ANSI/AISC 342-22
An American National Standard

Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings

August 1, 2022

Approved by the Committee on Specifications

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
FOUNDED 1921

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Stronger.
Steel.

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PREFACE

(This Preface is not part of ANSI/AISC 342-22, *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*; it is included for informational purposes only.)

These Provisions are based upon past successful usage and advances in the state of knowledge relative to the retrofit of structures subjected to seismic loads. Where required herein or by ASCE/SEI 41-17, *Seismic Evaluation and Retrofit of Existing Buildings*, these Provisions are intended to be used in conjunction with ANSI/AISC 360-22, *Specification for Structural Steel Buildings*, and ANSI/AISC 341-22, *Seismic Provisions for Structural Steel Buildings*.

These Provisions are ANSI-approved and have been developed as a consensus document using ANSI-accredited procedures to provide a uniform practice for the seismic retrofit of steel-framed buildings, and also those buildings that may include composite, cast iron, and wrought iron elements. The intention is to provide design criteria to be used in conjunction with ASCE/SEI 41-17. It is intended that the next edition of ASCE/SEI 41 adopt these Provisions in Chapter 9 of that standard. The intention is also to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems, which occur in the full range of structural design.

The Provisions are a result of the consensus deliberations of a committee of structural engineers with wide experience and high professional standing, representing a wide geographical distribution throughout the United States. The committee includes approximately equal numbers of engineers in private practice and code agencies, engineers involved in research and teaching, and engineers employed by steel fabricating and producing companies.

The Symbols, Glossary, and Abbreviations to these Provisions are an integral part of the Provisions. The Symbols, Glossary, and Abbreviations are consistent with those used in ASCE/SEI 41-17 for ease of adoption by the next edition of ASCE/SEI 41, and for ease of use with ASCE/SEI 41-17. A nonmandatory Commentary has been prepared to provide background for the Provisions. The user is encouraged to consult the Commentary. Additionally, nonmandatory User Notes are interspersed throughout the Provisions to provide concise and practical guidance in the application of the Provisions.

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SYMBOLS

Some definitions in the list below have been simplified in the interest of brevity. In all cases, the definitions given in the body of these Provisions govern. Symbols without text definitions, or used only in one location and defined at that location, are omitted in some cases. The section or table number in the righthand column refers to the Section where the symbol is first defined.

Symbol	Definition	Section
A_b	Gross area of rivet or bolt, in.^{2} (mm^2)	C5.3a.2(a)(i)
A_{cf}	Area of column flange, in.^{2} (mm^2)	C4.4a.2
A_{conn}	Cross-sectional area of BRB connection, in.^{2} (mm^2)	C3.4a.1.b
A_{core}	Cross-sectional area of BRB core, in.^{2} (mm^2)	C3.4a.1.b
A_e	Effective net area of horizontal angle leg, in.^{2} (mm^2)	C5.3a.2(a)(ii)
A_e	Effective net area of split-tee stem, in.^{2} (mm^2)	C5.3a.2(b)(iii)
A_e	Effective net area of flange plate, in.^{2} (mm^2)	C5.3a.2(c)
A_g	Gross area of cross section, in.^{2} (mm^2)	C3.2b
A_g	Gross area of flange plate, in.^{2} (mm^2)	C5.3a.2(c)
A_g	Gross area of horizontal angle leg, in.^{2} (mm^2)	C5.3a.2(a)(ii)
A_g	Gross area of split-tee stem, in.^{2} (mm^2)	C5.3a.2(b)(iii)
A_g	Gross area of smaller member, in.^{2} (mm^2)	C5.3b.3.b
A_g	Gross area of gusset plate, in.^{2} (mm^2)	C7.2b
A_g	Gross area of link beam, in.^{2} (mm^2)	E2.4b
A_s	Effective shear area of the cross section, in.^{2} (mm^2)	C2.4a.1.b
B_w	Effective gusset plate width, in. (mm)	C7.2b
E	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	C2.4a.1.b
E_{ci}	Modulus of elasticity of cast iron = 20,000 ksi (138 000 MPa)	I3.1
F_{cr}	Critical stress, ksi (MPa)	I3.1
F_{crL}	Critical stress of the plate computed using F_{yL}, ksi (MPa)	C7.3b.4
F_e	Elastic buckling stress, ksi (MPa)	I3.1
F_n	Nominal stress, ksi (MPa)	C7.3b.4
F_{nt}	Nominal tensile stress of bolt or driven rivet, given in *Specification* Section J3.7 or *Specification* Appendix 5, Section 5.3, ksi (MPa)	C5.3a.2(a)(iii)
F_{nv}	Nominal shear stress of bolt or driven rivet for bearing-type connections, given in *Specification* Section J3.7 or *Specification* Appendix 5, Section 5.3, ksi (MPa)	C5.3a.2(a)(i)
F_{nw}	Nominal stress of weld metal, ksi (MPa)	C5.3a.2(c)
F_{te}	Expected tensile stress of bolt or rivet, taken as F_{nt}, given in the *Specification*, ksi (MPa)	C5.3a.2(a)(iii)
F_u	Specified minimum tensile strength, ksi (MPa)	A5.2a
F_{ue}	Expected tensile strength, ksi (MPa)	A5.2a
F_{uL}	Lower-bound tensile strength, ksi (MPa)	A5.2a
F_y	Specified minimum yield stress, ksi (MPa)	A5.2a
F_y	Specified minimum yield stress of the column web, ksi (MPa)	C4.4a.2

Symbol	Definition	Section
F_{ve}	Expected shear stress of bolt or rivet, taken as F_{nv}, given in the *Specification*, ksi (MPa)	C5.3a.2(a)(i)
F_{ye}	Expected yield stress, ksi (MPa)	A5.2a
F_{yL}	Lower-bound yield stress, ksi (MPa)	A5.2a
G	Shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)	C2.4a.1.b
I	Moment of inertia about the axis of bending, in.^{4} (mm^4)	C2.4a.1.b
I_b	Moment of inertia of beam about the axis of bending, in.^{4} (mm^4)	C5.2a.2
I_c	Moment of inertia of a column or brace about the axis of bending, in.^{4} (mm^4)	C3.2b
K	Effective length factor	C3.3a.1
K_e	Elastic shear stiffness, kip/in. (N/mm)	C2.4a.2.b
K_{IC}	Fracture toughness parameter determined in accordance with Table C5.3 or by other approved methods, ksi$\sqrt{\text{in.}}$ (MPa$\sqrt{\text{mm}}$)	C5.3b.3.b
K_w	Elastic shear stiffness of a steel plate wall with a web plate that is sufficiently thick or stiffened to avoid shear buckling, kip/in. (N/mm)	C6.2c
K_θ	Elastic stiffness of the partially restrained connection, kip-in./rad (N-mm/rad)	C5.2a.2
K_θ	Rotational stiffness of base connection, kip-in./rad (N-mm/rad)	C5.2b
K_θ	Elastic rotational spring stiffness in the plane of brace buckling, kip-in./rad (N-mm/rad)	C7.2b
L	Laterally unbraced length of member, in. (mm)	Table C3.6
L	Length of span, in. (mm)	Table C5.4
L_{avg}	Average unrestrained length of gusset plate, in. (mm)	C7.2b
L_b	Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm)	Table C5.4
L_c	Effective length, in. (mm)	C3.3a.1
L_c	Laterally unbraced length of column, in. (mm)	I3.1
L_{cf}	Length of beam taken as the clear span between column flanges, in. (mm)	C5.4a.1.a.1
L_{CL}	Length of beam taken between column centerlines, in. (mm)	C2.4a.1.b
L_{CL}	Length of member taken between beam centerlines, in. (mm)	C3.4a.2.b
L_{CL}	Centerline length of the beam taken between joints, in. (mm)	C5.2a.2
L_{conn}	Length of BRB connection, in. (mm)	C3.4a.1.b
L_{core}	Length of BRB core, in. (mm)	C3.4a.1.b
L_{ee}	End-to-end brace length, in. (mm)	C3.2a
L_{ell}	Rotational clearance, in. (mm)	C7.4a
L_p	Limiting laterally unbraced length for the limit state of yielding, in. (mm)	E1.4b
L_r	Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm)	E1.4b
L_v	Clear length between supports that resist translation in the direction of the shear force, in. (mm)	C2.1
L_{vert}	Vertical clearance between the brace end and beam flange, in. (mm)	C7.4a
M_{CE}	Expected flexural strength, kip-in. (N-mm)	C2.1
M_{CEx}	Expected flexural strength of the member about the *x*-axis, kip-in. (N-mm)	C3.4a.2.a.2

Symbol	**Definition**	**Section**
M_{CExLB}	Expected local buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.2
M_{CExLTB}	Expected lateral-torsional buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.2
M_{CEy}	Expected flexural strength of the member about the y-axis, kip-in. (N-mm)	C3.4a.2.a.2
M_{CEyLB}	Expected local buckling flexural strength of the member about the y-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.2
M_{CL}	Lower-bound flexural strength, kip-in. (N-mm)	C2.3b.1
M_{CLc}	Lower-bound flexural strength of connection at the face of column, determined in accordance with Section C5.3b.1, kip-in. (N-mm)	C5.4a.1.a.1
M_{CLx}	Lower-bound flexural strength of the member about the x-axis, kip-in. (N-mm)	C3.4a.2.a.2
M_{CLxLB}	Lower-bound local buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.2
M_{CLxLTB}	Lower-bound lateral-torsional buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.2
M_{CLy}	Lower-bound flexural strength of the member about the y-axis, kip-in. (N-mm)	C3.4a.2.a.2
M_{CLyLB}	Lower-bound local buckling flexural strength of the member about the y-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.2
M_{Cx}	Flexural strength of the member about the major principal axis (x-axis) at P_{UF}, kip-in. (N-mm)	C3.4a.2.a.2
M_{CxLB}	Local buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.2
M_{CxLTB}	Lateral-torsional buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.2
M_{Cy}	Flexural strength of the member about the minor principal axis (y-axis) at P_{UF}, kip-in. (N-mm)	C3.4a.2.a.2
M_{CyLB}	Local buckling flexural strength of the member about the y-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.2
M_n	Nominal flexural strength, kip-in. (N-mm)	C2.3a.1
M_{pcex}	Expected plastic flexural strength of the section about the major principal axis (x-axis) at P_{UF}, kip-in. (N-mm)	C3.4a.2.a.1
M_{pcey}	Expected plastic flexural strength of the section about the minor principal axis (y-axis) at P_{UF}, kip-in. (N-mm)	C3.4a.2.a.1
M_{pe}	Expected plastic flexural strength, kip-in. (N-mm)	C2.3a.1
M_{pe}	Expected plastic flexural strength of the section, at the location of the plastic hinge, about the axis of bending defined in Section C2.3a.1, kip-in. (N-mm)	C2.4a.1.a
M_{peb}	Expected plastic flexural strength of beam, determined in accordance with Section C2.3a at the plastic hinge location, projected to the face of column, kip-in. (N-mm)	C5.4a.1.a.1
M_{pex}	Expected plastic flexural strength of the section about the x-axis in the absence of axial force, determined in accordance with Section C2.3a.1 at $P_{UF} = 0$, kip-in. (N-mm)	C3.4a.2.a.1

Symbol	Definition	Section
M_{pey}	Expected plastic flexural strength of the section about the y-axis in the absence of axial force, determined in accordance with Section C2.3a.1 at $P_{UF} = 0$, kip-in. (N-mm)	C3.4a.2.a.1
M_{pLx}	Lower-bound plastic flexural strength about the x-axis, kip-in. (N-mm)	C3.4b.2.a
M_{pLy}	Lower-bound plastic flexural strength about the y-axis, kip-in. (N-mm)	C3.4b.2.a
M_{UDx}	Bending moment about the x-axis determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kip-in. (N-mm)	C3.4a.2.a.1
M_{UDy}	Bending moment about the y-axis determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kip-in. (N-mm)	C3.4a.2.a.1
M_{UFx}	Bending moment about the x-axis determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kip-in. (N-mm)	C3.4a.2.a.2
M_{UFy}	Bending moment about the y-axis determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kip-in. (N-mm)	C3.4a.2.a.2
M_{Ux}	Bending moment about the x-axis, kip-in. (N-mm)	C3.4a.2.a.2
M_{Uy}	Bending moment about the y-axis, kip-in. (N-mm)	C3.4a.2.a.2
$M_{y(beam)}$	Expected first yield moment of beam, kip-in. (N-mm)	C5.4a.1.a.1
N_b	Least number of bolts or rivets connecting the top or bottom angle to the beam flange	C5.3a.2(a)(i)
N_b	Least number of bolts or rivets connecting the flange of the top or bottom split-tee to the column flange	C5.3a.2(b)(ii)
P_{CE}	Expected compressive strength, kips (N)	C2.1
P_{CE}	Expected tensile strength of horizontal angle leg, kips (N)	C5.3a.2(a)(ii)
P_{CL}	Lower-bound compressive strength, kips (N)	C3.3b.1
P_G	Axial force component of the gravity load as determined by ASCE/SEI 41, Equation 7-3, kips (N)	Table C3.6
P_n	Nominal axial strength, kips (N)	C3.3a.1
P_n	Nominal compressive strength, kips (N)	C3.3a.1
P_{ns}	Cross-section compressive strength, kips (N)	C3.2b
P_r	Required axial strength using LRFD or ASD load combinations, kips (N)	Table C2.1
P_r	Required axial compressive strength using LRFD or ASD load combinations, kips (N)	C3.2b
P_{UD}	Tensile force in the member determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)	C3.4a.2.a.2
P_{UF}	Axial force (compression or tension) determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)	Table C2.1
P_{UF}	Axial compressive force determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)	C3.2b
P_{ye}	Expected axial yield strength, kips (N)	C3.2b
$P_{ye,cf}$	Expected axial yield strength of the column flange, kips (N)	C4.4a.2
P_{yL}	Lower-bound axial yield strength, kips (N)	C3.4b.2.a
Q	Component force, kips (N) or kip-in. (N-mm)	Figure C1.1
Q_{CE}	Expected component strength, kips (N) or kip-in. (N-mm)	B2.3a
Q_{CL}	Lower-bound component strength, kips (N) or kip-in. (N-mm)	B2.3b

Symbol	Definition	Section
Q_{CL}	Lower-bound shear strength, kips (N)	G1.3b
Q_{UF}	Force-controlled action caused by gravity loads and earthquake forces, kips (N) or kip-in. (N-mm)	C7.3b
Q_y	Component yield strength, kips (N) or kip-in. (N-mm)	Figure C1.1
Q_y	Expected component yield strength, kips (N) or kip-in. (N-mm)	C2.3a.1
R_n	Nominal strength	C4.3a
R_t	Ratio of the expected tensile strength to the specified minimum tensile strength, F_u	A5.2a
R_y	Ratio of the expected yield stress to the specified minimum yield stress, F_y	A5.2a
S	Elastic section modulus about the axis of bending, in.^{3} (mm^3)	C2.4a.1.a
S_b	Elastic section modulus of beam, in.^{3} (mm^3)	C5.3a.1
S_n	Nominal diaphragm strength, kip/in. (N/mm)	G1.3a
S_{nb}	Nominal shear strength per unit length of a diaphragm controlled by out-of-plane buckling, kip/in. (N/mm)	G1.3a
S_{nf}	Nominal shear strength per unit length of diaphragm controlled by connections, kip/in. (N/mm)	G1.3a
S_x	Elastic section modulus of the smaller member taken about the *x*-axis, in.^{3} (mm^3)	C5.3b.3.b
S_y	Elastic section modulus of the smaller member taken about the *y*-axis, in.^{3} (mm^3)	C5.3b.3.b
T_{CE}	Expected tensile strength, kips (N)	C2.1
T_{CL}	Lower-bound tensile strength, kips (N)	C3.3b.1
T_{CL}	Gross section yield strength, kips (N)	C7.3b.1
T_{CL}	Block shear rupture strength, kips (N)	C7.3b.3
V_{CE}	Expected shear strength, kips (N)	C2.1
V_{CL}	Lower-bound shear strength, kips (N)	C2.3b.1
V_n	Nominal shear strength, kips (N)	C2.3a.2
V_{pcex}	Expected plastic shear strength of the section along the major principal axis (*x*-axis) at P_{UF}, kips (N).	C3.4a.3.a
V_{pcey}	Expected plastic shear strength of the section along the minor principal axis (*y*-axis) at P_{UF}, kips (N).	C3.4a.3.a
V_{pe}	Expected plastic shear strength, kips (N)	C2.3a.2
V_{pex}	Expected plastic shear strength along the *x*-axis in the absence of axial force, kips (N)	C3.4a.3.a
V_{pey}	Expected plastic shear strength along the *y*-axis in the absence of axial force, kips (N)	C3.4a.3.a
V_{pLx}	Lower-bound plastic shear strength along the *x*-axis, kips (N)	C3.4b.3.b
V_{pLy}	Lower-bound plastic shear strength along the *y*-axis, kips (N)	C3.4b.3.b
V_{PZ}	Panel-zone shear, kips (N)	C4.4a.1
V_{PZ}	Panel-zone shear at the development of a hinge (expected first yield) at the critical location of the connection, kips (N)	C5.4a.1.a.1
V_{UDx}	Shear along the *x*-axis determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)	C3.4a.3.a
V_{UDy}	Shear along the *y*-axis determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)	C3.4a.3.a

