w - (d - 2t_f)/8$.

(b) For links of length $1.6M_p/V_p$ or less and with web depth-to-thickness ratio, h/t_w, less than $0.67\sqrt{\dfrac{E}{R_y F_y}}$, no intermediate web stiffeners are required.

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

Intermediate web stiffeners shall be full depth and 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 $F_y A_{st}/\alpha_s$, 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 satisfy one of the following requirements:

(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, $R_y M_p$, multiplied by 1.1 and divided by α_s,

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

(2) A moment corresponding to the sum of the expected column flexural strengths, $\Sigma\left(R_y F_y Z\right)$, 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 CJP groove welds. Column splices shall be designed to develop at least 50% of the lesser plastic moment of the connected members, M_p, divided by α_s.

The required shear strength shall be $\Sigma M_p / (\alpha_s H_c)$,

where

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

ΣM_p = sum of the plastic moments, $F_y Z$, 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 earthquake displacement, 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 F4.5b 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 F4.5b 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 as 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.

For the purpose of designating a brace as acting in tension or in compression, in order to establish the expected brace strength the horizontal component of the design earthquake loads shall be applied in one direction per analysis. Analyses shall be performed for each direction of frame loading. For systems that include columns that form part of two intersecting frames in orthogonal or multi-axial directions, the analysis shall consider the potential for brace yielding in both directions simultaneously.

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 using the capacity-limited seismic load effect. 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- and inverted V-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 seismic load 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- or inverted V-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 strength and stiffness for column point bracing as prescribed in the *Specification*. P_r may be taken as the adjusted brace strength in compression of the BRBF brace.

4b. K-Braced Frames

K-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 buckling-restrained braced frame (MT-BRBF) when the following requirements are satisfied.

(a) Struts shall be provided at every brace-to-column connection location.

(b) 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 buckling-restrained brace (BRB) using the desired brace capacity, P_{ysc}, rather than a desired core area is recommended for the MT-BRBF 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 larger of the frame shear strengths of adjacent tiers as determined using adjusted brace strengths. 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.

(c) 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.

4e. Overall Stability of BRB and Connection Assemblies

The design of a buckling-restrained brace (BRB) and its connections shall include consideration of combined buckling modes that include imperfections and flexibility of the gusset plate, casing, and other elements that significantly affect stability. At a minimum, the following shall be considered:

(a) Initial imperfections in the brace and gusset plates

(b) Flexibility of the core extension

(c) Flexibility of the BRB core and casing interconnection

(d) Casing flexibility

User Note: Global stability of the BRB and connections can be demonstrated through calculations or through testing that is representative of the project connection.

5. Members

5a. Basic Requirements

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

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:

$$P_{ysc} = F_{ysc} A_{sc} \qquad \text{(F4-1)}$$

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

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)

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. If the prototype has a lower axial yield strength, P_{ysc}, than all qualifying tests that meet the requirements of Section K3, the similarity limits specified in Section K3.3c(b) shall be restricted such that the axial yield strength of the steel core of the test specimen is no more than 120% of the prototype. 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. In addition, the rational analysis shall address 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 satisfy one of the following requirements:

(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 moment, kip-in. (N-mm)

(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,

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 BRBF 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 of gusset plates 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 CJP groove welds. Column splices shall be designed to develop at least 50% of the lesser plastic moment of the connected members, M_p, divided by α_s.

The required shear strength, V_r, shall be determined as follows:

$$V_r = \frac{\Sigma M_p}{\alpha_s H_c} \tag{F4-2}$$

where

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

ΣM_p = sum of the plastic 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 the 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 resist flexural forces at each end corresponding to moments equal to $1.1 R_y M_p / \alpha_s$,

where

F_y = specified minimum yield stress, ksi (MPa)

M_p = plastic 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

Analyses shall be performed for each direction of frame loading. For systems that include columns that form part of two intersecting frames in orthogonal or multi-axial directions, the analysis shall consider the potential for web yielding in both directions simultaneously.

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 in accordance with Equation E1-1 are 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 enough that member design is governed by shear yielding.

4. System Requirements

4a. Stiffness of Boundary Elements

The stiffness of VBE and HBE shall be such that the entire web plate is yielded at the design earthquake displacement. 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 $0.0031 t_w h^4 / L$. The HBE shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_b, not less than $0.0031 L^4 / h$ times the difference in web plate thicknesses above and below,

where

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-to-VBE connections without including the effects of SPSW web plates.

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, 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, $\phi_v V_n$ (LRFD), and the allowable shear strength, V_n / Ω_v

(ASD), in accordance with the limit state of shear yielding, shall be determined as follows:

$$V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha \tag{F5-1}$$

$$\phi_v = 0.90 \text{ (LRFD)} \qquad \Omega_v = 1.67 \text{ (ASD)}$$

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, may be taken as 45°.

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 CJP groove welds. Column splices shall be designed to develop at least 50% of the lesser plastic 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 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, $\phi_v V_n$ (LRFD), and the allowable shear strength, V_n/Ω_v (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:

$$V_n = 0.42 F_y t_w L_{cf}\left(1 - \frac{0.7D}{S_{diag}}\right) \tag{F5-2}$$

$$\phi_v = 0.90 \text{ (LRFD)} \qquad \Omega_v = 1.67 \text{ (ASD)}$$

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 the regularly perforated plate shall be calculated using an effective web-plate thickness, t_{eff}, given by:

$$t_{eff} = \frac{1-\dfrac{\pi}{4}\left(\dfrac{D}{S_{diag}}\right)}{1-\dfrac{\pi}{4}\left(\dfrac{D}{S_{diag}}\right)\left(1-\dfrac{N_r D \sin\alpha}{H_c}\right)}t \qquad \text{(F5-3)}$$

where

H_c = clear column (and web-plate) height between beam flanges, in. (mm)
N_r = number of horizontal rows of perforations
t = thickness of web plate, 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 imposes the development of web yielding in a direction parallel to that of the hole alignment. Therefore, α is equal to 45° for the case addressed by Section F5.7a.

4. Effective Expected Tension Stress

The effective expected tension for analysis is $R_y F_y \left(1-0.7D/S_{diag}\right)$.

7b. Reinforced Corner Cut-Out

Quarter-circular cut-outs are permitted at the corners of the webs provided that the webs are connected to arching plates that align with the edge of the cut-outs and serve to reinforce the web along 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 earthquake displacement.

1. Design for Tension

The required axial strength of the arching plate in tension, P_r, resulting from web-plate tension in the absence of other forces, shall be taken as:

$$P_r = \frac{R_y F_y t_w r^2 / \alpha_s}{4e} \qquad \text{(F5-4)}$$

where

F_y = specified minimum yield stress of the web plate, in.[2] (mm[2])
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) (F5-5)
r = radius of the cut-out, in. (mm)

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 required strength of the arching plate under the combined effects of axial compression force, P_r, and moment, M_r, in the plane of the web resulting from connection deformation in the absence of other forces shall be taken as:

$$P_r = \frac{15EI_y}{\alpha_s\left(16e^2\right)}\left(\frac{\Delta_{DE}}{H}\right) \tag{F5-6}$$

$$M_r = P_r e \tag{F5-7}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

H = height of story, in. (mm)

I_y = moment of inertia of the plate about the *y*-axis, in.^{4} (mm^4)

Δ_{DE} = frame drift corresponding to the design earthquake displacement, in. (mm)

HBE and VBE shall be designed to resist the combined axial and flexural required strengths acting at the end of the arching plate.