Symbol	Definition	Section
V_{ye}	Expected shear yield strength, kips (N)	C4.3a
Z_b	Plastic section modulus of beam, in.^{3} (mm^3)	C5.3a.1
a	Modeling parameter shown in Figure C1.1	C2.4a.1.b
a	Clear width of wall between vertical boundary elements, in. (mm)	C6.2c
a_0	Dimension of smaller flange or web thickness that is not welded, including any applicable loss, in. (mm)	C5.3b.3.b
b	Modeling parameter shown in Figure C1.1	C2.4a.1.b
b_a	Distance from the exterior flange face to the resultant tensile force of the bolt or rivet group, as shown in Figure C5.1, in. (mm)	C5.3a.2(a)(iii)
b_{bf}	Width of beam flange, in. (mm)	C5.4a.1.a.1
b_{cf}	Width of the column flange, in. (mm)	C4.4a.2
b_f	Width of flange, in. (mm)	Table C3.6
b_t	Distance between the nearest row of fasteners in the flange of the split-tee and the centerline of the split-tee stem, as shown in Figure C5.2, in. (mm)	C5.3a.2(b)(iv)
c	Modeling parameter shown in Figure C1.1	C2.4a.1.b
d	Modeling parameter shown in Figure C3.1	C3.4a.1.b
d	Full nominal depth of member, in. (mm)	Table C3.6
d	Modeling parameter shown in Figure C1.1	G1.4a.2
d_b	Depth of beam, in. (mm)	C2.4a.1.b
d_b	Smallest depth of the connecting beams at a panel zone, in. (mm)	C4.4a.2
d_{bg}	Depth of bolt group, in. (mm)	Table C5.6
d_c	Depth of column, in. (mm)	C5.4a.1.a.1(b)
d_{max}	Larger of d_1 and d_2, in. (mm)	Table C5.7
d_1	Vertical distance from center of bolt group to top of beam, in. (mm)	Table C5.7
d_2	Vertical distance from center of bolt group to bottom of beam, in. (mm)	Table C5.7
e	Modeling parameter shown in Figure C1.1	G1.4a.2
f	Modeling parameter shown in Figure C3.1	C3.4a.1.b
f	Resistance immediately prior to fracture	Table C7.1
f_{tCE}	Tensile strength of the weld group, kip/in. (N/mm)	C7.4a
f_{tUD}	Yield strength of gusset plate, kip/in. (N/mm)	C7.4a
g	Acceleration of gravity = 32.2 ft/s^2 (9.81 m/s^2)	A4.2a
g	Gap distance between the end of beam and face of column, in. (mm)	Table C5.7
h	For rolled shapes, the clear distance between flanges less the fillet or corner radii; for built-up welded sections, the clear distance between flanges; for built-up bolted sections, the distance between fastener lines; and for tees, the overall depth, in. (mm)	Table C3.6
h	Clear height of wall between horizontal boundary elements, in. (mm)	C6.2c
h_{avg}	Average story height of columns above and below panel zone, in. (mm)	C5.4a.1.a.1
m	Component capacity modification factor	B2.4a
m_e	Effective component capacity modification factor due to lateral-torsional buckling	C2.4a.1.a
m_t	Component capacity modification factor, m, for column or brace in axial tension taken from Table C3.1 or C3.2	C3.4a.2.a.2

Symbol	Definition	Section
m_x	Component capacity modification factor, *m*, for column flexure about the *x*-axis at P_{UF} in accordance with Table C3.5	C3.4a.2.a.1
m_x	Component capacity modification factor, *m*, for shear along the *x*-axis at P_{UF} in accordance with Table C3.7	C3.4a.3.a
m_y	Component capacity modification factor, *m*, for column flexure about the *y*-axis at P_{UF} in accordance with Table C3.5	C3.4a.2.a.1
m_y	Component capacity modification factor, *m*, for shear along the *y*-axis at P_{UF} in accordance with Table C3.7	C3.4a.3.a
n_p	Modification factor for connection robustness	C7.4a.1
r	Radius of gyration, in. (mm)	C3.3a.1
r_y	Radius of gyration about *y*-axis, in. (mm)	Table C3.6
t	Thickness of continuity plate, in. (mm)	C5.4a.1.a.1
t_a	Thickness of angle, in. (mm)	C5.3a.2(a)(ii)
t_{bf}	Thickness of beam flange, in. (mm)	C5.4a.1.a.1
t_{cf}	Thickness of column flange, in. (mm)	C4.4a.2
t_f	Thickness of flange, in. (mm)	Table C3.6
t_f	Thickness of flange of the split-tee, in. (mm)	C5.3a.2(b)(iv)
$t_{f,u}$	Thickness of the smaller flange or web, in. (mm)	C5.3b.3.b
t_p	Total thickness of panel zone, including doubler plates, in. (mm)	C4.3a
t_p	Thickness of flange plate, in. (mm)	C5.3a.2(c)
t_p	Thickness of gusset plate, in. (mm)	C7.2b
t_s	Thickness of split-tee stem, in. (mm)	C5.3a.2(b)(ii)
t_w	Thickness of web, in. (mm)	C2.4a.1.b
t_w	Thickness of column web, in. (mm)	C4.3a
t_w	Thickness of steel plate shear wall, in. (mm)	C6.2c
w	Length of flange angle, in. (mm)	C5.3a.2(a)(iv)
w	Length of split-tee, in. (mm)	C5.3a.2(b)(iv)
Δ_C	Axial deformation at expected compressive buckling strength, in. (mm)	C3.4a.1.b
Δ_p	Plastic axial deformation, in. (mm)	C3.4a.1.b
Δ_T	Axial deformation at expected tensile yield strength, in. (mm)	C3.4a.1.b
Δ_y	Yield axial deformation, in. (mm)	C3.4a.1.b
α	ASD/LRFD force level adjustment factor, specified in the *Specification*	C3.2b
α_h	Post-elastic slope	C2.4a.1.b
α_s	LRFD-ASD force level adjustment factor, specified in the *Seismic Provisions*	Table C2.1
β	Compression strength adjustment factor	C3.3a.1
γ	Total shear deformation, rad	G1.4b.2
γ_i	Initial shear deformation, rad	G3.4a.2
γ_p	Plastic shear deformation, rad	C2.4a.2.b
$\gamma_{p,pz}$	Permissible plastic shear deformation of the panel zone, rad	C4.4a.2
$\gamma_{u,pz}$	Permissible ultimate shear deformation of the panel zone, rad	C4.4a.2
γ_y	Yield shear deformation, rad	C2.4a.1.b
θ_{gp}	Welded gusset plate rotation capacity, rad	C7.4a
θ_p	Plastic chord rotation, rad	C2.4a.1.b
θ_p	Plastic chord rotation demand, rad	C3.4a.2.b

Symbol	Definition	Section
θ_p	Plastic rotation angle, rad	C5.4a.1.a.2
θ_y	Yield chord rotation, rad	C2.4a.1.b
κ	Knowledge factor	A4.1
λ	Width-to-thickness ratio for the element as defined in the *Seismic Provisions*	Table C2.1
λ_{hd}, λ_{md}	Limiting slenderness parameter for highly and moderately ductile compression elements	Table C2.1
σ_{cr}	Lower-bound tensile strength of splices made with partial-joint-penetration groove welds, ksi (MPa)	C5.3b.3.b
σ_{UF}	Weld stress demand on the splice, ksi (MPa)	C5.3b.3.b
τ_b	Stiffness reduction parameter as given in *Specification* Section C2.3, except that P_{ye} shall be substituted for P_{ns} and P_{UF} shall be substituted for αP_r	C3.2b
ω	Strain-hardening adjustment factor	C3.3a.1

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.

Action. An internal moment, shear, torque, axial force, deformation, displacement, or rotation corresponding to a behavior caused by a structural degree of freedom; designated as force- or deformation-controlled.

Applicable building code. Building code under which the structure is evaluated or retrofitted.

Assembly. Two or more interconnected components.

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

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

BSE-1N. Basic Safety Earthquake-1 for use with the Basic Performance Objective Equivalent to New Building Standards, taken as two-thirds of the BSE-2N at a site, as defined in ASCE/SEI 41.

BSE-2N. Basic Safety Earthquake-2 for use with the Basic Performance Objective Equivalent to New Building Standards, taken as the ground shaking based on the Risk-Targeted Maximum Considered Earthquake (MCE_R) at a site, as defined in ASCE/SEI 41.

Buckling brace. A brace that is permitted to buckle under seismic load.

Buckling-restrained brace (BRB). A pre-fabricated, or manufactured, brace element consisting of a steel core and a buckling-restraining system as described in AISC *Seismic Provisions for Structural Steel Buildings* Section F4 and qualified by testing as required in AISC *Seismic Provisions for Structural Steel Buildings* Section K3.

Buckling-restrained braced frame (BRBF). A diagonally braced frame employing buckling-restrained braces and meeting the requirements of AISC *Seismic Provisions for Structural Steel Buildings* Section F4.

Capacity. The permissible strength or permissible deformation for a component action.

Cast iron. A hard, brittle, nonmalleable iron and carbon alloy containing 2.0 to 4.5% carbon. Shapes are obtained by reducing iron ore in a blast furnace, forming it into bars (or pigs), and remelting and casting it into its final form.

Chord. See chords and collectors.

Chord rotation. General measure of deformation of a beam or column between end points in the plane of a frame.

User Note: Two examples of chord rotation are shown in Figure C-C1.1 of the Commentary.

Chords and collectors. Diaphragm members resisting axial forces as part of a complete load path between the diaphragm mass and the lateral-load resisting frame or wall (or between offset lateral-load resisting frames and walls). Collectors are generally aligned with the lateral-load resisting frames and walls, and chords are generally perpendicular to lateral-load resisting frames and walls in buildings with orthogonal layouts.

Collector. See chords and collectors.

Component. A part of an architectural, mechanical, electrical, or structural system of a building.

Concentrically braced frame (CBF). Braced frame element in which component work lines intersect at a single point or at multiple points such that the distance between intersecting work lines (or eccentricity) is less than or equal to the width of the smallest component joined at the connection.

Connectors. Screws, bolts, rivets, gusset plates, shear plates, headed studs, and welds used to link components to other components.

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

Deformation-controlled action. An action that has an associated deformation that is allowed to exceed the yield value of the element being evaluated.

Demand. The amount of force or deformation imposed on an element or component.

Diaphragm chord. See chords and collectors.

Diaphragm collector. See chords and collectors.

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

Element. An assembly of structural components that act together in resisting forces, including gravity frames, moment-resisting frames, braced frames, shear walls, and diaphragms.

Evaluation. An approved process or methodology of evaluating a building for a selected Performance Objective.

Expected strength. The mean value of resistance of a component at the deformation level anticipated for a population of similar components, including consideration of the bias in nominal material strength as well as strain-hardening and plastic section development.

Force-controlled action. An action that is not allowed to exceed the permissible strength of the component being evaluated.

Infill. A panel of masonry placed within a steel or concrete frame. Panels separated from the surrounding frame by a gap are termed "isolated infills." Panels that are in full contact with a frame around its full perimeter are termed "shear infills."

Knowledge factor. Factor used to reduce component strength based on the level of knowledge obtained for individual component during data collection. Refer to ASCE/SEI 41, Section 6.2.4.

Linear dynamic procedure. A Tier 2 or Tier 3 response-spectrum-based modal analysis procedure, the use of which is required where the distribution of lateral forces is expected to depart from that assumed for the linear static procedure.

Linear static procedure. A Tier 2 or Tier 3 lateral force analysis procedure using a pseudo lateral force. This procedure is used for buildings for which the linear dynamic procedure is not required.

Link beam. A component between points of eccentrically connected members in an eccentrically braced frame element.

Liquefaction. An earthquake-induced process in which saturated, loose, granular soils lose shear strength and liquefy as a result of increase in pore-water pressure during earthquake shaking.

Load path. A path through which seismic forces are delivered from the point at which inertial forces are generated in the structure to the foundation and, ultimately, the supporting soil.

Lower-bound strength. The mean minus one standard deviation of the yield strengths, Q_y, for a population of similar components.

Masonry. The assemblage of masonry units, mortar, and possibly grout or reinforcement; classified with respect to the type of masonry unit, including clay-unit masonry, concrete masonry, or hollow-clay tile masonry.

Nonstructural component. An architectural, mechanical, or electrical component of a building that is permanently installed in, or is an integral part of, a building system.

Out-of-plane wall. A wall that resists lateral forces applied normal to its plane.

Performance Objective. One or more pairings of a selected seismic hazard level with an acceptable or desired Structural Performance Level.

Permissible performance parameters. Limiting values of properties, such as drift, strength demand, and inelastic deformation, used to determine the acceptability of a component at a given Performance Level.

Primary component. An element that is required to resist the seismic forces and accommodate seismic deformations for the structure to achieve the selected performance level.

Profiled steel panel. Steel plate that is formed from a steel coil into a fluted profile with top and bottom flanges connected by web members.

Reinforced masonry. Masonry with the following minimum amounts of vertical and horizontal reinforcement: vertical reinforcement of at least 0.20 in.^{2} (130 mm^2) in cross section at each corner or end, at each side of each opening, and at a maximum spacing of 4 ft (1.2 m) throughout; horizontal reinforcement of at least 0.20 in.^{2} (130 mm^2) in cross section at the top of the wall, at the top and bottom of wall openings, at structurally connected roof and floor openings, and at a maximum spacing of 10 ft (3 m).

Required Resistance. The capacity of a structure, component, or connection to resist the effects of loads.

Retrofit. Improving the seismic performance of structural or nonstructural components of a building.

Retrofit measures. Modifications to existing components, or installation of new components, that correct deficiencies identified in a seismic evaluation as part of a scheme to rehabilitate a building to achieve a selected Performance Objective.

Rigid diaphragm. A diaphragm with horizontal deformation along its length less than half the average story drift.

Row of fasteners. Two or more fasteners aligned with the direction of load.

Secondary component. An element that accommodates seismic deformations but is not required to resist the seismic forces it may attract for the structure to achieve the selected performance level.

Story. The portion of a structure between the tops of two successive finished floor surfaces and, for the topmost story, from the top of the floor finish to the top of the roof structural element.

Strength. The maximum axial force, shear force, or moment that can be resisted by a component.

Structural component. A component of a building that provides gravity- or lateral-load resistance as part of a continuous load path to the foundation, including beams, columns, slabs, braces, walls, wall piers, coupling beams, and connections; designated as primary or secondary.

Structural Performance Level. A limit state; used in the definition of Performance Objective.

Subassembly. A portion of an assembly.

Wrought iron. An easily welded or forged iron containing little or no carbon; initially malleable, it hardens quickly when rapidly cooled.

ABBREVIATIONS

The following abbreviations appear within these Provisions. 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)

BRB (buckling-restrained brace)

BRBF (buckling-restrained braced frame)

CBF (concentrically braced frame)

CJP (complete joint penetration)

CP (collapse prevention)

CVN (Charpy V-notch)

EBF (eccentrically braced frame)

FR (fully restrained)

HSS (hollow structural section)

IEBC (International Existing Building Code)

IO (immediate occupancy)

IWUF-B (improved welded unreinforced flange—bolted web)

LAST (lowest anticipated service temperature)

LRFD (load and resistance factor design)

LS (life safety)

PR (partially restrained)

SEI (Structural Engineering Institute)

WPS (welding procedure specification)

WUF (welded unreinforced flange)

WUF-W (welded unreinforced flange—welded web)

CHAPTER A
GENERAL PROVISIONS

This chapter states the scope of the Provisions, summarizes referenced specifications, code and standard documents, general requirements, and provides requirements for condition assessment, material properties, and subassembly tests.

This chapter is organized as follows:

A1. Scope
A2. Referenced Specifications, Codes, and Standards
A3. General Requirements
A4. Document Review and Condition Assessment
A5. Material Properties
A6. Subassembly Tests

A1. SCOPE

The *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings* (ANSI/AISC 342), hereafter referred to as these Provisions, shall govern the seismic evaluation and retrofit of structural steel, composite steel-concrete, wrought iron, and cast iron components of existing buildings subjected to seismic forces and deformations. The requirements of these Provisions shall apply to existing components of a building system, retrofitted components of a building system, and new components added to an existing building system.

ASCE *Seismic Evaluation and Retrofit of Existing Buildings*, hereafter referred to as ASCE/SEI 41, shall be used to compute the force and deformation demands on all primary and secondary structural steel, composite, wrought iron, and cast iron components.

Existing and new components shall be evaluated in accordance with the requirements in these Provisions. Where required by ASCE/SEI 41, these Provisions are intended to be used in conjunction with the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341), hereafter referred to as the *Seismic Provisions*. The strength of existing and new components shall be determined by considering the applicable provisions of Chapters B through K of the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360), hereafter referred to as the *Specification*.

These Provisions include the Symbols, the Glossary, Abbreviations, and Chapters A through I. The Commentary to these Provisions and the User Notes interspersed throughout are not part of these Provisions. The phrases "is permitted" and "are permitted" in this document identify provisions that comply with these Provisions, but are not mandatory.

User Note: The *Specification* sets forth the overarching procedures to determine the strength of structural steel members and their connections, which are collectively called components in these Provisions. There are specific instances in these Provisions where an alternate formulation for strength is specified. In such cases, the alternate provides for lower strength than would be obtained from the *Specification* for the specific action or condition being referenced.

A2. REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

The following specifications, codes, and standards are referenced in these Provisions:

(a) American Concrete Institute (ACI)

ACI 318-19 *Building Code Requirements for Structural Concrete and Commentary*
ACI 318M-19 *Metric Building Code Requirements for Structural Concrete and Commentary*

(b) American Institute of Steel Construction (AISC)

ANSI/AISC 360-22 *Specification for Structural Steel Buildings*
ANSI/AISC 341-22 *Seismic Provisions for Structural Steel Buildings*
ANSI/AISC 358-22 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*

(c) American Institute of Steel Construction (AISC)—Past Versions of Standards

AISC *Seismic Provisions for Structural Steel Buildings (1997) Supplement No. 2 (2000)*
ANSI/AISC 341-05 *Seismic Provisions for Structural Steel Buildings*
ANSI/AISC 341-10 *Seismic Provisions for Structural Steel Buildings*
ANSI/AISC 341-16 *Seismic Provisions for Structural Steel Buildings*

(d) American Iron and Steel Institute (AISI)

ANSI/AISI S310-20 *North American Standard for the Design of Profiled Steel Diaphragm Panels*

(e) American Society of Civil Engineers (ASCE)

ASCE/SEI 41-17 *Seismic Evaluation and Retrofit of Existing Buildings*

(f) ASTM International (ASTM)

ASTM A6/A6M-19 *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*

(g) ASTM International (ASTM)—Withdrawn and Superseded Standards

ASTM A7 (1939–1967) *Specification for Steel for Bridges and Buildings*
ASTM A36 (1960–1993) *Standard Specification for Carbon Structural Steel*
ASTM A373 (1958–1966) *Specification for Structural Steel for Welding*
ASTM A441 (1963–1988) *Specification for High-Strength Low Alloy Structural Manganese Vanadium Steel*

> **User Note:** Because of these Provisions' unique application to existing buildings, the requirements herein cite ASTM standard specifications that have been withdrawn, which means that the standard specification is considered obsolete and is no longer maintained by ASTM. Availability of withdrawn standard specifications may be limited. The Commentary provides information regarding alternative sources for information specified in the withdrawn standard specifications of interest to users of these Provisions.

(h) American Welding Society (AWS)

AWS D1.1/D1.1M-2020 *Structural Welding Code—Steel*
AWS D1.8/D1.8M-2021 *Structural Welding Code—Seismic Supplement*

(i) Steel Deck Institute (SDI)

ANSI/SDI C-2017 *Standard for Composite Steel Floor Deck-Slabs*
ANSI/SDI NC-2017 *Standard for Non-Composite Steel Floor Deck*

A3. GENERAL REQUIREMENTS

A document review and condition assessment shall be conducted in accordance with Section A4.

Material properties of existing components shall be determined in accordance with Section A5.

Testing of subassemblies of components shall be in accordance with Section A6.

General analysis and design requirements for components shall be in accordance with Chapter B.

A4. DOCUMENT REVIEW AND CONDITION ASSESSMENT

1. General

A condition assessment of the existing structure shall be performed as specified in this section and in ASCE/SEI 41, Section 3.2 and Section 6.2, where applicable.

> **User Note:** ASCE/SEI 41, Section 3.2 and Section 6.2, provide requirements for the condition assessment that are in addition to the requirements given in these Provisions.

The condition assessment shall include a review of available construction documents to identify the gravity and lateral load-carrying systems, components of these systems, and any modifications to these systems, their components, and the overall configuration of the structure. Where the available construction documents fail to provide adequate information to identify these aspects of the structure, field survey drawings shall be prepared as required by the data collection requirements of ASCE/SEI 41, Section 6.2. In the absence of construction documents, or where available construction documents do not provide the required connection information, assessment of connections shall be conducted in accordance with Section A4.3.

> **User Note:** ASCE/SEI 41, Section 6.2, indicates that construction documents of interest include design drawings, specifications, material test records, and quality assurance reports covering original construction and subsequent modifications to the structure.

A condition assessment shall include the following:

(a) Examination of the physical condition of representative components and documentation of the presence of any degradation

(b) Verification of the presence and configuration of representative components, and the continuity of load paths among representative components of the systems

(c) Identification and documentation of other conditions, including neighboring party walls and buildings, the presence of nonstructural components that influence building performance, and prior structural modification

(d) Visual inspection of representative structural components involved in seismic force resistance to verify information shown on available documents

(e) Collection of information needed to obtain representative component properties in accordance with Section A4.4

(f) Collection of information needed to develop the analytical model in accordance with Section B1.1

(g) Collection of information needed to select a knowledge factor, κ, in accordance with Section B1.2

> **User Note:** If coverings or other obstructions exist that prevent visual access to a component, a partial visual inspection may be performed through the use of drilled holes and a fiberscope, or a complete visual inspection may be performed by removal of covering materials.

In addition to the requirements of this section, visual or comprehensive condition assessments shall be performed in accordance with Sections A4.2 or A4.3, respectively, where required by the data collection requirements of ASCE/SEI 41, Section 6.2. Components shall be characterized in accordance with Section A4.4.

2. Visual Condition Assessment

If available construction documents specify the details of the connections, at least one connection of each connection type and a portion of each connected component shall be exposed and visually inspected. Bolt heads shall be examined for grade marks and the grade recorded, where grade marks are found. If no deviations from the available drawings exist, the inspected connection is permitted to be considered as representative. If the inspected connection deviates from the available drawings, visual inspection of additional connections of that connection type and its connected components shall be performed until the extent of deviations is determined.

> **User Note:** Connections of the same connection type are characterized by similar limit states and similar modes of nonlinear behavior. Refer to the Commentary for additional information.

2a. Buildings Previously Subjected to Ground Shaking

Existing buildings that have been subjected to ground shaking with a peak acceleration of 0.2*g* or greater, where *g* is the acceleration of gravity equal to 32.2 ft/s^2 (9.81 m/s^2), shall be inspected by a registered design professional to determine the extent of damage to existing components. Inspection protocols shall use a visual inspection approach to identify damage that significantly reduces the seismic force resistance of the structural system. Inspection for damage due to past ground shaking need not be performed where documentation exists indicating that such an inspection was previously performed by a registered design professional after the ground shaking occurred, and the documentation identifies what damage was discovered and any subsequent repair actions that were taken.

3. Comprehensive Condition Assessment

In the absence of construction documents, or where available construction documents do not provide the required connection information, at least three connections of each connection type for primary structural components shall be identified and each identified connection and its connected component shall be exposed and visually inspected. Bolt heads shall be examined for grade marks and the grade recorded, where grade marks are found. If no deviations within a connection type group are observed, the inspected connections shall be considered as representative of that connection type. If deviations within a connection type group are observed, additional connections of the same connection type and their connected components shall be visually inspected until the extent of deviations is determined.

The requirements of Section A4.2a are also applicable to a comprehensive condition assessment.

4. Component Properties

The following characteristics of representative components shall be obtained:

(a) Size and thickness of connecting materials, including cover plates, bracing, and stiffeners

(b) Cross-sectional area, section moduli, moment of inertia, and torsional properties

(c) As-built configuration of connections

(d) Current physical condition of base metal and connector materials, including presence of deformation and extent of deterioration

In the absence of deterioration of a component, use of documented geometric properties of components, connecting elements, and fasteners is permitted.

User Note: Documented geometric properties of components and fasteners can be found in publications by AISC, AISI, ASTM, materials manufacturers, and trade associations.

A5. MATERIAL PROPERTIES

1. General

Material properties shall be based on available construction documents, test reports, manufacturers' data, and as-built conditions as required by these Provisions and as specified in ASCE/SEI 41, Section 3.2. Where such documentation fails to provide adequate information to quantify material properties or capacities of assemblies, such documentation shall be supplemented by sampling and testing of in-place materials, mock-up tests of assemblies, and assessments of existing conditions, as required by these Provisions and as specified in ASCE/SEI 41, Section 6.2.

User Note: Material properties typically of interest include properties related to yield stress, tensile strength, elongation, and notch toughness.

Where permitted by ASCE/SEI 41, Section 6.2, default material properties shall be determined in accordance with Section A5.2. Where default material properties cannot be determined in accordance with Section A5.2, or where materials testing is required by these Provisions or by ASCE/SEI 41, Section 6.2, testing to quantify properties of in-place material shall be in accordance with Section A5.3 and extent of testing shall comply with the requirements of Section A5.4.

User Note: These Provisions include requirements for both material properties determined by sampling and testing, and default values for material properties that may be used without the need for testing. Default values for material properties determined in accordance with Section A5.2 may be used only where permitted by these Provisions or ASCE/SEI 41. Otherwise, material properties are to be determined by sampling and testing of in-place materials, and subsequent analysis of the test results, in accordance with Sections A5.3 and A5.4 and ASCE/SEI 41.

User Note: The predecessor documents to these Provisions provided various requirements for determination of lower-bound yield stress and tensile strength, sometimes using specified minimum values and at times using values greater than specified minimum based on a rule-of-thumb approach for analysis of materials test data. These Provisions resolve such differences by always establishing lower-bound values for yield stress and tensile strength as specified minimum values as determined from information found in available construction documents, or as equivalent specified minimum values derived from reliability-based statistical analysis of materials test data. Refer to the Commentary and to User Notes in Section A5.3b for additional information.

TABLE A5.1
Default Properties for Steel Materials from 1901 and After

Listing in Construction Documents	Date of Standard Specification	F_y and F_u, ksi (MPa)	Default F_{yL} and Default F_{uL}, ksi (MPa)	Default F_{ye} and Default F_{ue}, ksi (MPa)
Standard specification is listed	1901 to 1960	F_y and F_u are obtained from the standard specification	$F_{yL} = 1.0F_y$ $F_{uL} = 1.0F_u$	$F_{ye} = 1.1F_y$ $F_{ue} = 1.0F_u$
Standard specification is listed	1961 and after	F_y and F_u are obtained from the standard specification	$F_{yL} = 1.0F_y$ $F_{uL} = 1.0F_u$	$F_{ye} = 1.1F_y$ $F_{ue} = 1.1F_u$
F_y and F_u are listed, but no standard specification is listed	Not applicable	F_y and F_u are as listed in construction documents	$F_{yL} = 1.0F_y$ $F_{uL} = 1.0F_u$	$F_{ye} = 1.1F_y$ $F_{ue} = 1.1F_u$
No information listed	Not applicable	Default values not provided		

User Note: The approach of using specified minimum material strengths as lower-bound material strengths provides appropriate lower-bound component strengths where the component strength is highly correlated with material strength.

The material properties of steel reinforcement and concrete in composite members shall be determined in accordance with the requirements of ASCE/SEI 41, Section 10.2.

2. Default Material Properties

2a. Structural Steel Materials from 1901 and After

Default lower-bound material properties, F_{yL} and F_{uL}, and expected material properties, F_{ye} and F_{ue}, shall be determined in accordance with Table A5.1, where F_{yL} is the lower-bound yield stress, F_{uL} is the lower-bound tensile strength, F_{ye} is the expected yield stress, and F_{ue} is the expected tensile strength.

Exception: Where Table A5.2 includes the standard specification that is listed in the available construction documents, including the applicable date of the listed standard specification, default material properties shall be determined as $F_{yL} = F_y$, $F_{uL} = F_u$, $F_{ye} = R_yF_y$, and $F_{ue} = R_tF_u$, where R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y, and R_t is the ratio of the expected tensile strength to the specified minimum tensile strength, F_u. R_y and R_t are determined from Table A5.2 and F_y and F_u are obtained from the listed standard specification.

User Note: Summaries of various editions of historical standard specifications, including values of F_y and F_u from the standard specifications, are available in the published literature. Refer to the Commentary for further information, including an abridged summary of selected historical standard specifications.

TABLE A5.2
Factors R_y and R_t for Use in Determining Alternative Default Expected Properties for Steel Materials from 1939 and After[a]

Listing in Construction Documents	Date of Standard Specification	Factors R_y and R_t
Standard specification is listed as ASTM A7	1939 to 1960	R_y = 1.15 and R_t = 1.05
Standard specification is listed as ASTM A36	For wide-flange shapes:	
	1961 to 1970	R_y = 1.2 and R_t = 1.15
	1971 to 1980	R_y = 1.3 and R_t = 1.15
	1981 to 1993	R_y = 1.4 and R_t = 1.2
	For plates, bars, and all shapes other than wide-flange:	
	1961 to 1993	R_y = 1.1 and R_t = 1.1
The listed standard specification is also listed in Table I-6-1 of AISC *Seismic Provisions (1997) Supplement No. 2*	1994 to 2000	Use R_y and R_t for the listed specification from Table I-6-1 of the AISC *Seismic Provisions (1997) Supplement No. 2*
The listed standard specification is also listed in Table I-6-1 of ANSI/AISC 341-05	2001 to 2005	Use R_y and R_t for the listed specification from Table I-6-1 of ANSI/AISC 341-05
The listed standard specification is also listed in Table A3.1 of ANSI/AISC 341-10	2006 to 2010	Use R_y and R_t for the listed specification from Table A3.1 of ANSI/AISC 341-10
The listed standard specification is also listed in Table A3.1 of ANSI/AISC 341-16	2011 to 2016	Use R_y and R_t for the listed specification from Table A3.1 of ANSI/AISC 341-16
The listed standard specification is also listed in Table A3.2 of the *Seismic Provisions*	2017 to 2022	Use R_y and R_t for the listed specification from Table A3.2 of the *Seismic Provisions*

[a] If there is no entry in this table corresponding to the standard specification that is listed in the construction documents, or if the applicable date of the standard specification listed in the construction documents is not included in this table, then Table A5.1 shall be used.

For the determination of default material properties in accordance with Table A5.1 and Table A5.2, the applicable date of the standard specification shall be the date as listed in the available construction documents.

Exception: Where the standard specification is listed in the available construction documents without any date, then the date of standard specification is permitted to be taken as the date of the edition of the listed standard specification reasonably anticipated to have been used for production of the in-place structural steel based on the documented date of construction of the building.