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, A3.1, 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 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:

$$E_{cl} = 2\left(1.1M_{pbe}\right)/L_h \qquad \text{(G2-1)}$$

where

M_{pbe} = 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_{pbe} shall be calculated using the plastic stress distribution or the strain compatibility method as described in *Specification* Section I1.2a or I1.2b, respectively. 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_{pbe} 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 moment, M_{pcc}, of the smaller composite column. The required shear strength of column web splices shall be at least equal to $\Sigma M_{pcc}/H$,

where

H = height of story, in. (mm)

ΣM_{pcc} = sum of the plastic moments 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:

$$\frac{\Sigma M^*_{pcc}}{\Sigma M^*_{pbe}} > 1.0 \tag{G3-1}$$

where

ΣM^*_{pcc} = sum of the projections of the plastic moments, 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 moment, M_{pcc}, shall satisfy the requirements of *Specification* Chapter I including the required axial strength, P_{rc}. For reinforced concrete columns, the plastic moment, 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.

ΣM^*_{pbe} = 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 $\Sigma M^*_{pbe} = \Sigma(1.1M_{pbe}+M_{uv})$, where M_{pbe} 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 $P_{rc} < 0.1P_{yc}$.

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 transverse reinforcement in regions where plastic hinges are expected to occur under seismic deformations. Transverse 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.

$$Y_{PNA} \le \frac{Y_{con} + d}{1 + \left(\dfrac{1{,}700\, F_y}{E}\right)} \qquad \text{(G3-2)}$$

where

Y_{PNA} = distance from the extreme concrete compression fiber to the plastic neutral axis, in. (mm)

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) CJP 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:

$$E_{cl} = 2\left(1.1M_{pbe}\right)/L_h \quad \text{(G3-3)}$$

where

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

M_{pbe} = expected flexural strength of the steel, concrete-encased, or composite beams, kip-in. (N-mm). For concrete-encased or composite beams, M_{pbe} shall be calculated according to Section G2.6d

6e. Connection Diaphragm Plates

The diaphragm plates used in connections to filled composite columns 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:

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), where at least one of the elements (columns, beams, or braces) is a composite or reinforced concrete member, 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.

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, A3.1, 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.

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.

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 satisfy one of the following requirements:

(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_{pbe}, where M_{pbe} 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 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 moment, M_{pcc}, of the smaller composite column. The required shear strength of column web splices shall be at least equal to $\Sigma M_{pcc}/H$, where ΣM_{pcc} is the sum of the plastic moments 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 sections. Braces shall be structural steel sections or filled composite members.

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 satisfy one of the following requirements:

(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 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. Composite coupling beam effective stiffness shall be taken as the following:

(1) For flexure

$$(EI)_{eff} = 0.07\left(\frac{g}{h}\right)(EI)_{trans} \tag{H4-1}$$

(2) For axial strength and deformation

$$(EA)_{eff} = 1.0(EA)_{trans} \tag{H4-2}$$

(3) For shear

$$(GA)_{eff} = 1.0GA_w \tag{H4-3}$$

where

A_w = area of steel beam web, in.^{2} (mm^2)
$(EI)_{trans}$ = flexural rigidity of the cracked transformed section
$(EA)_{trans}$ = axial rigidity of the transformed section
G = shear modulus of steel, ksi (MPa)
g = clear span of coupling beam, in. (mm)

h = overall depth of composite section, in. (mm)

(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

There are no requirements specific to this system.

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. Wide-flange steel coupling beams shall satisfy the following requirements.

(a) Structural steel coupling beams shall be designed in accordance with Chapters F and G of the *Specification*.

(b) The design connection shear strength, $\phi_v V_{n,connection}$, shall be computed from Equations H4-4 and H4-4M, with $\phi_v = 0.90$. The embedment length provided shall not be less than d, irrespective of the calculated value of L_e.

$$V_{n,connection} = 1.54\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left(\frac{0.58-0.22\beta_1}{0.88+\dfrac{g}{2L_e}}\right) \quad \text{(H4-4)}$$

$$V_{n,connection} = 4.04\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left(\frac{0.58-0.22\beta_1}{0.88+\dfrac{g}{2L_e}}\right) \quad \text{(H4-4M)}$$

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

(c) M_u shall be multiplied by $1+[(2L_e)/(3g)]$

where
L_e = minimum embedment length of coupling beam measured from the face of the wall that provides sufficient connection shear strength based on Equation H4-4 or H4-4M, in. (mm)

(d) Wall longitudinal reinforcement with nominal axial strength equal to the required shear strength of the coupling beam multiplied by

$$\frac{\dfrac{g}{2L_e}+0.33\beta_1}{0.88-0.33\beta_1}\geq 1.0 \qquad \text{(H4-5)}$$

shall be placed over the embedment length of the beam. This wall longitudinal reinforcement shall extend a distance of at least one tension development length above and below the flanges of the structural steel coupling beam. It is permitted to use longitudinal reinforcement placed for other purposes, such as for vertical boundary members, as part of the required longitudinal reinforcement.

2. Composite Coupling Beams

Encased composite sections serving as coupling beams shall satisfy the following requirements:

(a) Structural steel 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 H4-4 or H4-4M. The embedment length provided shall not be less than d, irrespective of the calculated value of L_e.

(b) The design shear strength of the composite beam, $\phi_v V_{nc}$, is computed from Equation H4-6 and H4-6M, with $\phi_v = 0.90$.

$$V_{nc}=V_p+\left(0.0632\sqrt{f_c'}b_{wc}d_c+\frac{A_{sr}F_{ysr}d_c}{s}\right) \qquad \text{(H4-6)}$$

$$V_{nc}=V_p+\left(0.166\sqrt{f_c'}b_{wc}d_c+\frac{A_{sr}F_{ysr}d_c}{s}\right) \qquad \text{(H4-6M)}$$

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)

(c) Longitudinal and transverse coupling beam reinforcement shall be distributed around the beam perimeter with total area in each direction of at least $0.002b_{wcs}$ and spacing not exceeding 12 in. (300 mm). Longitudinal reinforcement shall not extend into the wall and shall not be included in the computation of flexural strength.

(d) The requirements of Sections H4.5b.1(c) and H4.5b.1(d) shall be satisfied.

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 to 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-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 earthquake displacement 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 earthquake displacement 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 $g \le 1.6 M_p / V_p$ 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 $g \ge 2.6 M_p / V_p$ 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, for wall piers.

(b) Effects of shear distortion of the steel coupling beam shall be taken into account.

4. System Requirements

For walls that are not part of coupled wall systems, i.e., isolated walls, there are no specific system-level 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. In addition, the following requirements shall be satisfied:

(a) In coupled walls, coupling beams shall be designed to 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).

Longitudinal wall reinforcement as specified in Section H4.5b.1(d) shall be confined by transverse reinforcement that meets the requirements for boundary members of ACI 318 Section 18.10.6. Unless vertical transfer reinforcement is provided in accordance with Section H5.5c(e), transverse reinforcement satisfying ACI 318 Section 18.7.5.2(a) through (e) over the distance calculated in accordance with ACI 318 Section 18.10.6.4(a) shall be provided between a height of L_e below the bottom flange and L_e above the top flange of an embedded steel section with a vertical spacing not exceeding the lesser of 8 in. (203 mm) and eight times the diameter of the smallest longitudinal reinforcement confined by this transverse reinforcement.