TABLE A5.3
Default Specified Minimum Material Strengths for Historical Structural Metals

Year of Construction	Material	Default F_y and F_u, ksi (MPa)	Default F_{yL} and Default F_{uL}, ksi (MPa)	Default F_{ye} and Default F_{ue}, ksi (MPa)
Before 1920	Wrought iron	$F_y = 18$ (125) $F_u = 25$ (170)	$F_{yL} = 1.0F_y$ $F_{uL} = 1.0F_u$	$F_{ye} = 1.1F_y$ $F_{ue} = 1.1F_u$
Before 1901	Pre-standardized structural steel	$F_y = 24$ (165) $F_u = 36$ (250)	$F_{yL} = 1.0F_y$ $F_{uL} = 1.0F_u$	$F_{ye} = 1.1F_y$ $F_{ue} = 1.1F_u$

TABLE A5.4
Default Lower-Bound Tensile Strength for Existing Welds

Listing in Construction Documents	Construction Date or Date Listed on Construction Documents, Whichever Is Older	Default Value
Filler metal listed	Any	The specified minimum tensile strength for the filler metal classification
Filler metal not listed	1980 or later	70 ksi (485 MPa)
	Before 1980	60 ksi (415 MPa)

2b. Structural Steel Materials from Before 1901, Wrought Iron Materials, and Cast Iron Materials

Default specified minimum material properties, F_y and F_u, default lower-bound material properties, F_{yL} and F_{uL}, and default expected material properties, F_{ye} and F_{ue}, for wrought iron and pre-standardized structural steel shall be as shown in Table A5.3.

Default lower-bound compressive strength of cast iron shall be determined in accordance with Chapter I.

> **User Note:** Because the historical gray cast iron addressed by these Provisions lacks reliable tensile resistance, default tensile material properties are not provided. Refer to the Commentary of Chapter I for further information.

2c. Weld Metal

1. Default Lower-Bound Tensile Strength

The default lower-bound tensile strength for weld metal shall be as shown in Table A5.4.

TABLE A5.5
Default Lower-Bound CVN Toughness for Existing Welds

Listing in Construction Documents	Filler Metal Properties	Default Value
Filler metal listed	The filler metal classification has specified CVN toughness requirements.	The specified minimum CVN notch toughness for the filler metal classification
	The filler metal met the requirements of *Structural Welding Code—Seismic Supplement* (AWS D1.8/D1.8M) for a demand critical weld.	40 ft-lb (54 J) at 70°F (21°C)
	The filler metal classification has no specified minimum CVN toughness requirements.	7 ft-lb (9.5 J) at 70°F (21°C)
Filler metal not listed	Any	7 ft-lb (9.5 J) at 70°F (21°C)

TABLE A5.6
Default Tensile Strength for Existing Rivets

Listing in Construction Documents	Default F_u, ksi (MPa)	Default F_{uL}, ksi (MPa)	Default F_{ue}, ksi (MPa)
Standard specification designation and grade of rivet are listed	Use F_u as listed in the standard specification	$1.00F_u$	$1.10F_u$
Minimum tensile strength of the rivet material is listed	Use the listed minimum tensile strength as F_u		
Rivet material not listed	Determined rivet grade in accordance with *Specification* Appendix 5, Section 5.2.6; F_u is then based upon the grade so determined		

2. Default Lower-Bound Charpy V-Notch Toughness

The default lower-bound Charpy V-notch (CVN) toughness for weld metal shall be as shown in Table A5.5.

2d. Rivet Material

The default specified minimum, default lower-bound, and default expected tensile strengths for rivet material shall be determined in accordance with Table A5.6.

2e. Bolt Material

The default specified minimum and default lower-bound tensile strength for bolt material shall be determined in accordance with Table A5.7.

TABLE A5.7
Default Tensile Strength for Existing Bolts

Listing in Construction Documents	Default F_u, ksi (MPa)[a]	Default F_{uL}, ksi (MPa)	Default F_{ue}, ksi (MPa)
Standard specification designation and grade of bolt are listed	Use F_u as listed in the standard specification	$1.00F_u$	Not required
Minimum tensile strength of the bolt material is listed	Use the listed minimum tensile strength as F_u		
Bolt material not listed	Determined bolt grade in accordance with *Specification* Appendix 5, Section 5.2.6; F_u is then based upon the grade so determined		

[a]If a grade mark is observed on the head of the in-place bolt during the condition survey, F_u as determined from the grade of bolt indicated by the observed grade mark shall be used.

3. Testing to Determine Properties of In-Place Materials

Where testing is required by these Provisions or ASCE/SEI 41, the properties of in-place material shall be determined through removal of samples of the in-place material and subsequent laboratory testing of the removed samples. Laboratory testing of samples to determine properties of the in-place material shall be performed in compliance with standards published by ASTM, AISI, or AWS, as applicable, and in accordance with *Specification* Appendix 5. Alternatively, it is permitted to use in-situ testing of in-place materials where the testing and subsequent data analyses are in accordance with standard test methods.

3a. Sampling and Repair of Sampled Locations

Where the decreased section strength caused by sampling becomes lower than the required capacity, the affected component having the lost section shall be temporarily supported and subsequently repaired to restore the required capacity before temporary supports are removed.

User Note: Sampling locations should be carefully selected, with due consideration given to the loss of capacity and the ease of repair. Sampling should take place in regions of components where the decreased section strength caused by the sampling remains higher than the demands in the component at the reduced section to resist forces and deformations. It is strongly advised that sampling should avoid locations where significant inelastic behavior is expected under seismic ground shaking.

Where a weld or a portion of a weld is to be sampled for testing, details regarding weld sample removal shall be defined.

Where repairs are necessary to compensate for removed material, including where a weld sample is removed, details describing the repairs to the sampled component shall be defined. All welds associated with the repair shall be ground smooth. The repair shall be designed to provide equivalent or better strength and deformation capability as compared to the existing condition.

Where a fastener such as a bolt or rivet is removed for testing, a new bolt of the same nominal diameter and of at least the same tensile strength as the removed connector shall be installed and pretensioned at the time of sampling to replace the removed connector.

3b. Interpretation of Test Results

Expected properties of in-place material shall be taken as mean test values.

Lower-bound properties of in-place material shall be taken as an equivalent specified minimum value determined from test values, such that it is 90% confident that 95% of the test values fall above the equivalent specified minimum value, except that where the in-place material is identified in the available construction documents as conforming to a standard specification or where specified minimum values are listed in the available construction documents, lower-bound properties of the in-place material need not be taken as less than the specified minimum properties listed in the standard specification or in the available construction documents, respectively.

> **User Note:** Refer to the Commentary for description of a statistical procedure that may be used to determine an equivalent specified minimum value as specified in this requirement.

> **User Note:** This statistical analysis for lower-bound properties is intended to be applied to test values obtained from tensile tests on samples removed from in-place materials. Results from tests of subassemblies of components are to be reduced in accordance with Section A6.

4. Extent of Testing of In-Place Materials

The extent of testing required to determine properties of the in-place material of steel and wrought iron components shall be in accordance with Sections A5.4a, A5.4b, or A5.4c, as required by the data collection requirements in ASCE/SEI 41, Section 6.2.

Sampling of cast iron is not required.

> **User Note:** It is inadvisable to sample the historical cast iron that falls under the scope of these Provisions. Refer to the Commentary for further information.

4a. Testing Not Required

Materials testing is not required if material properties are specified on the available construction documents that include certified material test reports or certified reports of tests made in accordance with ASTM A6/A6M. The results of material tests obtained from such reports are permitted to be taken as properties of in-place material for both usual testing and comprehensive testing when statistically analyzed in accordance with Section A5.3b.

Where default yield stress and default tensile strength, both lower-bound and expected, for structural steel materials from 1901 and after are established in accordance with Section A5.2a, it is permitted to use the resulting default yield stress and default tensile strength as the yield stress and tensile strength, respectively, of the in-place materials without additional testing for both usual testing and comprehensive testing.

4b. Usual Testing

The minimum number of tests to determine properties of in-place material for usual data collection is based on the following criteria:

(a) In the absence of construction documents defining properties of the in-place material, at least one strength sample from each component type shall be removed from in-place material and subsequently tested to determine yield stress and tensile strength of the in-place material.

(b) In the absence of construction documents defining filler metal classification and welding processes used for existing welds, default values for weld metal strength are permitted to be used as the existing weld metal strength, provided that the standard specification used to produce the existing steel is defined in the construction documents and the existing steel is permitted for use with prequalified welding procedure specifications (WPS) in accordance with Table B3.1. Alternatively, at least one sample of existing weld metal for each component type having welded joints shall be obtained for laboratory testing to establish weld metal strength, or weld metal strength is permitted to be determined by hardness testing on existing welds in the structure without removal of weld metal samples.

> **User Note:** Guidance for hardness testing of existing steels is provided in Commentary Section A5.3.

4c. Comprehensive Testing

The minimum number of tests to determine properties of the in-place material for comprehensive data collection is based on the following criteria:

(a) Where available construction documents defining properties of the in-place material are inconclusive, or do not exist, but the date of construction is known and the material used is confirmed to be carbon steel, at least three tensile strength samples or three bolts and rivets, as applicable, shall be randomly removed from each component type and subsequently tested to determine yield stress, where applicable, and tensile strength of the in-place material.

(b) In the absence of construction documents defining properties of the in-place material, at least two tensile strength samples or two bolts and rivets, as applicable, shall be removed from each component type for every four floors or every 200,000 ft^2 (19 000 m^2) and subsequently tested to determine yield stress, where applicable, and tensile strength of the in-place material. If it is determined from testing that more than one material grade exists, additional sampling and testing shall be performed until the extent of each grade in component fabrication has been established.

(c) For historical structural wrought iron or pre-standardized structural steel, at least three tensile strength samples shall be removed for each component type for every four floors or 200,000 ft^2 (19 000 m^2) of construction and subsequently tested to determine tensile properties of the in-place material. If initial tests provide material properties that are consistent with properties given in Table A5.3, further tests shall be required only for every six floors or 300,000 ft^2 (28 000 m^2) of construction. If these tests provide material properties with significant differences, additional tests shall be performed until the extent of different materials is established.

(d) In the absence of construction documents defining filler metal classification and welding processes used for existing welds, default values for weld metal strength are permitted to be used provided that the standard specification used to produce the existing steel is defined in the available construction documents and the existing steel is permitted for use with prequalified WPS in accordance with Table B3.1. Alternatively, at least two samples of each component type having welded joints shall be obtained for laboratory testing. The testing shall determine the weld metal strength and CVN impact toughness. The CVN tests shall be performed at a temperature not greater than the lowest ambient service temperature (LAST) plus 20°F (11°C), but not higher than +70°F (+21°C). In lieu of tensile testing, it is permitted to determine the hardness of the welds in the structure without removal of weld metal samples.

> **User Note:** Guidance for hardness testing of existing steels is provided in Commentary Section A5.3.

For other properties of in-place materials, a minimum of three tests shall be conducted.

The results of any testing of in-place material of structural steel and wrought iron shall be compared to the default lower-bound values in Tables A5.1 and A5.3 for the particular era of building construction, where the standard specification used with Table A5.1 is permitted to be taken as the standard specification representing the commercially dominant grade of structural steel for the applicable era of building construction. The amount of testing shall be doubled if the expected and lower-bound yield stress and tensile strength determined from testing of the in-place material are lower than the default lower-bound values.

> **User Note:** Refer to Commentary Section A5.2 for an abridged summary of selected commercially dominant historical standard specifications.

A6. SUBASSEMBLY TESTS

Physical tests of subassemblies of components, including data reduction and reporting, shall be in accordance with ASCE/SEI 41, Section 7.6.

CHAPTER B

GENERAL REQUIREMENTS OF COMPONENTS

This chapter addresses the required characteristics of components to be used to determine compliance with the selected performance objective. The component characteristics are stiffness, strength, and permissible performance parameters.

Every structural component resisting seismic force or deformations in an existing building is to be evaluated in accordance with ASCE/SEI 41. The level of effort required depends on the Tier procedure selected, as defined in ASCE/SEI 41, Chapter 1, and the associated analysis procedure performed.

The chapter is organized as follows:

B1. General
B2. Component Stiffness, Strength, and Permissible Performance Parameters
B3. Retrofit Measures

B1. GENERAL

1. Basis of the Analytical Model

The results of the condition assessment, as specified in Section A4, shall be used to quantify the following items needed to create an analytical model of the building for structural analysis.

(a) Component section properties and dimensions;

(b) Component configuration and eccentricities;

(c) Interaction of nonstructural components and their involvement in seismic force resistance; and

(d) Presence and effects of alterations to the structural system.

If no damage, alteration, or degradation is observed in the condition assessment, component section properties shall be taken from available construction documents, unless the actual properties are known to be otherwise. If sectional material loss or deterioration has occurred, the loss shall be quantified by direct measurement and section properties shall be reduced accordingly using principles of structural mechanics. All deviations noted between available construction records and as-built conditions shall be accounted for in the structural analysis.

2. Knowledge Factor

The data collected, condition assessment, and materials testing shall be used to determine the knowledge factor, κ.

2a. Structural Steel

The knowledge factor, κ, for computation of the permissible performance parameters for steel components shall be selected in accordance with ASCE/SEI 41, Section 6.2.4.

2b. Cast Iron and Wrought Iron

For computation of cast iron and wrought iron component capacities, a knowledge factor, κ, shall be taken as 0.75.

B2. COMPONENT STIFFNESS, STRENGTH, AND PERMISSIBLE PERFORMANCE PARAMETERS

1. General

The behavior of a component action for a specific system shall be designated as either deformation-controlled or force-controlled in accordance with Chapters D through I.

Use of default material properties from Chapter A of these Provisions to determine component strengths is permitted in accordance with ASCE/SEI 41, Chapter 7.

2. Stiffness Criteria

Component stiffness shall be calculated in accordance with Chapter C and any system-specific requirements set forth in Chapters D through I.

3. Strength Criteria

Component strengths for both existing and new components shall be determined in accordance with the general requirements in ASCE/SEI 41, Section 7.5; this chapter; Chapter C; and any system-specific requirements set forth in Chapters D through I. Unless otherwise required in these Provisions, component strength shall be determined using the provisions for nominal strength provided for in *Specification* Chapters B through K, substituting expected or lower-bound properties as determined using these Provisions for the specified minimum properties of the *Specification*. Where a component is not covered by the *Specification* or these Provisions, component strengths are permitted to be obtained by testing in accordance with ASCE/SEI 41, Section 7.6, or by analysis using accepted principles of structural mechanics, subject to the approval of the authority having jurisdiction.

> **User Note:** When using material properties of these Provisions to determine component strength on the basis of the calculation methods of the *Specification*, the resulting component strength is not factored by a resistance factor (i.e., ϕ) or a safety factor (i.e., Ω) when evaluating the component in these Provisions.

3a. Deformation-Controlled Actions

Strengths for deformation-controlled actions on components shall be classified as expected component strengths, Q_{CE}. Where calculations are used to determine expected component strength, expected material properties, including strain hardening where applicable, shall be used.

3b. Force-Controlled Actions

Strengths for force-controlled actions on components shall be classified as lower-bound component strengths, Q_{CL}. Where calculations are used to determine lower-bound component strength, lower-bound material properties shall be used. Where calculations are used to determine lower-bound component strength, a factor of 0.85 shall be applied to elastic buckling limit states.

> **User Note:** Elastic buckling limit states refer to those that the strength is a function of the member or section slenderness and the modulus of elasticity, but not material yield stress. Some examples include elastic flexural buckling of components in compression, elastic lateral-torsional buckling, and elastic shear buckling.

4. Permissible Performance Parameters

Component permissible performance parameters shall be determined in accordance with the general requirements in ASCE/SEI 41, Section 7.5; this chapter; Chapter C; and any system-specific requirements set forth in Chapters D through I.

> **User Note:** The acceptance criteria in ASCE/SEI 41, Section 7.5, is the verification process that a force or deformation demand on a component action does not exceed the permissible performance parameter for that action for a given performance level. Permissible performance parameters for a component action are given in terms of a permissible strength or permissible deformation, dependent upon the analysis type selected, and represent the capacity of an action in a component for a given performance level.

4a. Deformation-Controlled Actions

For linear analysis procedures, the permissible strength for a deformation-controlled action shall be taken as the expected component strength set forth in Section B2.3a adjusted by a component capacity modification factor, *m*.

For nonlinear analysis procedures, the permissible deformation, determined in Chapter C, for a deformation-controlled action shall be taken as the expected deformation capacity.

4b. Force-Controlled Actions

For linear and nonlinear analysis procedures, the permissible strength for a force-controlled action is taken as the lower-bound component strength set forth in Section B2.3b.

> **User Note:** The force-deformation behavior of a force-controlled action can be modeled in a nonlinear analysis in accordance with ASCE/SEI 41, Section 7.5.1.2.

B3. RETROFIT MEASURES

1. General

Seismic retrofit measures shall satisfy the requirements of these Provisions and the applicable provisions of ASCE/SEI 41.

If replacement of an existing component is selected as the retrofit measure, the new component shall be assessed in accordance with these Provisions and detailed and constructed in accordance with the applicable building code.

2. Welds—General

Where welding to existing structural steel components is required as part of a retrofit, the requirements of Table B3.1 shall apply in addition to the requirements of *Structural Welding Code—Steel* (AWS D1.1/D1.1M), hereafter referred to as AWS D1.1/D1.1M. For welding of components that are comprised entirely of new structural steel, without any welding to existing structural steel, the requirements of AWS D1.1/D1.1M shall apply.

3. Welds Resisting Seismic Forces

All new welds added to resist seismic forces in primary components shall conform to *Seismic Provisions* Section A3.4 and *Specification* Chapter J. Welds added to resist seismic forces in primary components shall be designated as conforming to *Seismic Provisions* Section A3.4a, unless required by these Provisions to be designated as demand critical, in which case they shall be designated as conforming to *Seismic Provisions* Section A3.4b.

> **User Note:** Welds in certain connections as identified in Table C5.1 of these Provisions should be designated as demand critical.

TABLE B3.1
Requirements for New Welds to Existing Structural Steel Components

Existing Steel Classification	Welding Requirements
The standard specification used to produce the existing steel is identified in the construction documents and is listed in AWS D1.1/D1.1M, Table 5.3.	AWS D1.1/AWS D1.1M requirements shall apply. It is permitted to use prequalified weld procedure specifications (WPS) in accordance with AWS D1.1/D1.1M, clause 5.
The standard specification used to produce the existing steel is identified in the construction documents and is listed in AWS D1.1/D1.1M, Table 6.9.	AWS D1.1/D1.1M requirements shall apply. WPS shall be qualified by testing in accordance with AWS D1.1/D1.1M, clause 6.
The standard specification used to produce the existing steel is identified in the construction documents as ASTM A7, the existing steel was manufactured after 1950, and the maximum thickness of any element of the existing steel component to be welded is equal to or less than 1½ in. (38 mm).	AWS D1.1/D1.1M requirements shall apply. It is permitted to use prequalified WPS in accordance with AWS D1.1/D1.1M, clause 5. The existing steel shall be considered as either a Group I or Group II base metal in accordance with AWS D1.1/D1.1M, clause 5, based on the tensile properties that are specified in the standard specification used to produce the existing steel. Preheat levels shall be 50°F (28°C) higher than the values listed in AWS D1.1/D1.1M, Table 5.8, with a minimum preheat of 100°F (38°C). Sampling and testing of the existing steel are not required.[a]
The standard specification used to produce the existing steel is identified in the construction documents as ASTM A373 or ASTM A441.	
All other steels, including steels where the standard specification used to manufacture the steel is unknown, or the specification used to manufacture the steel is not a standard specification.	Welding requirements shall be established by the Engineer.[b]

[a]**User Note:** ASTM A7 steel of thickness equal to or less than 1½ in. (38 mm), ASTM A373, and ASTM A441 steels are not listed in AWS D1.1/D1.1M, Table 5.3, but are permitted by these Provisions to be welded with prequalified WPS when the specified additional requirements are met.

[b]**User Note:** Commentary Section B3.2 provides guidance for establishing welding requirements.

CHAPTER C

COMPONENT PROPERTIES AND REQUIREMENTS

This chapter addresses the stiffness and strength of steel and composite steel-concrete members and connections subject to seismic forces and deformations. Expected (deformation-controlled) and lower-bound (force-controlled) strengths are given.

There are four analysis procedures detailed in ASCE/SEI 41 as follows:

(a) Linear static procedure

(b) Linear dynamic procedure

(c) Nonlinear static procedure

(d) Nonlinear dynamic procedure

A performance objective is a set of building performance levels, each coupled with a seismic hazard level. Additionally in this chapter, permissible performance parameters (component capacity modification factor and expected deformation capacity) for primary and secondary structural components are given for three Structural Performance Levels, as defined in ASCE/SEI 41, Chapter 2, and for each analysis type (linear and nonlinear), as follows:

(a) Immediate Occupancy (IO)

(b) Life Safety (LS)

(c) Collapse Prevention (CP)

For linear analysis procedures, permissible strengths are given independently for primary and secondary components, as defined in ASCE/SEI 41, Section 7.5. For nonlinear analysis procedures, permissible deformations are applicable for both primary and secondary components. Interpolation of permissible performance parameters to intermediate performance levels not listed in these Provisions, such as Damage Control and Limited Safety, shall be in accordance with ASCE/SEI 41, Chapter 2.

This chapter is organized as follows:

C1. General
C2. Beams
C3. Members Subjected to Axial or Combined Loading
C4. Panel Zones
C5. Beam and Column Connections
C6. Steel Plate Used as Shear Walls
C7. Braced Frame Connections

C1. GENERAL

ASCE/SEI 41 requires that all structural components subject to seismic forces and deformations be modeled such that forces and deformations induced in the components can be estimated. The analysis procedure selected for assessment will necessitate which component characteristics are required in the analytical component model and means to model the component.

For linear analysis procedures, the force-deformation model shall account for all significant sources of deformation that affect the behavior of the structure, either explicitly or implicitly.

User Note: Complete representation of the nonlinear force-deformation behavior is not required for linear analysis. However, approximate secant stiffnesses may be needed to represent the effects of connection flexibility, concrete cracking of composite components, bolt slip, and similar phenomena.

User Note: A significant source of deformation by a component may be included in the analytical model either explicitly, by directly modeling the component with finite elements and springs, or implicitly, by modifying the nominal properties of the finite elements representing adjacent components. For example, the beam in a reduced beam section moment connection can be modeled with independent line elements for a segment at the beam ends or the beam can be modeled prismatically with a single line element having a reduced stiffness.

For nonlinear analysis procedures, when constructing the nonlinear force-deformation model, the force-deformation behavior of a component shall be determined in accordance with ASCE/SEI 41, Section 7.5, with the permissible performance parameters provided in this chapter. Figure C1.1 depicts Type 1 response, as defined in ASCE/SEI 41, Section 7.5, for use with the modeling parameters of these Provisions. Alternatively, this model may be derived from testing or analysis in accordance with ASCE/SEI 41, Section 7.6.

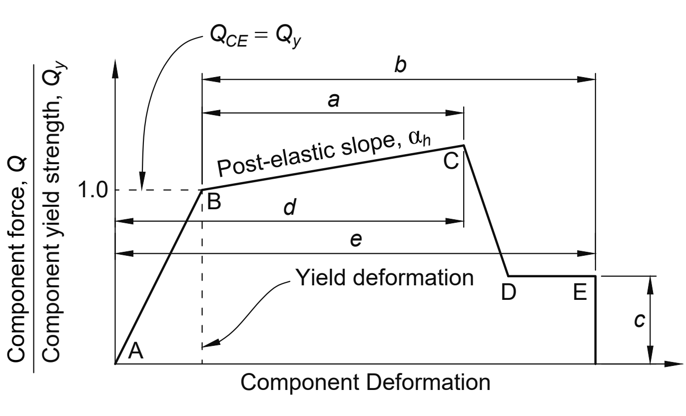


Fig. C1.1. Generalized force-deformation relation for steel components (Type 1 component behavior).

User Note: In the case presented in Figure C1.1, the Provisions use a fully yielded component action to define Point B. Point C represents the peak inelastic strength of the component action and its associated deformation. Point D represents the residual strength of the component action and its associated deformation. Other model types are discussed in ASCE/SEI 41, Section 7.5.

When a component exhibits a total deformation at Point C greater than two times the yield deformation, the post-elastic slope, α_h, is the ratio of the inelastic stiffness to the elastic stiffness, and it can be positive or negative. If α_h is negative, then the peak inelastic strength is less than the yield strength.

C2. BEAMS

1. General

The component characteristics of steel and composite steel-concrete beams subject to seismic forces or deformations from flexural and/or shear actions shall be determined in accordance with this section. This section shall apply to a member when the axial force (compression or tension) in the member does not exceed 10% of the expected compressive strength, P_{CE}, or the expected tensile strength, T_{CE}, whichever applies, determined in accordance with Section C3.3a.1.

The flexural and shear behavior of a beam shall be designated as either deformation-controlled or force-controlled in accordance with Chapters D through I.

If the clear length between supports that resist translation in the direction of the shear force, L_v, is greater than $2.6M_{CE}/V_{CE}$, where M_{CE} is the expected flexural strength and V_{CE} is the expected shear strength, the beam shall be designated as flexure-controlled. If L_v is less than $1.6M_{CE}/V_{CE}$, the beam shall be designated as shear-controlled. For lengths of L_v between $1.6M_{CE}/V_{CE}$ and $2.6M_{CE}/V_{CE}$, the beam shall be designated as shear-flexure-controlled. M_{CE} and V_{CE} shall be determined in accordance with Section C2.3.

Provisions for connections between beams and other structural components are provided in Section C5.

2. Stiffness

The calculation of stiffness of steel beams, either bare or composite with concrete, shall be based on principles of structural mechanics and as specified in the *Specification* unless superseded by supplemental provisions of this section or system-specific sections in Chapters D through I.

The force-deformation model shall account for all significant sources of deformation that affect its behavior, including those from axial, flexural, and shear actions.

2a. Flexural Stiffness

For components encased in concrete, the flexural stiffness shall be determined using full composite action, a cracked section at the onset of yield, and an equivalent width equal to the minimum web width of the concrete section. An effective width of the concrete floor slab, as permitted in *Specification* Section I3.1a, is permitted to be considered if an identifiable shear transfer mechanism between the concrete slab and the steel flange is shown to meet the applicable permissible performance parameters for the selected performance level.

2b. Axial Stiffness

For components fully encased in concrete and where axial tensile forces remain below the cracking limit, the axial stiffness shall be determined using 100% of the steel and 70% of the concrete area, assuming full composite action, if confining reinforcement consisting of at least a No. 3 (10 mm) at 12 in. (300 mm) spacing or a No. 4 (13 mm) at 16 in. (400 mm) spacing is provided, and the spacing of the confining reinforcement is no more than 0.5 times the least encasing dimension. If this confining reinforcement requirement is not satisfied, the axial stiffness shall be determined assuming no composite action is achievable.

Concrete confined on at least three sides, or over 75% of its perimeter, by elements of the steel component shall be permitted to be considered adequately confined to provide full composite action.

2c. Shear Stiffness

For composite beams, the shear stiffness shall be taken as that of the steel section alone, unless otherwise justified by test or analysis.

3. Strength

The flexural and shear strengths of a beam shall be determined in accordance with this section.

3a. Deformation-Controlled Actions

1. Expected Flexural Strength

The expected flexural strength, M_{CE}, shall be determined using equations for nominal flexural strength, M_n, given in *Specification* Chapter F, except that the expected yield stress, F_{ye}, determined in accordance with Chapter A, shall be substituted for the specified minimum yield stress, F_y, and the expected strength, $Q_{CE} = M_{CE}$. For the limit state of shear yielding, M_{CE} shall not be taken greater than $V_{CE}L_v/2$, or as required by analysis based on support conditions, where V_{CE} is determined in accordance with Section C2.3a.2.

For beams expected to experience inelastic action through flexural yielding, the beam shall have adequate compactness or be sufficiently braced laterally to develop the expected plastic flexural strength, M_{pe}, of the section determined using the equation for M_n from *Specification* Chapter F for the limit state of yielding at the yielding locations, except that F_{ye} shall be substituted for F_y. In this case, the expected component strength, $Q_{CE} = Q_y = M_{CE}$, where Q_y is the expected component yield strength; otherwise, $Q_{CE} < Q_y$.

For beams fully encased in concrete where confining reinforcement is provided to ensure that the concrete remains in place during seismic loading, the limit states of local buckling and lateral-torsional buckling need not be considered.

2. Expected Shear Strength

The expected shear strength, V_{CE}, shall be determined using equations for nominal shear strength, V_n, given in *Specification* Chapter G, except that F_{ye} shall be substituted for F_y, and the expected component strength, $Q_{CE} = V_{CE}$.

For beams expected to experience inelastic action through shear yielding, the shear yielding zone shall be sufficiently stiffened or the section shall have adequate compactness to prevent shear buckling before shear yielding to develop the expected plastic shear strength, V_{pe}, determined using equations for nominal shear strength, V_n, given in *Seismic Provisions* Section F3, with F_{ye} substituted for F_y. In this case, $Q_{CE} = Q_y = V_{CE}$; otherwise, $Q_{CE} < Q_y$. Stiffener strength, stiffness, spacing, and web compactness shall be in accordance with the requirements in *Seismic Provisions* Section F3.

3b. Force-Controlled Actions

1. Lower-Bound Flexural Strength

The lower-bound flexural strength, M_{CL}, shall be determined using equations for nominal strength, M_n, given in *Specification* Chapter F, except that the lower-bound yield stress determined in accordance with Chapter A, F_{yL}, shall be substituted for F_y, and the lower-bound component strength, $Q_{CL} = M_{CL}$. For the limit state of shear yielding, M_{CL} shall not be taken greater than $V_{CL}L_v/2$, or as required by analysis based on support conditions, where the lower-bound shear strength, V_{CL}, is determined in accordance with Section C2.3b.2.

2. Lower-Bound Shear Strength

The lower-bound shear strength, V_{CL}, shall be determined using equations for nominal strength, V_n, given in *Specification* Chapter G, except that F_{yL} shall be substituted for F_y, and $Q_{CL} = V_{CL}$.

4. Permissible Performance Parameters

Permissible strengths and deformations for flexural and shear actions in a beam shall be computed in accordance with this section.

4a. Deformation-Controlled Actions

1. Flexural Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the flexural behavior is considered deformation-controlled, the expected flexural strength, $Q_{CE} = M_{CE}$, shall be determined in accordance with Section C2.3a.1 and m shall be taken from Table C2.1. If $M_{CE} < M_{pe}$, then m shall be replaced by the effective component capacity modification factor due to lateral-torsional buckling, me, determined from Equation C2-1:

$$m_e = m - (m-1)\left[\frac{M_{pe} - M_{CE}}{M_{pe} - 0.7F_{ye}S}\right] \geq 1.0 \qquad \text{(C2-1)}$$

TABLE C2.1
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Beams Subjected to Flexure[a] [b]

Section Compactness[c]	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
1. Highly ductile ($\lambda \le \lambda_{hd}$)	2	6	8	10	12
2. Non-moderately ductile ($\lambda \ge \lambda_{md}$)	1.25	2	3	3	4
3. Other	Linear interpolation between the values on lines 1. and 2. for flange, wall, and web slenderness shall be performed, and the lowest resulting value shall be used.				

λ = width-to-thickness ratio for the element as defined in the *Seismic Provisions*.
CP = collapse prevention performance level as defined in ASCE/SEI 41, Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41, Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41, Chapter 2
[a] Regardless of the modifiers applied, *m* need not be taken less than 1.0.
[b] Tabulated values are applicable for flexure-controlled beams with $L_v \ge 2.6M_{CE}/V_{CE}$. Values of *m* shall be 1.0 when $L_v \le 1.6M_{CE}/V_{CE}$. For $1.6M_{CE}/V_{CE} < L_v < 2.6M_{CE}/V_{CE}$, *m* shall be linearly interpolated between the tabulated values and 1.0.
[c] The limiting slenderness parameters for highly and moderately ductile compression elements, λ_{hd} and λ_{md}, respectively, are defined in *Seismic Provisions* Table D1.1, with R_yF_y replaced by F_{ye} and α_sP_r replaced by P_{UF}. λ shall be compared to λ_{hd} and λ_{md} for each element of the cross section and the element producing the lowest value of *m* shall be used, where

P_r = required axial strength using load and resistance factor design (LRFD) or allowable stress design (ASD) load combinations, kips (N)
P_{UF} = axial force (compression or tension) determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)
R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y
α_s = LRFD-ASD force level adjustment factor, specified in the *Seismic Provisions*

where

M_{pe} = expected plastic flexural strength of the section, at the location of the plastic hinge, about the axis of bending defined in Section C2.3a.1, kip-in. (N-mm)
S = elastic section modulus about the axis of bending, in.^{3} (mm^3)

If M_{CE} is limited by the limit state of shear yielding, the beam shall be assessed in accordance with Section C2.4a.2.