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.

$$V_{be} = 1.54\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left(\frac{0.58-0.22\beta_1}{0.88+\dfrac{g}{2L_e}}\right) \tag{H5-1}$$

$$V_{be} = 4.04\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66}\beta_1 b_f L_e\left(\frac{0.58-0.22\beta_1}{0.88+\dfrac{g}{2L_e}}\right) \tag{H5-1M}$$

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)

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

V_{be} = expected shear strength of a steel coupling beam computed from Equation H5-2, kips (N)

$$= \frac{2(1.1R_y)M_p}{g} \le (1.1R_y)V_p \tag{H5-2}$$

where

A_{tw} = area of structural steel beam web, in.^{2} (mm^2)

F_y = specified minimum yield stress, ksi (MPa)

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

V_p = $0.6F_yA_{tw}$, 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, PJP, or CJP 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 earthquake displacement. 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) The requirements of Section H4.5b.1(c) shall be satisfied, except that the minimum value of L_e shall be determined based on the calculations for shear strength given by Equation H5-1 or H5-1M rather than Equation H4-4 or H4-4M.

(e) The requirements of Section H4.5b.1(d) shall be satisfied with V_{be} used instead of the required shear strength. The area of vertical reinforcement used to satisfy Section H4.5b.1(d) may include 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 reinforcement 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 shall meet the development length requirements of ACI 318, Chapter 18, 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:

$$A_{tb} \geq 0.03 f_c' L_e b_f \big/ F_{ystr} \qquad \text{(H5-3)}$$

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_{ystr} = 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:

$$\Sigma A_{tb} < 0.08 L_e b_w - A_{sr} \qquad \text{(H5-4)}$$

where

ΣA_{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 for the following: the requirements of Section F3.5b.4 need not be met; the use of face bearing plates specified in Section H5.5c(b) need not be provided; and V_{ce} shall replace $(1.1R_y)V_p$ and $1.1M_{pbe}$ shall replace $(1.1R_y)M_p$ in Equation H5-2. M_{pbe} shall be computed using Section G2.6d. The limiting expected shear strength, V_{ce}, is:

$$V_{ce} = 1.1 R_y V_p + 0.08\sqrt{R_c f_c'}\, b_{wc} d_c + \frac{1.33 R_{yr} A_{sr} F_{ysr} d_c}{s} \qquad \text{(H5-5)}$$

$$V_{ce} = 1.1 R_y V_p + 0.21\sqrt{R_c f_c'}\, b_{wc} d_c + \frac{1.33 R_{yr} A_{sr} F_{ysr} d_c}{s} \qquad \text{(H5-5M)}$$

where

A_{sr} = area of transverse reinforcement within s, in.^{2} (mm^2)

F_{ysr} = specified minimum yield stress of transverse reinforcement, ksi (MPa)

R_c = factor to account for expected strength of concrete = 1.3

R_{yr} = ratio of the expected yield stress of the transverse reinforcement material to the specified minimum yield stress, to be taken as the R_y value from Table A3.1 for the corresponding steel reinforcement material, 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. This section is applicable to C-PSW/CE consisting of steel plates with reinforced concrete encasement on one or both sides of the plate and structural steel sections 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 earthquake displacement,

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 3/8 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, $\phi_v V_n$, for the limit state of shear yielding with a composite plate conforming to Section H6.5c, shall be:

$$V_n = 0.6 A_{sp} F_y \tag{H6-1}$$

$$\phi_v = 0.90 \text{ (LRFD)}$$

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 design 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. Longitudinal and transverse 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 sections 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 earthquake displacement. Composite and reinforced concrete boundary members shall also satisfy the requirements of Section H5.5b. Structural 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 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 moment, M_{pcc}, of the smaller composite column. The required shear strength of column web splices shall be at least equal to $\Sigma M_{pcc}/H$, where ΣM_{pcc} is the sum of the plastic moments 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 C-PSW/CF consisting of planar, C-shaped, or I-shaped walls, where each wall element consists of two planar steel plates with concrete infill between them. Composite action between the plates and concrete infill is achieved using either tie bars or combination of tie bars and steel headed stud anchors.

In each wall element, the two steel plates shall be of equal nominal thickness and connected using tie bars. The steel plates shall comprise at least 1%, but no more than 10% of the gross wall area.

Boundary elements or flange or closure plates shall be used at the open ends of the wall elements. The boundary elements shall be either: (a) half-circular section of diameter equal to the distance between the two web plates, or (b) circular filled composite members.

The height-to-length ratio, h_w/L_w, of the composite walls shall be greater than or equal to 3.

2. Basis of Design

C-PSW/CF, 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 steel plates 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 plates shall not be the governing mechanism.

3. Analysis

The effective stiffness of composite walls shall be calculated in accordance with *Specification* Section I1.5.

4. System Requirements for C-PSW/CF with Half-Circular or Circular Boundary Elements

4a. Steel Web Plate of C-PSW/CF with Half-Circular or Circular Boundary Elements

The maximum spacing of tie bars in vertical and horizontal directions, w_1, shall be:

$$w_1 = 1.8t\sqrt{\frac{E}{F_y}} \quad \text{(H7-1)}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

F_y = specified minimum yield stress, ksi (MPa)
t = thickness of plate, in. (mm)

When tie bars are welded to the steel plates, the thickness of the plate shall develop the tension strength of the tie bars.

4b. Half Circular or Circular Boundary Elements

The diameter-to-thickness ratio, D/t, for the circular part of the C-PSW/CF cross section shall conform to:

$$\frac{D}{t} \leq 0.044\frac{E}{F_y} \qquad \text{(H7-2)}$$

where
D = outside diameter of round HSS, in. (mm)
t = thickness of HSS, in. (mm)

4c. Tie Bars

Tie bars shall be designed to resist the tension force, T_{req}, while remaining elastic for all applicable load combinations, determined as follows:

$$T_{req} = T_1 + T_2 \qquad \text{(H7-3)}$$

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:

$$T_1 = 2\left(\frac{w_2}{w_1}\right)t_s^2 F_y \qquad \text{(H7-4)}$$

where
t = thickness of plate, 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.

$$T_2 = \left(\frac{tF_{y,plate}t_{sc}}{4}\right)\left(\frac{w_2}{w_1}\right)\left[\frac{6}{18\left(\frac{t_{sc}}{w_{min}}\right)^2+1}\right] \qquad \text{(H7-5)}$$

where
t_{sc} = total thickness of composite plate shear wall, in. (mm)
w_{min} = minimum of w_1 and w_2, in. (mm)

5. System Requirements for C-PSW/CF with Flange or Closure Plates

5a. Steel Web and Flange Plates

In regions of flexural yielding (at the base), the steel plate slenderness ratio, b/t, shall be limited as follows:

$$\frac{b}{t} \leq 1.05\sqrt{\frac{E}{R_y F_y}} \qquad \text{(H7-6)}$$

where
E = modulus of elasticity of plate = 29,000 ksi (200 000 MPa)

b = largest unsupported length of plate between rows of steel anchors or ties, in. (mm)
t = thickness of plate, in. (mm)

5b. Tie Bars

The maximum spacing and diameter of tie bars shall be limited as follows:

$$w_1 \leq 1.0\, t \sqrt{\frac{E}{2\alpha + 1}} \tag{H7-7}$$

$$\alpha = 1.7 \left[\frac{t_{sc}}{t} - 2 \right] \left[\frac{t}{d_{tie}} \right]^4 \tag{H7-8}$$

where
d_{tie} = diameter of tie bar, in. (mm)
t = thickness of plate, in. (mm)
t_{sc} = total thickness of composite plate shear wall, in. (mm)
w_1 = largest clear spacing of ties in vertical and horizontal directions, in. (mm)

6. Members

6a. Flexural Strength

The available plastic moment strength of the C-PSW/CF shall be determined in accordance with *Specification* Section I1.2a.

6b. Shear Strength

The available shear strength of C-PSW/CF shall be determined in accordance with *Specification* Section I4.4.

7. Connection Requirements

7a. 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.

7b. Connection between C-PSW/CF Steel Components

Welds between the steel web plates and the boundary elements or flange or closure plates shall be CJP groove welds.

7c. C-PSW/CF and Foundation Connection

Where the composite walls are connected directly to the foundation at the point of maximum moment in the walls, the composite wall-to-foundation connections 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. The plastic flexural composite strength of the wall shall be obtained by the plastic stress distribution method described in *Specification* Section I1.2a. 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.

8. Protected Zones

The regions subjected to inelastic straining at the base of the composite walls shall be designated as protected zones.

8a. Splices

Steel plate and boundary element splices located in the designated protected zones shall develop the full strength of the weaker of the two connected elements.

9. Demand Critical Welds in Connections

Where located within the protected zones, the following welds shall be demand critical and shall satisfy the applicable requirements:

(a) Welds connecting the composite wall web plates to the boundary elements or to the flange or closure plates

(b) Welds in the composite wall steel plate splices

(c) Welds at composite wall steel plate-to-base plate connections

H8. COUPLED COMPOSITE PLATE SHEAR WALLS—CONCRETE FILLED (CC-PSW/CF)

1. Scope

Coupled composite plate shear walls—concrete filled (CC-PSW/CF) shall be designed in accordance with this section. This section is applicable to CC-PSW/CF consisting of (i) concrete-filled composite plate shear walls, and (ii) filled composite coupling beams.

The composite plate shear walls of CC-PSW/CF consist of planar, C-shaped, I-shaped, or L-shaped walls, where each wall element consists of two planar steel plates with concrete infill between them. Composite action between the plates and concrete infill is achieved using either tie bars or a combination of tie bars and steel headed stud anchors. In each wall element, the two steel plates shall be of equal nominal thickness and connected using tie bars. A flange or closure plate shall be used at the open ends of the wall elements. No additional boundary elements besides the closure plate are required to be used with the composite walls. The wall height-to-length ratio, h_w/L_w, of the composite walls shall be greater than or equal to 4.

Coupling beams shall consist of concrete-filled built-up box sections of uniform cross-section along their entire length, and with a width equal to or greater than the wall thickness at the connection. For at least 90% of the stories of the building, the clear length-to-section depth ratios, L/d, of the coupling beams shall be greater than or equal to 3 and less than or equal to 5.

2. Basis of Design

CC-PSW/CF designed in accordance with these provisions shall provide significant inelastic deformation capacity through flexural plastic hinging in the composite coupling beams, and through flexural yielding at the base of the composite wall elements.

3. Analysis

3a. Stiffness

The effective flexural, axial, and shear stiffness of composite walls and filled composite coupling beams shall be calculated in accordance with *Specification* Section I1.5.

3b. Required Strength for Coupling Beams

The required strengths for the coupling beams shall be determined based on analyses in conformance with the applicable building code.

3c. Required Strengths for Composite Walls

The required strengths for the composite walls shall be determined using the capacity-limited seismic load The capacity-limited seismic load refers to the capacity-limited horizontal seismic load effect, E_{cl}, which shall be determined from an analysis in which all the coupling beams are assumed to develop plastic hinges at both ends with expected flexural capacity of $1.2M_{p,exp}$, and the maximum overturning moment is amplified to account for the increase in lateral loading from the formation of the earliest plastic hinges to the formation of plastic hinges in all coupling beams. The earthquake-induced axial force in the walls for determining the required wall strength shall be calculated as the sum of the capacity-limited coupling beam shear forces, using Equation H8-7, along the height of the structure. The portion of the maximum overturning moment resisted by coupling action shall be calculated as the couple caused by the wall axial forces associated with the coupling beam strengths. The remaining portion of the earthquake-induced overturning moment shall be distributed to the composite walls in accordance with their flexural stiffness, while accounting for the effects of simultaneous axial force. The required axial and flexural strengths for the composite walls shall be determined directly from this analysis, while the required wall shear strengths determined from this analysis shall be amplified by a factor of 4.

4. Composite Wall Requirements

The composite wall shall be designed in accordance with the requirements of this section.

4a. Area of Steel Requirements

The steel plates shall comprise at least 1%, but no more than 10%, of the total composite cross-sectional area.

4b. Steel Plate Slenderness Requirement

In regions of flexural yielding (at the base), the steel plate slenderness ratio, b/t, shall be limited as follows.

$$\frac{b}{t} \leq 1.05\sqrt{\frac{E_s}{R_y F_y}} \tag{H8-1}$$

where

b = largest unsupported length of the plate between horizontal and vertical rows of steel anchors or tie bars, in. (mm)

t = thickness of plate, in. (mm)

4c. Tie Bar Spacing Requirement

The tie bar spacing-to-plate thickness ratio, s_t/t, shall be limited as follows:

$$\frac{s_t}{t} \leq 1.0\sqrt{\frac{E_s}{2\alpha+1}} \tag{H8-2}$$

$$\frac{s_t}{t} \leq 0.38\sqrt{\frac{E_s}{2\alpha+1}} \tag{H8-2M}$$

$$\alpha = 1.7\left[\frac{t_{sc}}{t} - 2\right]\left[\frac{t}{d_{tie}}\right]^4 \tag{H8-3}$$

where

d_{tie} = diameter of tie bar, in. (mm)

s_t = largest center-to-center spacing of the tie bars, in. (mm)

t_{sc} = total thickness of composite plate shear wall, in. (mm)

4d. Tie Bar-to-Plate Connection

The tie bar-to-steel plate connection shall develop the full yield strength of the tie bar.

5. Composite Coupling Beam Requirements

The composite coupling beam shall be designed in accordance with the requirements of this section.

5a. Minimum Area of Steel

The cross-sectional area of the steel section shall comprise at least 1% of the total composite cross-section of the coupling beam.

5b. Slenderness Requirements for Coupling Beams

The slenderness ratios of the flanges and webs of the filled composite coupling beam, b_c/t_f and h_c/t_w, shall be limited as follows:

$$\frac{b_c}{t_f} \le 2.37\sqrt{\frac{E_s}{R_y F_y}} \tag{H8-4}$$

$$\frac{h_c}{t_w} \le 2.66\sqrt{\frac{E_s}{R_y F_y}} \tag{H8-5}$$

where

b_c = clear width of the coupling beam flange plate, in. (mm)
h_c = clear depth of the coupling beam web plate, in. (mm)
t_f = thickness of the coupling beam flange plate, in. (mm)
t_w = thickness of the coupling beam web plate, in. (mm)

5c. Flexure-Critical Coupling Beams

The composite coupling beams shall be proportioned to be flexure critical with expected shear strength, $V_{n,exp}$, as follows:

$$V_{n,exp} \ge \frac{2.4M_{p,exp}}{L_{cb}} \tag{H8-6}$$

where

L_{cb} = clear span length of the coupling beam, in. (mm)
$M_{p,exp}$ = expected flexural capacity of composite coupling beam calculated in accordance with Section H8.7a while using the expected yield strength, R_yF_y, for steel and the expected compressive strength, $R_cf'_c$, for concrete, kip-in. (N-mm)
$V_{n,exp}$ = expected shear strength of composite coupling beam calculated in accordance with *Specification* Section I4.2 while using expected yield strength, R_yF_y, for steel and expected compressive strength, $R_cf'_c$, for concrete, kips (N)

6. Composite Wall Strength

The nominal strengths of composite walls shall be calculated in accordance with this section. The design strengths shall be calculated using resistance factor (ϕ) equal to 0.90.

6a. Tensile Strength

The nominal tensile strength shall be determined in accordance with *Specification* Section I2.3.

6b. Compressive Strength

The nominal compressive strength shall be determined in accordance with *Specification* Section I2.3.