For beams fully encased in concrete where confining reinforcement is provided to ensure that the concrete remains in place during seismic loading, the limit states of local buckling and lateral-torsional buckling need not be considered for the purpose of determining the component capacity modification factor, *m*.

For built-up shapes, where the strength is governed by the strength of the lacing plates that carry component shear, *m* shall be taken as 0.5 times the applicable value in Table C2.1, unless larger values are justified by tests or analysis; however, *m* need not be taken as less than 1.0. The adequacy of lacing plates shall be evaluated using the provisions for tension braces in the *Specification*. For built-up laced beams fully encased in concrete, local buckling of the lacing need not be considered where confining reinforcement is provided to allow the encasement to remain in place during an earthquake.

b. Nonlinear Analysis Procedures

When constructing the nonlinear force-deformation model for use in the nonlinear analysis procedures, the generalized force-deformation curve for flexural behavior shown in Figure C1.1, with modeling parameters a, b, and c as given in Table C2.2, shall be used for beams. Alternatively, these relationships may be derived from testing or analysis. For beams, it is permitted to take the post-elastic slope, α_h, for flexural action as 3% of the elastic slope. Further modification of the curve is permitted if a greater value for α_h is justified by testing or analysis.

When the flexural behavior is considered deformation-controlled, the plastic chord rotation, θ_p, predicted by analysis shall be not greater than the permissible plastic chord rotation provided in Table C2.2 for a given performance level. If the beam is flexure-controlled, the yield chord rotation, θ_y, shall be determined from analysis as the rotation at which the computed moment at the location of flexural yielding is M_{CE}. If the beam is flexure-controlled and fully restrained at both ends without consideration of the panel-zone stiffness, and loading is such that the point of inflection under seismic loading is located at the beam midspan, it is permitted to determine θ_y from Equation C2-2. Otherwise, if the beam is shear-controlled or shear-flexure-controlled, θ_y shall be taken as the yield shear deformation, γ_y, determined from Section C2.4a.2.b.

$$\theta_y = \frac{M_{CE} L_{CL}(1+\eta)}{6EI} \tag{C2-2}$$

where

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

I = moment of inertia about the axis of bending, in.^{4} (mm^4)

L_{CL} = length of beam taken between column centerlines, in. (mm)

$$\eta = \frac{12EI}{L_{CL}^2 GA_s} \tag{C2-3}$$

A_s = effective shear area of the cross section, in.^{2} (mm^2)
(for a wide-flange section in strong-axis bending, $A_s = d_b t_w$)

d_b = depth of beam, in. (mm)

t_w = thickness of web, in. (mm)

G = shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)

Where shear deformation in a beam does not change the component deformation by more than 5% or is not included in the analysis of the analytical model, it is permitted to take η as zero.

User Note: Shear deformation (accounted for by η) in a flexure-controlled beam with a length greater than $10M_{CE}/V_{CE}$ is generally small and can be neglected in Equation C2-2.

TABLE C2.2
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Beams Subjected to Flexure[a]

Modeling Parameters		Expected Deformation Capacity		
Plastic Chord Rotation Angle *a* and *b*, rad Residual Strength Ratio *c*		Plastic Chord Rotation Angle, rad		
		IO	LS	CP
Section Compactness[b]				
1. Highly ductile ($\lambda \le \lambda_{hd}$)	$a = 9\theta_y$ $b = 11\theta_y$ $c = 0.6$	0.25a	a	b
2. Non-moderately ductile ($\lambda \ge \lambda_{md}$)	$a = 4\theta_y$ $b = 6\theta_y$ $c = 0.2$	0.25a	0.75a	a
3. Other		Linear interpolation between the values on lines 1. and 2. for flange, wall, and web slenderness shall be performed, and the lowest resulting value shall be used.		

[a]Tabulated values are applicable for flexure-controlled beams with $L_v \ge 2.6M_{CE}/V_{CE}$. Values shall be taken as 0.0 when $L_v \le 1.6M_{CE}/V_{CE}$. For $1.6M_{CE}/V_{CE} < L_v < 2.6M_{CE}/V_{CE}$, values shall be linearly interpolated between the tabulated values and 0.0.
[b]The limiting width-to-thickness ratios, λ_{hd} and λ_{md}, are defined in *Seismic Provisions* Table D1.1, with R_yF_y replaced by F_{ye} and $\alpha_s P_r$ replaced by P_{UF}. λ shall be compared to λ_{hd} and λ_{md} for each element of the cross section and the element producing the lowest permissible deformation shall be used.

If $M_{CE} < M_{pe}$, the values in Table C2.2 shall be multiplied by the factor, Ψ, determined from Equation C2-4:

$$\Psi = 1 - \frac{M_{pe} - M_{CE}}{M_{pe} - 0.7 F_{ye} S} \ge 0 \tag{C2-4}$$

2. Shear Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior is considered deformation-controlled, the expected shear strength, $Q_{CE} = V_{CE}$, shall be determined in accordance with Section C2.3a.2 and m shall be taken from Table C2.3.

b. Nonlinear Analysis Procedures

When constructing the nonlinear force-deformation model for use in the nonlinear analysis procedures, the generalized force-deformation curve for shear behavior shown in Figure C1.1, with modeling parameters a, b, and

TABLE C2.3
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures— Beams Subjected to Shear[a] [b] [c] [d]

Length, L_v	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
$L_v \le \dfrac{1.6M_{CE}}{V_{CE}}$ (shear-controlled)	1.5	9	13	13	15

[a]Values are applicable for shear-controlled beams with three or more web stiffeners. If there are no stiffeners, divide values for shear-controlled beams by 2.0, but values need not be taken less than 1.25. Linear interpolation is permitted for one or two stiffeners.
[b]Assumes ductile detailing for beam in the shear yielding zone in accordance with the *Seismic Provisions*.
[c]Regardless of the modifiers applied, m need not be taken as less than 1.0.
[d]Values of m shall be 1.0 when $L_v \ge 2.6M_{CE}/V_{CE}$. For $1.6M_{CE}/V_{CE} < L_v < 2.6M_{CE}/V_{CE}$, m shall be linearly interpolated between the tabulated values and 1.0.

c as given in Table C2.4, shall be used for beams. Alternatively, these relationships are permitted to be derived from testing or analysis. For beams, it is permitted to take α_h for shear action as 6% of the elastic slope. Further modification of the curve is permitted if a greater value for α_h is justified by testing or analysis.

When the shear behavior is considered deformation-controlled, the plastic shear deformation, γ_p, predicted by analysis shall be not greater than the permissible plastic shear deformation provided in Table C2.4 for a given performance level. The yield shear deformation, γ_y, shall be determined from Equation C2-5:

$$\gamma_y = \frac{V_{CE}}{K_e L_v} \tag{C2-5}$$

where

K_e = elastic shear stiffness, determined in accordance with Section C2.2, kip/in. (N/mm)

L_v = clear length between supports that resist translation in the direction of the shear force, in. (mm)

V_{CE} = expected shear strength of the beam determined in accordance with Section C2.3a.2, kips (N)

4b. Force-Controlled Actions

1. Flexural Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the flexural behavior is considered force-controlled, the lower-bound flexural strength, $Q_{CL} = M_{CL}$, shall be determined in accordance with Section C2.3b.1.

TABLE C2.4
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Beams Subjected to Shear[a] [b] [c] [d]

	Modeling Parameters			Expected Deformation Capacity		
	Plastic Shear Deformation, rad		Residual Strength Ratio	Plastic Shear Deformation, rad		
Length, L_v	*a*	*b*	*c*	IO	LS	CP
$L_v \leq \dfrac{1.6M_{CE}}{V_{CE}}$ (shear-controlled)	0.15	0.17	0.8	0.005	0.14	0.16

[a]Deformation is the rotation angle between the beam and column or portion of beam outside the shear yielding zone.
[b]Values are applicable for shear-controlled beams with three or more web stiffeners. If no stiffeners, divide values for shear-controlled beams by 2.0. Linear interpolation is permitted for one or two stiffeners.
[c]Assumes ductile detailing for beam in the shear yielding zone in accordance with the *Seismic Provisions*.
[d]Values shall be taken as 0.0 when $L_v \geq 2.6M_{CE}/V_{CE}$. For $1.6M_{CE}/V_{CE} < L_v < 2.6M_{CE}/V_{CE}$, values shall be linearly interpolated between the tabulated values and 0.0.

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the flexural behavior is considered force-controlled, the lower-bound flexural strength, $Q_{CL} = M_{CL}$, shall be determined in accordance with Section C2.3b.1.

2. Shear Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior is considered force-controlled, the lower-bound shear strength, $Q_{CL} = V_{CL}$, shall be determined in accordance with Section C2.3b.2.

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the shear behavior is considered force-controlled, the lower-bound shear strength, $Q_{CL} = V_{CL}$, shall be determined in accordance with Section C2.3b.2.

C3. MEMBERS SUBJECTED TO AXIAL OR COMBINED LOADING

1. General

The component characteristics of steel and composite steel-concrete members subjected to seismic forces or deformation from axial action alone, or flexural and/or shear actions with concurrent axial action, shall be determined in accordance with this section. This section shall apply to a member when the axial force (compression or tension) in the member equals or exceeds 10% of P_{CE} or T_{CE}, whichever applies, determined in accordance with Section C3.3a.1.

User Note: Beams with an axial force equal to or exceeding 10% of P_{CE} or T_{CE} should be evaluated in accordance with Section C3. Most beams in braced frames meet this requirement. Beams meeting this criterion are denoted as columns in this section.

The axial, flexural, and shear behavior of a column or brace shall be designated as either deformation-controlled or force-controlled in accordance with Chapters D through I.

If the clear length between supports that resist translation in the direction of shear force, L_v, is greater than $2.6M_{CE}/V_{CE}$, the column or brace shall be designated as flexure-controlled. If L_v is less than $1.6M_{CE}/V_{CE}$, the column or brace shall be designated as shear-controlled. For lengths of L_v between $1.6M_{CE}/V_{CE}$ and $2.6M_{CE}/V_{CE}$, the column or brace shall be designated as shear-flexure-controlled. M_{CE} and V_{CE}, the expected flexural and shear strengths, respectively, shall consider the effect of axial force interaction, determined in accordance with Section C3.3.

Provisions for connections of columns and braces to other structural components are provided in Sections C5 and C7.

Buckling braces shall use the generalized force-deformation relation in Figure C3.1 for both the compressive and tensile response. This relation accounts for the degradation in brace strength with increasing deformation. The parameters shall be computed differently for tensile and compressive brace response as specified in Section C3.4.

2. Stiffness

The stiffness of columns or braces shall be based on principles of structural mechanics and as specified in the *Specification* unless superseded by supplemental provisions of this section or system-specific sections in Chapters D through I.

The force-deformation model for a column or brace shall account for all significant sources of deformation that affect its behavior, including those from axial, flexural, and shear actions.

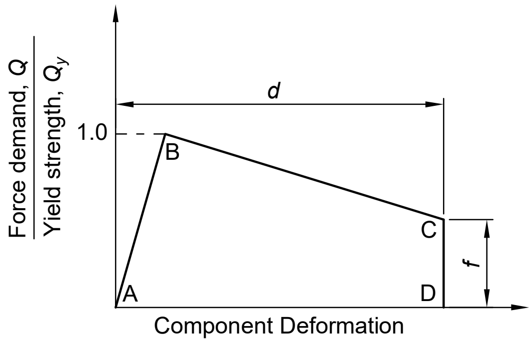


Fig. C3.1. Generalized force-deformation relation for buckling braces and their connections acting together.

2a. Axial Stiffness

Elastic stiffness of a buckling brace shall be calculated or modeled using the end-to-end brace length, L_{ee}. Buckling braces that are filled with concrete shall consider the full composite stiffness of the uncracked concrete in compression if the development of composite action can be justified; otherwise, the brace stiffness shall be based on the steel element only. Concrete fill in buckling braces, which engages the end connections of the brace, shall be evaluated as fully composite members with respect to compressive stiffness and resistance.

For buckling-restrained braces, the axial stiffness shall be modeled with the stiffness of the yielding core segment and transition segment added in series. A transition segment shall include the properties of the brace that is stiffened from the end of the core to the gusset connection. It is permitted to assume the gusset and beam-to-column connection are rigid relative to the brace.

For components fully encased in concrete and where axial tensile forces remain below the cracking limit, the axial stiffness shall be determined using 100% of the steel and 70% of the concrete area, assuming full composite action, if confining reinforcement consisting of at least a No. 3 (10 mm) at 12 in. (300 mm) spacing or a No. 4 (13 mm) at 16 in. (400 mm) spacing is provided, and the spacing of the confining reinforcement is no more than 0.5 times the least encasing dimension. If this confining reinforcement requirement is not satisfied, the axial stiffness shall be determined assuming no composite action is achievable. Concrete confined on at least three sides, or over 75% of its perimeter, by elements of the steel component is permitted to be considered adequately confined to provide full composite action.

2b. Flexural Stiffness

The flexural stiffness of a column or brace, EI_c, shall be modified by the stiffness reduction parameter, τ_b, as given in *Specification* Section C2.3, except that P_{ye} shall be substituted for P_{ns} and P_{UF} shall be substituted for αP_r,

where

I_c = moment of inertia of a column or brace about the axis of bending, in.^{4} (mm^4)

P_{ns} = cross-section compressive strength, kips (N)

P_r = required axial compressive strength using load and resistance factor design (LRFD) or allowable strength design (ASD) load combinations, kips (N)

P_{UF} = axial compressive force determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)

P_{ye} = $A_g F_{ye}$ = expected axial yield strength, kips (N)

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

α = ASD/LRFD force level adjustment factor, specified in the *Specification*

For nonlinear analysis of buckling braces, the flexural stiffness shall be modeled using the requirements of Section E1.2b.

The flexural stiffness of column or brace encased in concrete shall satisfy the requirements in Section C2.2a.

2c. Shear Stiffness

For composite members, the shear stiffness shall be taken as that of the steel section alone, unless otherwise justified by rational analysis.

3. Strength

The axial, flexural, and shear strengths of a column or brace shall be computed in accordance with this section.

3a. Deformation-Controlled Actions

1. Expected Axial Strength

The expected compressive strength, P_{CE}, of a steel column or brace, or a concrete-filled brace in which the concrete does not engage the brace end connections, shall be determined using equations for nominal compressive strength, P_n, given in *Specification* Chapter E, except that F_{ye} shall be substituted for F_y.

For buckling braces, the effective length, L_c, for calculation of member slenderness, L_c/r, shall be determined using the end-to-end brace length, L_{ee},

where

K = effective length factor

$L_c = KL_{ee}$ = effective length, in. (mm)

r = radius of gyration, in. (mm)

The expected tensile strength, T_{CE}, shall be determined using equations for nominal axial strength, P_n, given in *Specification* Chapter D, except that F_{ye} shall be substituted for F_y, and the expected tensile strength, F_{ue}, shall be substituted for the specified minimum tensile strength, F_u.

The expected compressive and tensile strength for a buckling-restrained brace, P_{CE} and T_{CE}, shall be the net area of the core multiplied by the expected yield stress, F_{ye}. For strength and modeling parameters, F_{ye} shall be taken as the specified minimum yield stress, F_y, multiplied by the ratio of the expected yield stress to the specified minimum yield stress, R_y, from *Seismic Provisions* Table A3.2. Where the yield stress is specified as a range, F_{ye} shall be based on the highest yield stress in the range for the determination of the maximum brace force. If F_{ye} is established by testing, that value shall be used. The buckling-restrained brace (BRB) casing system shall be designed to resist the maximum force that the steel core can develop. The maximum force that the core can develop in compression shall be determined as $\beta\omega P_{CE}$, and the maximum force that can be developed in tension as ωP_{CE}. Factors β and ω are the compression strength adjustment factor and the strain-hardening adjustment factor, respectively, as defined in *Seismic Provisions* Section F4.2. These factors shall be based on qualification testing, as described in the *Seismic Provisions*. Alternatively, for linear analysis, it is permitted to use $\beta = 1.1$ and $\omega = 1.3$ if no testing is available.

2. Expected Flexural Strength

The expected flexural strength, M_{CE}, of a column or brace shall be determined in accordance with Section C2.3a.1.

For columns or braces expected to experience inelastic action through flexural yielding, the column or brace shall have adequate compactness or be sufficiently braced laterally to develop the expected plastic flexural strength of the section at the yielding locations that accounts for the interaction of axial force and biaxial moments, if required. In this case, the expected component strength, $M_{CE} = Q_{CE} = Q_y$; otherwise, $Q_{CE} < Q_y$.