6c. Flexural Strength

The nominal flexural strength shall be determined in accordance with *Specification* Section I3.5.

6d. Combined Axial Force and Flexure

The nominal strength of composite walls subjected to combined axial force and flexure shall be determined in accordance with *Specification* Section I5(c).

6e. Shear Strength

The nominal in-plane shear strength, V_n, shall be determined in accordance with *Specification* Section I4.4.

7. Composite Coupling Beam Strength

The nominal strengths of composite coupling beams shall be calculated in accordance with this section. The available strengths shall be calculated using resistance factor, ϕ, equal to 0.90.

7a. Flexural Strength

The nominal flexural strength of composite coupling beams shall be determined in accordance with the *Specification* Section I1.2a.

7b. Shear Strength

The nominal shear strength, V_n, of composite coupling beams shall be determined in accordance with the *Specification* Section I4.2.

8. Coupling Beam-to-Wall Connections

The coupling beam-to-wall connections shall be designed in accordance with the requirements of this section.

8a. Required Flexural Strength

The required flexural strength, M_u, for the coupling beam-to-wall connection shall be 120% of the expected flexural capacity of the coupling beam ($M_{p,exp}$).

8b. Required Shear Strength

The required shear strength, V_u, for the coupling beam-to-wall connection shall be determined using capacity-limited seismic load effect as follows:

$$V_u = 2.4 M_{p,\exp} / L_{cb} \tag{H8-7}$$

where

L_{cb} = clear span length of the coupling beam, in. (mm)

$M_{p,exp}$ = expected flexural capacity of composite coupling beam calculated using expected yield strength, $R_y F_y$, for steel and the expected compressive strength, $R_c f'_c$, for concrete, kip-in. (N-mm)

8c. Rotation Capacity

The coupling beam-to-wall connection shall be detailed to develop a rotation capacity of 0.030 rad before flexural strength decreases to 80% of the flexural plastic strength of the beam. Connection details that have been previously demonstrated to have adequate rotation capacity shall be approved for use. The available rotation capacity of the coupling beam using other connection details shall be verified through testing, advanced analysis calibrated to physical testing, or a combination thereof.

9. Composite Wall-to-Foundation Connections

Where the composite walls are connected directly to the foundation at the point of maximum moment in the walls, the composite wall-to-foundation connections shall be designed in accordance with the requirements of this section.

9a. Required Strengths

The required strengths for the composite wall-to-foundation connections shall be determined using the capacity-limited seismic load effect. The coupling beams shall be assumed to have developed plastic hinges at both ends with the expected flexural capacity of $1.2M_{p,exp}$. The composite walls shall also be assumed to have developed plastic hinges at the base with expected flexural capacity of $1.2M_{p,exp}$, while accounting for the effects of simultaneous axial force. The required shear strength for the composite wall-to-foundation connections shall be equal to the required shear strength for the composite walls calculated in accordance with Section H8.3d.

10. Protected Zones

The following regions shall be designated as protected zones and shall meet the requirements of Section D1.3:

(a) The regions at ends of the coupling beams subject to inelastic straining.

(b) The regions at the base of the composite walls subject to inelastic straining.

The extent of each protected zone shall be determined by rational analysis.

11. Demand Critical Welds in Connections

Where located within the protected zones identified in Section H8.10, welds in the composite wall steel plate splices shall be demand critical and shall satisfy the applicable requirements of Sections A3.4b and I2.3.

User Note: Demand critical welds are generally: (a) CJPs, (b) subject to yield level or higher stress, and (c) in a joint where weld failure would cause significant strength or stiffness degradation of the seismic force resisting system. Most welds in the CC-PSW/CF system can be designed and detailed for the required strengths such that weld stresses remain in the elastic range; otherwise, they may be deemed demand critical.

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. Fabrication and Erection Documents
I2. Fabrication and Erection

I1. FABRICATION AND ERECTION DOCUMENTS

1. Fabrication Documents for Steel Construction

Fabrication documents 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 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 Documents for Steel Construction

Erection documents 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. Fabrication and Erection Documents for Composite Construction

Fabrication documents and erection documents for the steel components of composite steel-concrete construction shall satisfy the requirements of Sections I1.1 and I1.2. The fabrication and erection documents shall also satisfy the requirements of Section A4.3.

User Note: For reinforced concrete and composite steel-concrete construction, the provisions of ACI PRC-315-18 *Guide to Presenting Reinforcing Steel Design Details* and ACI MNL-66(20) *ACI Detailing Manual* 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 6.16, except at the outboard ends of continuity-plate-to-column welds, weld tabs, and weld metal need not be removed closer than 1/4 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. General Provisions
J2. Fabricator and Erector Quality Control Program
J3. Fabricator and Erector Documents
J4. Quality Assurance Agency Documents
J5. Inspection and Nondestructive Testing Personnel
J6. Inspection Tasks
J7. Welding Inspection and Nondestructive Testing
J8. Inspection of High-Strength Bolting
J9. Other Steel Structure Inspections
J10. Inspection of Composite Structures
J11. Inspection of H-Piles

J1. GENERAL PROVISIONS

Quality Control (QC), as specified in this chapter, shall be provided by the fabricator and erector. Quality Assurance (QA), as specified in this chapter, shall be provided by others when required by the authority having jurisdiction (AHJ), applicable building code, purchaser, owner, or engineer of record (EOR), and when required, responsibilities shall be specified in the contract documents. 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 in Section J4 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 QUALITY CONTROL PROGRAM

In addition to the provisions in Chapter N of the *Specification*, the fabricator and erector shall establish, maintain, and implement quality control procedures to ensure that their work is performed in accordance with the additional provisions of this chapter.

J3. 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 by the fabricator and/or erector 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, 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 testing performed and provide the applicable test reports in accordance with AWS D1.8/D1.8M.

(d) Supplemental notch toughness data for intermix testing per AWS D1.8/D1.8M, if applicable.

(e) Manufacturer’s product data sheets or catalog data for welding filler metals and fluxes to be used. The product data sheets shall describe the product, limitations of use, welding parameters, and storage and exposure requirements, including backing, if applicable.

(f) Bolt installation procedures.

2. Documents to be Available for Review for Steel Construction

In addition to the requirements of *Specification* Section N3.2, documents required by the EOR in the contract documents shall be made available by the fabricator or erector for review by the EOR or the EOR’s designee prior to fabrication or erection, as applicable.

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 fabrication documents

(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) Material control procedure

(d) Welder performance qualification records (WPQR) as required by S*tructural Welding Code—Reinforcing Steel* (AWS D1.4/D1.4M)

(e) QC Inspector qualifications

J4. 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

J5. 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 International Code Council *Special Inspection Manual* contains one possible method to establish the qualifications of a bolting inspector.

J6. INSPECTION TASKS

Inspection tasks and documentation for QC and QA for the seismic force-resisting system (SFRS) shall be as provided in accordance with the tables in *Specification* Section N5. Any tasks listed as Observe (O) shall be performed at least daily.

1. 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.

J7. 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.

If welding involves the intermix of FCAW-S weld metal with weld metal from other processes, inspection prior to welding shall include a QC task and QA task to Observe (O) that the use of intermixed weld metals is supported by appropriate documentation in accordance with AWS D1.8/D1.8M.

User Note: AWS D1.8/D1.8M requires that the suitability of combining FCAW-S with other welding processes in a single joint be tested for acceptable CVN properties. These tests, in accordance with AWS D1.8/D1.8M, Annex B, may be performed and documented by the filler metal manufacturer, the Contractor, or an independent testing agency. AWS D1.8/D1.8M, Annex B, contains the minimum requirements for documentation.

If a reinforcing or contouring fillet weld is required, it shall be inspected by the QCI and QAI as a Perform (P) task.

For each individual welder, fit-up of a minimum of ten (10) groove welds or all groove welds if less than ten exist on the project shall be inspected to the Perform (P) task. If the Inspector ascertains that fit up of the groove welds meets the requirements of the welding procedure specification, this task shall be reduced from Perform (P) to Observe (O). Should the fit up not meet the welding procedure specification requirements, the task shall be returned to Perform (P) until such time as the fit up meets the welding procedure requirements.

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 Documentation

Visual welding inspection documentation after welding shall be performed by both quality control and quality assurance personnel. As a minimum, tasks shall be as listed in Table J7.1, where Documentation (D) is required as indicated.

TABLE J7.1
Documentation of Visual Inspection After Welding

Documentation of Visual Inspection After Welding	QC		QA	
	Task	Doc.	Task	Doc.
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[a]	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

[a] 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.

Note:
Doc. = documentation

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. Welds shall be inspected by UT in compliance with AWS D1.8/D1.8M.

Magnetic particle testing (MT) shall be performed on 25% of all beam-to-column CJP groove welds. Welds shall be inspected by MT in compliance with AWS D1.8/D1.8M.

For ordinary moment frames in structures in risk categories I or II, UT and MT of CJP groove welds shall be required only for demand critical welds.

The rate of UT and MT is permitted to be reduced in accordance with Sections J7.2g and J7.2h, respectively.

2b. PJP Groove Weld NDT

UT shall be performed using written procedures and UT technicians qualified in accordance with AWS D1.8/D1.8M. 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, and within 5/16 in. (8 mm) of the root, shall be considered false indications in procedure and personnel qualification. Procedures qualified using mock-ups with artificial flaws 1/16 in. (2 mm) in their smallest dimension are permitted.

The initial 5/16 in. (8 mm) from the root of the bevel shall be disregarded from the UT evaluation. QC shall perform visual testing (VT) of the root.

UT examination of welds using alternative techniques in compliance with AWS D1.1/D1.1M Annex O is permitted.

Weld discontinuities located within the groove weld throat shall be inspected by UT in compliance with AWS D1.8/D1.8M.

The rate of UT is permitted to be reduced in accordance with Section J7.2g.

(1) Column Splice Welds

UT is not required for partial-joint-penetration (PJP) groove welds in column splices designed to meet the requirements of Section D2.5b. UT shall be performed as described in this section on 100% of PJP welds meeting the requirements of Sections E3.6g.2 and E3.6g.4.

(2) Column to Base Plate Welds

UT shall be performed by QA on 100% of partial-joint-penetration (PJP) groove welds in column to base plate welds.

(3) Alternative Approach to UT

When requested by the fabricator or erector and approved by the engineer of record, as an alternative to performing UT on PJP welds, a combination of visual testing (VT) and magnetic particle testing (MT) is permitted to be used in accordance with written examination procedures.

2c. Base Metal NDT for Lamellar Tearing and Laminations

After joint completion, base metal thicker than 1-1/2 in. (38 mm) loaded in tension in the through-thickness direction in T- and corner-joints, where the connected material is greater than 3/4 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 8.2, where t is the thickness of the part subjected to the through-thickness strain.

2d. Beam Cope and Weld Access Hole NDT

At welded splices and connections, thermally cut surfaces of beam copes and weld access holes shall be tested using magnetic particle testing or penetrant testing, when the flange thickness exceeds 1-1/2 in. (38 mm) for rolled shapes, or when the web thickness exceeds 1-1/2 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 joints receiving UT as required under Section J7.2a. Except for demand critical welds, the rate of MT is permitted to be reduced in accordance with Section J7.2h. MT of continuity plate weld tab removal sites is not required.

2g. Reduction of Percentage of Ultrasonic Testing

The 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 percentage 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 at repair sites, weld tab removal sites for demand critical welds, backing removal sites, and weld access holes.

J8. INSPECTION OF HIGH-STRENGTH BOLTING

Bolting inspection shall satisfy the requirements of *Specification* Section N5.6.

J9. OTHER STEEL STRUCTURE INSPECTIONS

Other inspections of the steel structure shall satisfy the requirements of *Specification* Section N5.8 and this section. The inspection tasks listed in Table J9.1 shall be performed, as applicable.

TABLE J9.1
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 or unapproved attachments made by fabricator or erector, as applicable	P	D	P	D

Note:
Doc. = documentation

> **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.

J10. 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.

Inspection of structural steel elements used in composite structures shall comply with the requirements of this Chapter. 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 J7.

The minimum inspection tasks shall be as listed in Tables J10.1 and J10.2, where applicable to the type of composite construction.

TABLE J10.1
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	–
If welded, 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	–

Note:
Doc. = documentation
– = indicates no documentation is required

TABLE J10.2
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	–

Note:
Doc. = documentation
– = indicates no documentation is required

In composite structures, the concrete compressive strength shall be tested and documented at the specified age.

J11. 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. The inspection tasks listed in Table J11.1 shall be performed as applicable.

TABLE J11.1
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

Note:
Doc. = documentation

User Note: Splices of H-piles, as members subjected to axial and flexural loads, should be inspected as columns.

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

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 (AHJ).

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, and 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 documents 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 (EOR) and the AHJ.

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 in accordance with 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 AHJ.

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 (21 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 AHJ 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 clear description of the test subassemblage, including key dimensions, boundary conditions at loading and reaction points, and location of lateral braces.

(b) The connection detail, including 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 documents, 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 EOR and the AHJ. 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 AHJ.

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 50% nor more than 150% 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 AHJ.

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 earthquake displacement, in. (mm)

Δ_{by} = value of deformation quantity, Δ_b, at first yield of test specimen, in. (mm)

The frame drift at the design earthquake displacement 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/A6M, 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 clear description of the test specimen, including key dimensions, boundary conditions at loading and reaction points, and location of lateral bracing, if any.

(b) The connection details including 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 Section 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 documents, 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 AHJ.

APPENDIX 1

DESIGN VERIFICATION USING NONLINEAR RESPONSE HISTORY ANALYSIS

This appendix provides requirements for the use of nonlinear response history analysis for the design verification of steel and composite steel-concrete structures subjected to earthquake ground shaking.

The appendix is organized as follows:

1.1. SCOPE

Wherever these provisions refer to the applicable building code and there is none, the requirements for performing nonlinear response history analysis of seismic force-resisting systems of structural steel or of structural steel acting compositely with reinforced concrete shall be in accordance with those stipulated in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7) Chapter 16 and in conformance with this Appendix.

This appendix shall be limited to the systems specified in Section 1.7. All systems designed or verified by this appendix shall meet the requirements of the provisions within Chapters A through K using load and resistance factor design (LRFD). When approved by the authority having jurisdiction, exceptions to such requirements may be taken as justified by the nonlinear analysis in accordance with this appendix. All exceptions, including exceptions to this appendix, shall be documented and justified by the engineer of record. Where reference is made to the AISC *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings* (ANSI/AISC 342), it shall be permitted to use other substantiated guidelines subject to the approval by the authority having jurisdiction.

User Note: Although ANSI/AISC 342 is intended for existing structures, many of the provision therein apply to this Appendix and are referenced but not repeated herein.

User Note: When the analysis and design are subject to an independent structural design review according to the applicable building code or the authority having jurisdiction, ASCE/SEI 7, Chapter 16, includes requirements for such a review.

1.2. EARTHQUAKE GROUND MOTIONS

Ground motion acceleration histories shall be determined per the applicable building code.

1.3. LOAD FACTORS AND COMBINATIONS

Load factors and combinations for evaluation by nonlinear response history analysis shall conform to the requirements in the applicable building code.