User Note: When buckling braces are modeled with line elements that capture their nonlinear axial force-deformation behavior, including the effects of buckling, it is not necessary to explicitly evaluate their flexural strength. However, where explicit modeling of flexural behavior is performed to capture brace behavior, flexural strength should be evaluated in accordance with these requirements.

3. Expected Shear Strength

The expected shear strength, V_{CE}, of a column or brace shall be determined in accordance with Section C2.3a.2.

For columns or braces expected to experience inelastic action through shear yielding, the shear yielding zone shall be sufficiently stiffened or the section shall have adequate compactness to prevent shear buckling before shear yielding in order to develop the expected plastic shear strength of the section that accounts for the interaction of axial force and biaxial shears, if required. In this case, the expected component strength, $V_{CE} = Q_{CE} = Q_y$; otherwise, $Q_{CE} < Q_y$.

3b. Force-Controlled Actions

1. Lower-Bound Axial Strength

The lower-bound compressive strength, P_{CL}, of a column or brace shall be determined using equations for nominal strength, P_n, given in *Specification* Chapter E, except that F_{yL} shall be substituted for F_y.

The lower-bound tensile strength, T_{CL}, of a column or brace shall be determined using equations for nominal strength, P_n, given in *Specification* Chapter D, except that F_{yL} shall be substituted for F_y.

2. Lower-Bound Flexural Strength

The lower-bound flexural strength, M_{CL}, of a column or brace shall be determined in accordance with Section C2.3b.1.

3. Lower-Bound Shear Strength

The lower-bound shear strength, V_{CL}, of a column or brace shall be determined in accordance with Section C2.3b.2.

4. Permissible Performance Parameters

Permissible strengths and deformations for axial actions, and flexural and shear actions concurrent with axial action, in a column or brace shall be computed in accordance with this section.

4a. Deformation-Controlled Actions

1. Axial Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the axial behavior is considered deformation-controlled, the expected component strength, $Q_{CE} = P_{CE}$ or $Q_{CE} = T_{CE}$, shall be determined in accordance with Section C3.3a.1 and m shall be taken from Table C3.1 or Table C3.2, as appropriate.

b. Nonlinear Analysis Procedures

When constructing the nonlinear force-deformation model for use in the nonlinear analysis procedures, the generalized force-deformation curve for axial behavior shown in Figure C1.1, with modeling parameters a, b, and c as given in Table C3.3, shall be used for columns and buckling-restrained braces. Alternatively, these relationships are permitted to be derived from testing or analysis. For columns and buckling-restrained braces, it is permitted to take α_h for tension action as 3% of the elastic slope. Further modification of the curve is permitted if a greater value for α_h is justified by testing or analysis.

User Note: For a buckling-restrained brace, Point C in Figure C1.1 is ωQ_{CE} for tension and $\beta\omega Q_{CE}$ for compression. Refer to Section C3.3a.1 and the *Seismic Provisions* to determine the compression strength adjustment factor, β, and the strain-hardening adjustment factor, ω.

For nonlinear analysis procedures, the nonlinear force-deformation behavior of buckling braces, as depicted in Figure C3.1, with the modeling parameters d and f as defined in Table C3.4, shall be used. Alternatively, these relationships are permitted to be derived from testing.

When the axial behavior of a column or brace is considered deformation-controlled, the plastic axial deformation, Δ_p, predicted by analysis shall be not greater than the permissible plastic axial deformations provided in Table C3.3 or Table C3.4 for a given performance level. The yield axial deformation, Δ_y, shall be determined as follows:

For tension:
$$\Delta_y = \Delta_T = \frac{T_{CE} L_c}{EA_g} \qquad \text{(C3-1)}$$

For compression:
$$\Delta_y = \Delta_C = \frac{P_{CE} L_c}{EA_g} \qquad \text{(C3-2)}$$

For buckling-restrained braces:
$$\Delta_y = \frac{P_{CE} L_{core}}{EA_{core}} + 2\frac{P_{CE} L_{conn}}{EA_{conn}} \qquad \text{(C3-3)}$$

where

A_{conn} = cross-sectional area of BRB connection, in.^{2} (mm^2)
A_{core} = cross-sectional area of BRB core, in.^{2} (mm^2)
L_{conn} = length of BRB connection, in. (mm)
L_{core} = length of BRB core, in. (mm)

TABLE C3.1
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Columns and Buckling-Restrained Braces Subjected to Axial Force

Component	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
1. Columns in Tension	1.25	3	5	6	7
2. Buckling-Restrained Braces in Tension or Compression[a] [b]	2.3	5.6	7.5	7.5	10

CP = collapse prevention performance level as defined in ASCE/SEI 41, Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41, Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41, Chapter 2
[a]Maximum strain of the BRB core shall not exceed 2.5%.
[b]If testing to demonstrate compliance with Section E3.4a is not available, the values shall be multiplied by 0.7.

Δ_C = axial deformation at expected compressive buckling strength, in. (mm)
Δ_T = axial deformation at expected tensile yield strength, in. (mm)

User Note: The term Q_y, and associated Δ_y in Figures C1.1 and C3.1 refer to Point B in the force-deformation behavior, which is generally termed the "yield point" for a given action. For compressive axial actions for columns and buckling braces, Point B corresponds to buckling behavior rather than traditional yielding in compression. See ASCE/SEI 41, Figure 7-4.

2. Flexural Actions Concurrent with Axial Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the flexural behavior of a column or brace is considered deformation-controlled, the flexural behavior shall be evaluated in accordance with this section. A column or brace shall satisfy both section strength requirements and member strength requirements in accordance with this section.

1. Section Strength

For columns and braces under combined axial load and bending moment, development of a flexural plastic hinge shall be deformation-controlled for flexural behavior, and the combined axial-bending behavior of the section at the plastic hinge location shall be evaluated by Equation C3-4, with values for *m* taken from Table C3.5. Where $P_{UF}/P_{ye} > 0.6$, the component shall remain elastic for flexure.

TABLE C3.2
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Buckling Braces Subjected to Axial Force

Component	IO[f]	Primary Component LS	Primary Component CP	Secondary Component LS	Secondary Component CP
Buckling Braces in Compression[c] [d] [e]					
1. Wide-flange shapes, I-shaped members, double angles in-plane, double channels in-plane[b] $n = 5.6\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_{ye}}}\right)^{0.40}$	1.25	0.5n	0.75n	0.6n	0.9n
2. Double angles out-of-plane, double channels out-of-plane[b] $n = 4.7\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_{ye}}}\right)^{0.45}$	1.25	0.5n	0.75n	0.6n	0.9n
3. Rectangular hollow structural section (HSS)[a] $n = 3.0\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.0}\left(\frac{L_c/r}{\sqrt{E/F_{ye}}}\right)^{1.0}$	1.25	0.5n	0.75n	0.6n	0.9n
4. Round HSS and pipe[a] $n = 4.7\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_{ye}}}\right)^{0.45}$	1.25	0.5n	0.75n	0.6n	0.9n
5. Single angles $n = 12\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}$	1.25	0.5n	0.75n	0.6n	0.9n
Buckling Braces in Tension[c] [d] [e]					
1. Wide-flange shapes, I-shaped members, double angles in-plane, double channels in-plane[b] $n = 3.4\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_{ye}}}\right)^{0.4}$	1.25	0.5n	0.75n	0.6n	0.9n
2. Double angles out-of-plane, double channels out-of-plane[b] $n = 2.8\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_{ye}}}\right)^{0.45}$	1.25	0.5n	0.75n	0.6n	0.9n

[a]Where HSS or pipe braces are filled with concrete and λ/λ_{hd} is less than or equal to 2.5; λ/λ_{hd} need not exceed 1.0 for computing n.

[b]Connectors for built-up members: Where the connectors for built-up braces do not satisfy the requirements of *Seismic Provisions* Section F2.5b, the values shall be multiplied by 0.5.

[c]For tension-only bracing, the values shall be divided by 2.0.

[d]In addition to consideration of connection strength in accordance with Section E1.4, values for braces shall be modified for connection robustness using n_p in accordance with Section C7.

[e]The limiting slenderness parameter for highly ductile compression elements, λ_{hd}, is defined in *Seismic Provisions* Table D1.1, with R_yF_y replaced by F_{ye}. λ and λ_{hd} shall be determined for each element of the cross section and the element producing the lowest value of m shall be used.

[f] The component modification factor, m, for IO shall not exceed m for LS.

TABLE C3.2 (continued) Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Buckling Braces Subjected to Axial Force

Component	IO[f]	Primary Component		Secondary Component	
		LS	CP	LS	CP
Buckling Braces in Tension[c][d][e]					
3. Rectangular HSS[a] $n = 4.7\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.0}\left(\frac{L_c/r}{\sqrt{E/F_{ye}}}\right)^{0.24}$	1.25	0.5*n*	0.75*n*	0.6*n*	0.9*n*
4. Round HSS and pipe[a] $n = 2.8\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_{ye}}}\right)^{0.45}$	1.25	0.5*n*	0.75*n*	0.6*n*	0.9*n*
5. Single angles $n = 7.2\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}$	1.25	0.5*n*	0.75*n*	0.6*n*	0.9*n*

[a]Where HSS or pipe braces are filled with concrete and λ/λ_{hd} is less than or equal to 2.5; λ/λ_{hd} need not exceed 1.0 for computing *n*.
[b]Connectors for built-up members: Where the connectors for built-up braces do not satisfy the requirements of *Seismic Provisions* Section F2.5b, the values shall be multiplied by 0.5.
[c]For tension-only bracing, the values shall be divided by 2.0.
[d]In addition to consideration of connection strength in accordance with Section E1.4, values for braces shall be modified for connection robustness using n_p in accordance with Section C7.
[e]The limiting slenderness parameter for highly ductile compression elements, λ_{hd}, is defined in *Seismic Provisions* Table D1.1, with R_yF_y replaced by F_{ye}. λ and λ_{hd} shall be determined for each element of the cross section and the element producing the lowest value of *m* shall be used.
[f] The component modification factor, *m*, for IO shall not exceed *m* for LS.

TABLE C3.3 Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Columns and Buckling-Restrained Braces Subjected to Axial Force

Component	Modeling Parameters			Expected Deformation Capacity		
	Plastic Axial Deformation, in. (mm)		Residual Strength Ratio	Plastic Axial Deformation, in. (mm)		
	a	*b*	*c*	IO	LS	CP
Columns in Tension	$5\Delta_T$	$7\Delta_T$	1.0	$0.5\Delta_T$	$6\Delta_T$	$7\Delta_T$
Buckling-Restrained Braces in Tension or Compression[a][b]	$13.3\Delta_y$	$13.3\Delta_y$	1.0	$3.0\Delta_y$	$10\Delta_y$	$13.3\Delta_y$

[a]Maximum strain of the buckling-restrained brace core shall not exceed 2.5%.
[b]If testing to demonstrate compliance with Section E3.4a is not available, the values shall be multiplied by 0.7.

TABLE C3.4
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Buckling Braces Subjected to Axial Force

	Modeling Parameters		Expected Deformation Capacity		
	Total Axial Deformation, in. (mm)	Strength Ratio at Maximum Deformation	Plastic Axial Deformation, in. (mm)		
Component	***d***	***f***	**IO[g]**	**LS**	**CP**
Buckling Braces in Compression[a] [d] [e] [f]					
1. Wide-flange shapes, I-shaped members, double angles in-plane, double channels in-plane[c] $n = 5.6\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_y}}\right)^{0.40}$	$n\Delta_c$	0.2	$1.5\Delta_c$	$0.7n\Delta_c$	$n\Delta_c$
2. Double angles out-of-plane, double channels out-of-plane[c] $n = 4.7\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_y}}\right)^{0.45}$					
3. Rectangular HSS braces[b] $n = 3.0\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.0}\left(\frac{L_c/r}{\sqrt{E/F_y}}\right)^{1.0}$					
4. Round HSS and pipe braces[b] $n = 4.7\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_y}}\right)^{0.45}$					
5. Single angle (L) $n = 12\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}$					

[a]The strength ratio at maximum deformation for braces in compression corresponds to the degraded post-buckling strength. For braces in tension, it is the strength at incipient brace fracture.
[b]Where HSS or pipe braces are filled with concrete and λ/λ_{hd} is less than or equal to 2.5, λ/λ_{hd} need not exceed 1.0 for computing n.
[c]Connectors for built-up members: Where the connectors for built-up braces do not satisfy the requirements of *Seismic Provisions* Section F2.5b, the values shall be multiplied by 0.5.
[d]For tension-only bracing, the values shall be divided by 2.0.
[e]In addition to consideration of connection strength in accordance with Section E1.4, values for braces shall be modified for connection robustness using n_p in accordance with Section C7.
[f] The limiting width-to-thickness ratio, λ_{hd}, is defined in *Seismic Provisions* Table D1.1, with R_yF_y replaced by F_{ye}. For concrete-filled HSS or pipe braces, λ_{hd} shall be determined for the corresponding hollow section. λ and λ_{hd} shall be determined for each element of the cross section and the element producing the lowest permissible deformation shall be used.
[g]The permissible deformations for IO shall not exceed that for LS.

TABLE C3.4 (continued)
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Buckling Braces Subjected to Axial Force

<table>
<tr><th rowspan="3">Component</th><th colspan="2">Modeling Parameters</th><th colspan="3">Expected Deformation Capacity</th></tr>
<tr><th>Total Axial Deformation, in. (mm)</th><th>Strength Ratio at Maximum Deformation</th><th colspan="3">Plastic Axial Deformation, in. (mm)</th></tr>
<tr><th>d</th><th>f</th><th>IO[g]</th><th>LS</th><th>CP</th></tr>
<tr><td>Buckling Braces in Tension[a][d][e][f]</td><td rowspan="6">$n\Delta_T$</td><td rowspan="6">1.0</td><td rowspan="6">$1.5\Delta_T$</td><td rowspan="6">$0.7n\Delta_T$</td><td rowspan="6">$n\Delta_T$</td></tr>
<tr><td>1. Wide-flange shapes, I-shaped members, double angles in-plane, double channels in-plane[c]
$$n = 3.4\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_y}}\right)^{0.4}$$</td></tr>
<tr><td>2. Double angles out-of-plane, double channels out-of-plane[c]
$$n = 2.8\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_y}}\right)^{0.45}$$</td></tr>
<tr><td>3. Rectangular HSS braces[b]
$$n = 4.7\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.0}\left(\frac{L_c/r}{\sqrt{E/F_y}}\right)^{0.24}$$</td></tr>
<tr><td>4. Round HSS and pipe braces[b]
$$n = 2.8\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}\left(\frac{L_c/r}{\sqrt{E/F_y}}\right)^{0.45}$$</td></tr>
<tr><td>5. Single angles
$$n = 7.2\left(\frac{\lambda}{\lambda_{hd}}\right)^{-1.7}$$</td></tr>
</table>

[a]The strength ratio at maximum deformation for braces in compression corresponds to the degraded post-buckling strength. For braces in tension, it is the strength at incipient brace fracture.
[b]Where HSS or pipe braces are filled with concrete and λ/λ_{hd} is less than or equal to 2.5, λ/λ_{hd} need not exceed 1.0 for computing n.
[c]Connectors for built-up members: Where the connectors for built-up braces do not satisfy the requirements of *Seismic Provisions* Section F2.5b, the values shall be multiplied by 0.5.
[d]For tension-only bracing, the values shall be divided by 2.0.
[e]In addition to consideration of connection strength in accordance with Section E1.4, values for braces shall be modified for connection robustness using n_p in accordance with Section C7.
[f] The limiting width-to-thickness ratio, λ_{hd}, is defined in *Seismic Provisions* Table D1.1, with R_yF_y replaced by F_{ye}. For concrete-filled HSS or pipe braces, λ_{hd} shall be determined for the corresponding hollow section. λ and λ_{hd} shall be determined for each element of the cross section and the element producing the lowest permissible deformation shall be used.
[g]The permissible deformations for IO shall not exceed that for LS.