1.4. GENERAL MODELING REQUIREMENTS

Models for analysis shall be three-dimensional and shall conform to the requirements of the applicable building code. Members shall be designated and modeled as either force-controlled or deformation-controlled in accordance with the applicable building code.

Modeling of member nonlinear behavior, including effective stiffness, expected strength, expected deformation capacity, and hysteresis under force or deformation reversals, shall be substantiated by physical test data, detailed analyses, or other supporting evidence. The provisions given in this appendix shall be deemed to satisfy this requirement.

For deformation controlled members that are modeled inelastically, degradation in member strength or stiffness shall be included in the numerical models unless it can be demonstrated that the demand is not sufficiently large to produce these effects.

Member initial geometric imperfections shall be included in the numerical model in members where imperfections are required to capture the forces resisted by the member under nonlinear response.

Component modeling shall be based on expected material properties. Expected material strengths shall be as specified in Section A3.2.

Seismic force resisting systems shall be analyzed as required in Chapters E, F, G, and H. Component-specific and system-specific modeling and analysis shall conform to the requirements in Sections 1.5 through 1.7. The gravity framing system shall be modeled in the nonlinear analysis unless it can be demonstrated to not significantly contribute to the seismic force and deformation demands in the structure. If gravity columns are not explicitly modeled, the leaning column effect of the gravity system shall be modeled.

1.5. MEMBER MODELING REQUIREMENTS

The member modeling requirements in this section are invoked by the applicable requirements for each system, as specified in Section 1.7.

1. Beams

Modeling of beams shall address the following as applicable:

(a) Force-deformation/moment-rotation models for beams shall include inelastic flexural deformations, taking into account material yielding, strain hardening, and degradation due to local buckling and lateral-torsional buckling effects. The model shall consider *P-M* interaction unless the ratio of P/P_y is 0.1 or less.

(b) Where concentrated beam hinge models are used, the moment-rotation response shall be determined using the parameters in ANSI/AISC 342 Chapter C.

(c) Where fiber-type beam hinge or distributed plasticity models are used, strain hardening shall be considered when appropriate. Unless the fiber model accounts for local buckling and fracture effects, the inelastic beam rotations shall be limited to the hinge rotation at the peak strength of an equivalent concentrated plastic hinge model.

(d) Beam hinge properties shall be modeled considering the cross section characteristics of the beam. Concentrated hinge or fiber hinge models shall be located at the expected plastic hinge locations. Locating the concentrated hinge away from the actual hinge location is permitted, provided that the hinge properties are adjusted to account for the discrepancy in locations, considering the beam moment gradient and the difference between actual and modeled beam lengths.

(e) Where the steel beam acts compositely with a concrete floor slab (solid slab or slab on steel deck), adjusting the beam stiffness and hinge strength to account for composite action under positive and negative bending shall be considered, taking into account the force transfer mechanisms in the beam-to-column (or beam-to-wall) connections.

2. Links

Modeling of links shall address the following as applicable:

(a) Shear and flexural yielding or buckling, post-yielding or post-buckling, peak strength of the link.

(b) The component properties of the link shall be determined per ANSI/AISC 342 Chapter C, taking into account the effect of axial force in the link.

3. Columns

Modeling of columns shall address the following as applicable:

(a) Force-deformation model for columns shall include inelastic flexural deformations under combined bending and axial loads, taking into account yielding, strain hardening, local buckling effects, and flexural-torsional response.

(b) Where concentrated hinge models are used, the moment-rotation response shall be determined using the parameters in ANSI/AISC 342 Chapter C.

(c) Where fiber-type beam hinge or distributed plasticity models are used, strain hardening shall be considered. Unless the fiber model properties are adjusted to account for local buckling and flexural-torsional effects, the inelastic column rotations shall be limited to the peak point of the plastic hinge rotation of an equivalent concentrated hinge model.

4 Braces (except buckling-restrained braces)

Modeling of braces shall address the following as applicable:

(a) Brace yielding and elongation, including accumulation of permanent elongation due to cyclic loading.

(b) Brace buckling, including effects of initial imperfections, residual stress, equilibrium on the deformed brace geometry, and axial-flexural interaction.

(c) Post-buckling strength degradation under cyclic loading.

(d) Fracture due to low-cycle fatigue degradation and peak ductility effects. If fracture is not included in the brace model, the peak deformation shall not exceed the values specified in ANSI/AISC 342 Section C3.

(e) Rotational restraint of end connections.

(f) Restraint effects and appropriate constraints at locations where braces overlap or intersect.

(g) Actual brace end locations, which are offset from workpoint locations.

5. Buckling-Restrained Braces

Modeling of buckling-restrained braces shall address the following as applicable:

(a) Elastic stiffness considering variations in brace cross-sectional area along the length.

(b) Brace yielding in tension and compression, including accumulation of permanent axial deformation due to cyclic loading.

(c) The peak deformation demand and cumulative deformation demand for brace element models shall be limited to capacities determined from representative cyclic tests conducted in accordance with Section K3.

(d) Difference between tension and compression yield forces, as specified in Section F4.2.

(e) Actual brace end locations, which are offset from workpoint locations.

6. Steel Plate Shear Walls

Modeling of plate shear walls shall address the following as applicable:

(a) Yielding and elongation of the steel plate.

(b) Pinching of the hysteretic loop due to shear buckling.

(c) Distribution of transverse forces on horizontal and vertical boundary elements.

(d) The valid range of SPSW element models shall not extend beyond $15\Delta_y$ where Δ_y is the yield elongation of a diagonal strip of the web plate. For methods other than strip modeling an equivalent displacement measure shall be used.

7. Composite Plate Shear Walls

Modeling of composite plate shear walls shall address the following as applicable:

(a) Yielding of steel plates;

(b) Inelastic local buckling of steel plates in compression;

(c) Concrete compression behavior including effects of confinement on infill concrete;

(d) Concrete tension cracking;

(e) Pinching of hysteretic loops due to concrete crack closure and steel cyclic local buckling;

(f) Fracture of steel plates due to plastic strain accumulation.

1.6. CONNECTION MODELING REQUIREMENTS

The connection modeling requirements in this section are triggered by the applicable requirements for each system as specified in Section 1.7.

1. Panel Zones

Modeling of panel zone flexibility and yielding shall address the following as applicable:

(a) The panel zone expected shear yield strength shall be calculated with the expected steel yield strength, R_yF_y, using the equation in *Specification* Section J10.6.

(b) The panel zone finite size and deformations shall be modeled.

(1) Where panel zone shear demands exceed the expected shear yield strength, the panel zone shall be modeled explicitly with a model that takes into account the finite size and inelastic response.

(2) Where panel zone shear demands are less than the expected shear yield strength, the effect of finite size and elastic panel zone deformations shall be modeled in accordance with ANSI/AISC 342, Section C4.

2. Partially Restrained Connections

The response characteristics of partially restrained connections shall be included in the model. The response characteristics of the partially-restrained connection shall be based on the technical literature or established by analytical or experimental means.

3. Column Bases

Deformation and potential failure modes of column base plates and connections to the foundation shall be considered. Where capacity design is used to prevent inelastic response and limit deformations in the column base connection, the column base connection may be modeled assuming full fixity to the foundation. Otherwise, the column base connections shall be modeled using concentrated springs or fiber-type section models to capture the base connection deformations.

Foundation components shall be modeled in the analysis, unless it can be demonstrated that the foundation components remain essentially elastic and their deformations are small enough to not contribute to the structural system response.

4. Brace Gusset Plates

Brace gusset plate geometry, stiffness, strength, and inelastic response shall be considered in the model. The stiffening effect of gusset plates on adjacent beams and columns shall be considered in the model. Both in-plane and out-of-plane gusset plate properties shall be considered in the model, including interaction with the attached braces.

1.7. SYSTEM REQUIREMENTS

Component actions for the specified lateral force resisting systems shall be in accordance with this section. Components shall be modeled according to Sections 1.5 and 1.6. Definitions for member criticality and requirements for force and deformation controlled members are provided in the applicable building code.

User Note: Definitions for member criticality are provided in ASCE/SEI 7, Chapter 16.

1. Special Moment Frames (SMF)

Component actions for SMF systems shall be designated as force controlled or deformation-controlled and their criticality shall be as designated per Table A-1.7.1.

TABLE A-1.7.1 Requirements for Special Moment Frames (SMF)			
Item	**Action**	**Force or Deformation Controlled**	**Criticality**
Beam	Flexure	Deformation	Ordinary
Beam	Shear	Force	Critical
Column with $P_G/P_{ye} \le 0.6$	Axial	Force	Critical
Column with $P_G/P_{ye} \le 0.6$	Flexure	Deformation	Ordinary
Column with $P_G/P_{ye} > 0.6$	Axial, Flexure	Force	Critical
Column	Shear	Force	Critical
Panel Zone	Shear	Deformation	Ordinary
Column Base	Flexure	Deformation	Ordinary
Column Base	Axial, Shear	Force	Critical
P_G = axial force component of the gravity load, kips (N) P_{ye} = expected axial yield strength, kips (N)			

User Note: Substantiated guidelines for nonlinear modeling of steel moment frames include ANSI/AISC 342 and *Guidelines for Nonlinear Structural Analysis for Design of Buildings, Part IIa – Steel Moment Frames* or equivalent (NIST GCR 17-917-46v2).

2. Special Concentrically Braced Frames (SCBF)

Component actions for SCBF systems shall be designated as force controlled or deformation controlled, and their criticality shall be designated in accordance with Table A-1.7.2.

TABLE A-1.7.2 Requirements for Special Concentrically Braced Frames (SCBF)			
Item	**Action**	**Force or Deformation Controlled**	**Criticality**
Beam	Flexure	Deformation	Ordinary
Beam	Axial, Shear	Force	Critical
Column with $P_G/P_{ye} \le 0.6$	Axial	Force	Critical
Column with $P_G/P_{ye} \le 0.6$	Flexure	Deformation	Ordinary
Column with $P_G/P_{ye} > 0.6$	Axial, Flexure	Force	Critical
Column	Shear	Force	Critical
Brace	Axial	Deformation	Ordinary
Brace Connection	Axial	Force	Critical
Brace Connection	Flexure	Force or Deformation	Critical
Panel Zone	Shear	Deformation	Ordinary
Column Base	Flexure	Deformation	Ordinary
Column Base	Axial, Shear	Force	Critical

3. Eccentrically Braced Frames (EBF)

Component actions for EBF systems shall be designated as force controlled or deformation controlled, and their criticality shall be designated per Table A-1.7.3.

TABLE A-1.7.3
Requirements for Eccentrically Braced Frames (EBF)

Item	Action	Force or Deformation Controlled	Criticality
Beam	Axial, Flexure	Force	Ordinary
Beam	Shear	Force	Critical
Column with $P_G/P_{ye} \leq 0.6$	Axial	Force	Critical
Column with $P_G/P_{ye} \leq 0.6$	Flexure	Deformation	Ordinary
Column with $P_G/P_{ye} > 0.6$	Axial, Flexure	Force	Critical
Column	Shear	Force	Critical
Brace	Axial	Force	Critical
Brace Connection	Axial	Force	Critical
Link	Shear, Flexure	Deformation	Ordinary
Link	Axial	Force	Critical
Link Connection	Flexure, Shear, Axial	Force	Critical
Panel Zone	Shear	Deformation	Ordinary
Column Base	Flexure	Deformation	Ordinary
Column Base	Axial, Shear	Force	Critical

4. Buckling-Restrained Braced Frames (BRBF)

Component actions for BRBF systems shall be designated as force controlled or deformation controlled, and their criticality shall be designated per Table A-1.7.4.

TABLE A-1.7.4
Requirements for Buckling-Restrained Braced Frames (BRBF)

Item	Action	Force or Deformation Controlled	Criticality
Beam	Flexure	Deformation	Ordinary
Beam	Shear	Force	Critical
Column with $P_G/P_{ye} \leq 0.6$	Axial	Force	Critical
Column with $P_G/P_{ye} \leq 0.6$	Flexure	Deformation	Ordinary
Column with $P_G/P_{ye} > 0.6$	Axial, Flexure	Force	Critical
Column	Shear	Force	Critical
Brace	Axial	Deformation	Ordinary
Brace Connection	Axial	Force	Critical
Panel Zone	Shear	Deformation	Ordinary
Column Base	Flexure	Deformation	Ordinary
Column Base	Axial, Shear	Force	Critical

5. Special Plate Shear Walls (SPSW)

Component actions for SPSW systems shall be designated as force controlled or deformation controlled, and their criticality shall be designated per Table A-1.7.5.

TABLE A-1.7.5
Requirements for Special Plate Shear Walls (SPSW)

Item	Action	Force or Deformation Controlled	Criticality
HBE	Flexure	Deformation	Ordinary
HBE	Shear	Force	Critical
VBE (midspan)	Axial, Flexure	Force	Critical
VBE at connection with $P_G/P_{ye} \le 0.6$	Axial	Force	Ordinary
VBE at connection with $P_G/P_{ye} \le 0.6$	Flexure	Deformation	Ordinary
VBE at connection with $P_G/P_{ye} > 0.6$	Axial, Flexure	Force	Critical
VBE	Shear	Force	Critical
Panel Zone	Shear	Deformation	Ordinary
VBE Base	Flexure	Deformation	Ordinary
VBE Base	Axial, Shear	Force	Critical
HBE = horizontal boundary elements VBE = vertical boundary elements			

6. Composite Plate Shear Walls—Concrete Filled (C-PSW/CF)

Component actions for C-PSW/CF systems shall be designated as force controlled or deformation controlled, and their criticality shall be designated per Table A-1.7.6.

TABLE A-1.7.6
Requirements for Composite Plate Shear Walls-Concrete Filled (C-PSW/CF)

Item	Action	Force or Deformation Controlled	Criticality
Beam	Flexure	Deformation	Ordinary
Beam	Shear	Force	Critical
Wall	Axial, Flexure	Deformation	Critical
Wall	Shear	Force	Critical
Wall to Foundation Connection	Axial, Shear, Flexure	Force	Critical

7. Gravity Framing Systems

When included in the model, the requirements for gravity systems shall be as follows:

(a) Component actions for gravity systems shall be designated as force controlled or deformation controlled, and their criticality shall be designated per Table A-1.7.7.

(b) Gravity Steel Beams: Model bare steel beams or composite beam as elastic.

(c) Gravity Columns: For analysis to larger drifts (story drift ratios greater than 0.02), the columns should be modeled as inelastic. If the gravity columns meet the moderately to highly ductile requirements, their inelastic response may be modeled in accordance with Section 1.5.3.

(d) Model the behavior of partially restrained gravity connections according to Section 1.6.2

TABLE A-1.7.7
Requirements for Gravity Systems

Item	Action	Force or Deformation Controlled	Criticality
Gravity Connection	Shear	Force	Critical
Gravity Connection	Flexure	Deformation[a]	Ordinary

[a]The gravity connection shall be designed to maintain its required shear strength under the imposed flexural deformations.

1.8 GLOBAL ACCEPTANCE CRITERIA

Story drifts shall be limited per the applicable building code.