TABLE C3.5
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Columns and Braces Subjected to Flexure with Axial Compression or Tension[a]

Axial Load Ratio and Section Compactness	IO	Primary Component LS	Primary Component CP	Secondary Component LS	Secondary Component CP
Columns and Braces in Compression					
For $\frac{P_{UF}}{P_{ye}} < 0.2$					
1. Highly ductile ($\lambda \le \lambda_{hd}$)	2	6	8	10	12
2. Non-moderately ductile ($\lambda \ge \lambda_{md}$)	1.25	1.25	2	2	3
3. Other	Linear interpolation between the values on lines 1. and 2. for flange, wall, and web slenderness shall be performed, and the lowest resulting value shall be used.				
For $\frac{P_{UF}}{P_{ye}} \ge 0.2$					
1. Highly ductile ($\lambda \le \lambda_{hd}$)	$1.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$7.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$10.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$13.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$16.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$
2. Non-moderately ductile ($\lambda \ge \lambda_{md}$)	$0.375\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$0.375\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$1.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$1.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$4.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$
3. Other	Linear interpolation between the values on lines 1. and 2. for flange, wall, and web slenderness shall be performed, and the lowest resulting value shall be used.				
Columns and Braces in Tension					
For $\frac{P_{UF}}{P_{ye}} < 0.2$	2	6	8	10	12
For $\frac{P_{UF}}{P_{ye}} \ge 0.2$	$1.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$7.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$10.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$13.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$	$16.5\left(1-\frac{5}{3}\frac{P_{UF}}{P_{ye}}\right)+1 \ge 1$

[a]The limiting width-to-thickness ratios, λ_{hd} and λ_{md}, are defined in *Seismic Provisions* Table D1.1, with R_yF_y replaced by F_{ye} and α_sP_r replaced by P_{UF}. λ shall be compared to λ_{hd} and λ_{md} for each element of the cross section and the element producing the lowest value of *m* shall be used.

If the out-of-plane moment, M_{UDx} or M_{UDy}, is less than 0.15 times the out-of-plane expected plastic flexural strength, M_{pcex} or M_{pcey}, whichever applies, then the flexural behavior shall be designated as in-plane-controlled and it is permitted to neglect the effects of the out-of-plane moment demand in Equation C3-4:

$$\frac{|M_{UDx}|}{m_xM_{pcex}}+\frac{|M_{UDy}|}{m_yM_{pcey}} \le \kappa \tag{C3-4}$$

If M_{CE} is limited by the limit state of shear yielding, the column or brace shall be assessed in accordance with Section C3.4a.3,

where

M_{pcex}, the expected plastic flexural strength of the section about the major principal axis (x-axis) at P_{UF}, kip-in. (N-mm), is determined as follows:

(1) When $\dfrac{|P_{UF}|}{P_{ye}} < 0.2\kappa$

$$M_{pcex} = M_{CE} = \left(1 - \frac{|P_{UF}|}{2P_{ye}}\right) M_{pex} \quad \text{(C3-5)}$$

(2) When $\dfrac{|P_{UF}|}{P_{ye}} \geq 0.2\kappa$

$$M_{pcex} = M_{CE} = \frac{9}{8}\left(1 - \frac{|P_{UF}|}{P_{ye}}\right) M_{pex} \quad \text{(C3-6)}$$

M_{pcey}, the expected plastic flexural strength of the section about the minor principal axis (y-axis) at P_{UF}, kip-in. (N-mm), shall be determined using Equations C3-5 and C3-6, except that M_{pey} shall be substituted for M_{pex}. Exception: It is permitted to determine M_{pcey} for wide-flange sections as follows:

(1) When $\dfrac{|P_{UF}|}{P_{ye}} < 0.4\kappa$

$$M_{pcey} = M_{CE} = \left(1 - \frac{|P_{UF}|}{4P_{ye}}\right) M_{pey} \quad \text{(C3-7)}$$

(2) When $\dfrac{|P_{UF}|}{P_{ye}} \geq 0.4\kappa$

$$M_{pcey} = M_{CE} = \frac{3}{2}\left(1 - \frac{|P_{UF}|}{P_{ye}}\right) M_{pey} \quad \text{(C3-8)}$$

M_{pex} = expected plastic flexural strength of the section about the x-axis in the absence of axial force, determined in accordance with Section C2.3a.1 at $P_{UF} = 0$, kip-in. (N-mm)

M_{pey} = expected plastic flexural strength of the section about the y-axis in the absence of axial force, determined in accordance with Section C2.3a.1 at $P_{UF} = 0$, kip-in. (N-mm)

M_{UDx} = bending moment about the x-axis determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kip-in. (N-mm)

M_{UDy} = bending moment about the y-axis determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kip-in. (N-mm)

P_{UF} = axial force (compression or tension) determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)

m_x = component capacity modification factor, m, for column flexure about the x-axis at P_{UF} in accordance with Table C3.5

m_y = component capacity modification factor, m, for column flexure about the y-axis at P_{UF} in accordance with Table C3.5

κ = knowledge factor, determined in accordance with ASCE/SEI 41, Section 6.2.4

2. Member Strength

A steel column or brace in compression shall satisfy Equations C3-9, C3-10, and C3-11 for a given performance level.

If the out-of-plane moment, M_{Ux} or M_{Uy}, is less than 0.15 times the out-of-plane flexural strength, M_{Cx} or M_{Cy}, whichever applies, then the flexural behavior shall be designated as in-plane-controlled and it is permitted to neglect the effects of the out-of-plane moment in Equation C3-9.

If M_{CxLTB} or M_{CxLB} is less than M_{pex}, or M_{CyLB} is less than M_{pey}, then m_x or m_y, as applicable, in Equation C3-9 shall be replaced by m_e determined from Equation C2-1, except that M_{CxLTB}, M_{CxLB}, or M_{CyLB}, as applicable, shall be substituted for M_{CE}.

$$\frac{M_{Ux}}{m_x M_{Cx}} + \frac{M_{Uy}}{m_y M_{Cy}} \le \kappa \tag{C3-9}$$

and

$$\frac{|P_{UF}|}{P_{ye}} \le 0.75\kappa \tag{C3-10}$$

and

$$\frac{|P_{UF}|}{P_{CL}} \le \kappa \tag{C3-11}$$

where

M_{Cx} = flexural strength of the member about the major principal axis (x-axis) at P_{UF}, kip-in. (N-mm). If flexure is deformation-controlled, $M_{Cx} = M_{CEx}$; otherwise, flexure is force-controlled and $M_{Cx} = M_{CLx}$.

For lateral-torsional buckling of wide-flange and HSS, M_{Cx} is determined as follows:

(1) When $\dfrac{|P_{UF}|}{P_{CL}} < 0.2\kappa$

$$M_{Cx} = \left(1 - \frac{|P_{UF}|}{2P_{CL}}\right) M_{CxLTB} \tag{C3-12}$$

(2) When $\dfrac{|P_{UF}|}{P_{CL}} \geq 0.2\kappa$

$$M_{Cx} = \frac{9}{8}\left(1 - \frac{|P_{UF}|}{P_{CL}}\right) M_{CxLTB} \tag{C3-13}$$

For local buckling of all sections, M_{Cx} shall be determined using Equations C3-12 and C3-13, except that M_{CxLB} shall be substituted for M_{CxLTB}.

M_{CxLB} = local buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm), determined in accordance with Section C2.3a.1 or C2.3b.1. If flexure is deformation-controlled, $M_{CxLB} = M_{CExLB}$; otherwise, flexure is force-controlled and $M_{CxLB} = M_{CLxLB}$.

M_{CxLTB} = lateral-torsional buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm). If flexure is deformation-controlled, $M_{CxLTB} = M_{CExLTB}$; otherwise flexure is force-controlled and $M_{CxLTB} = M_{CLxLTB}$.

M_{Cy} = flexural strength of the member about the minor principal axis (y-axis) at P_{UF}, kip-in. (N-mm). If flexure is deformation-controlled, $M_{Cy} = M_{CEy}$; otherwise, flexure is force-controlled and $M_{Cy} = M_{CLy}$.

For all sections, M_{Cy} shall be taken as M_{pcey}, determined in accordance with Section C3.4a.2.a.1, unless governed by local buckling. If the section is governed by local buckling, M_{Cy} shall be determined using Equations C3-12 and C3-13, except that M_{CyLB} shall be substituted for M_{CxLTB}.

M_{CyLB} = local buckling flexural strength of the member about the y-axis in the absence of axial force, kip-in. (N-mm), determined in accordance with Section C2.3a.1 or C2.3b.1. If flexure is deformation-controlled, $M_{CyLB} = M_{CEyLB}$; otherwise, flexure is force-controlled and $M_{CyLB} = M_{CLyLB}$.

M_{CEx} = expected flexural strength of the member about the x-axis, kip-in. (N-mm)

M_{CExLB} = expected local buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)

M_{CExLTB} = expected lateral-torsional buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm). M_{CExLTB} shall be determined using equations for M_n for lateral-torsional buckling limit states given in *Specification* Chapter F, without the upper-bound limit of the plastic bending moment, M_p, except that F_{ye} shall be substituted for F_y.

M_{CEy} = expected flexural strength of the member about the y-axis, kip-in. (N-mm)

M_{CEyLB} = expected local buckling flexural strength of the member about the y-axis in the absence of axial force, kip-in. (N-mm)

M_{CLx} = lower-bound flexural strength of the member about the x-axis, kip-in. (N-mm)

M_{CLxLB} = lower-bound local buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)

M_{CLxLTB} = lower-bound lateral-torsional buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm). M_{CLxLTB} shall be determined using equations for M_n for lateral-torsional buckling limit states given in *Specification* Chapter F, without the upper-bound limit M_p, except that F_{yL} shall be substituted for F_y.

M_{CLy} = lower-bound flexural strength of the member about the y-axis, kip-in. (N-mm)

M_{CLyLB} = lower-bound local buckling flexural strength of the member about the y-axis in the absence of axial force, kip-in. (N-mm)

M_{Ux} = bending moment about the x-axis, kip-in. (N-mm). If flexure is deformation-controlled, $M_{Ux} = M_{UDx}$; otherwise, flexure is force-controlled and $M_{Ux} = M_{UFx}$.

M_{Uy} = bending moment about the y-axis, kip-in. (N-mm). If flexure is deformation-controlled, $M_{Uy} = M_{UDy}$; otherwise, flexure is force-controlled and $M_{Uy} = M_{UFy}$.

M_{UFx} = bending moment about the x-axis determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kip-in. (N-mm)

M_{UFy} = bending moment about the y-axis determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kip-in. (N-mm)

P_{CL} = lower-bound compressive strength determined in accordance with Section C3.3b.1, kips (N)

A steel column or brace in tension shall satisfy Equation C3-9, except that P_{CL} shall be taken as the expected tensile strength, T_{CE}, if the axial action is deformation-controlled or the lower-bound tensile strength, T_{CL}, if the axial action is force-controlled; these strengths shall be determined in accordance with Section C3.3a.1 and Section C3.3b.1, respectively.

If a column or brace yields in tension, it shall satisfy Equation C3-14 for each performance level:

$$\frac{P_{UD}}{m_t T_{CE}} \leq \kappa \tag{C3-14}$$

where

P_{UD} = tensile force in the member determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)

m_t = component capacity modification factor, m, for column or brace in axial tension taken from Table C3.1 or C3.2

b. Nonlinear Analysis Procedures

When constructing the nonlinear force-deformation model for use in the nonlinear analysis procedures, the generalized force-deformation curve for flexural behavior shown in Figure C1.1, with modeling parameters a, b, and c as given in Table C3.6, shall be used for columns or braces. Alternatively, these relationships may be derived from testing or analysis. For columns or braces, it is permitted to take α_h for flexural action as 3% of the elastic slope when $P_{UF}/P_{CL} \le 0.20$, and 1% of the elastic slope when $P_{UF}/P_{CL} > 0.20$. Further modification of the curve is permitted if a greater value for α_h is justified by testing or analysis. When the flexural behavior of a column or brace is considered deformation-controlled, the plastic chord rotation demand, θ_p, predicted by analysis shall be not greater than the permissible plastic chord rotation provided in Table C3.6 for a given performance level.

If the column or brace is flexure-controlled, the yield chord rotation, θ_y, of the column or brace shall be determined from analysis as the rotation at which the computed moment at the location of flexural yielding is M_{CE}. If the column or brace is flexure-controlled and fully restrained at both ends without consideration of panel-zone stiffness, and loading is such that the point of inflection under seismic loading is located at the midspan, it is permitted to determine θ_y from Equation C3-15. Otherwise, if the column or brace is shear-controlled or shear-flexure-controlled, θ_y shall be taken as γ_y determined from Section C3.4a.3.b.

$$\theta_y = \frac{M_{CE} L_{CL}(1+\eta)}{6(\tau_b E)I} \tag{C3-15}$$

where

L_{CL} = length of member taken between beam centerlines, in. (mm)

If $M_{CE} < M_{pe}$, the value computed from Equation C3-15 shall be multiplied by the factor, Ψ, determined from Equation C2-4.

Where shear deformation in a column or brace does not change the component deformation by more than 5% or is not included in the analysis of the analytical model, it is permitted to take η as zero.

> **User Note:** Shear deformation (accounted for by η) in a flexure-controlled column or brace with a length greater than $10M_{CE}/V_{CE}$, including axial force interaction, is generally small and can be neglected in Equation C3-15.

Where the modeling parameter, a, is equal to zero or where $P_G/P_{ye} > 0.6$, the component shall remain elastic for flexure.

Columns or braces classified as deformation-controlled for flexure shall also satisfy Equations C3-9, C3-10, and C3-11 when the column or brace is in compression except that values for m_x and m_y shall be taken as unity.

TABLE C3.6
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Columns and Braces Subjected to Flexure with Axial Compression or Tension[c][d]

Axial Load Ratio and Section Compactness	Expected Deformation Capacity		
	Plastic Rotation Angle, rad		
	IO	LS	CP
Columns and Braces in Compression[a][b]			
1. Highly ductile ($\lambda \le \lambda_{hd}$) $a = 5.5\left(\frac{h}{t_w}\right)^{-0.95}\left(\frac{L}{r_y}\right)^{-0.5}\left(1-\frac{P_G}{P_{ye}}\right)^{2.4} \le 0.07$ $b = 20\left(\frac{h}{t_w}\right)^{-0.9}\left(\frac{L}{r_y}\right)^{-0.5}\left(1-\frac{P_G}{P_{ye}}\right)^{3.4} \le 0.07$ $c = 0.4 - 0.4\frac{P_G}{P_{ye}}$	0.5a	0.75b	b
2. Non-moderately ductile ($\lambda \ge \lambda_{md}$) $a = 1.2\left(1-\frac{P_G}{P_{ye}}\right)^{1.2}\left(1.4\frac{L}{r_y}+0.1\frac{h}{t_w}+0.9\frac{b_f}{2t_f}\right)^{-1} - 0.0023 \ge 0$ $b = 2.5\left(1-\frac{P_G}{P_{ye}}\right)^{1.8}\left(0.1\frac{L}{r_y}+0.2\frac{h}{t_w}+2.7\frac{b_f}{2t_f}\right)^{-1} - 0.0097 \ge 0$ $c = 0.5 - 0.5\frac{P_G}{P_{ye}}$	0.5a	0.75b	b
3. Other: Linear interpolation between the values on lines 1. and 2. for both flange slenderness and web slenderness shall be performed, and the lowest resulting value shall be used.			

[a] θ_y for the purpose of computing a plastic rotation angle is determined using Equation C3-15.

[b] The limiting width-to-thickness ratios, λ_{hd} and λ_{md}, are defined in AISC *Seismic Provisions* Table D1.1, with R_yF_y replaced by F_{ye} and $\alpha_s P_r$ replaced with P_G. λ shall be compared to λ_{hd} and λ_{md} for each element of the cross section and the element producing the lowest permissible deformation shall be used.

[c] Values are applicable for flexure-controlled beams with $L_v \ge 2.6M_{CE}/V_{CE}$. Linearly interpolate values to 0.0 when $L_v \le 1.6M_{CE}/V_{CE}$.

[d] For beams in concentrically braced frames with V- or inverted V-bracing, the permissible performance parameters are permitted to be multiplied by 1.25.

where

L = laterally unbraced length of member, in. (mm)
P_G = axial force component of the gravity load as determined by ASCE/SEI 41, Equation 7-3, kips (N)
b_f = width of flange, in. (mm)
d = full nominal depth of member, in. (mm)
h = for rolled shapes, the clear distance between flanges less the fillet or corner radii;
= for built-up welded sections, the clear distance between flanges;
= for built-up bolted sections, the distance between fastener lines; and
= for tees, the overall depth, in. (mm)
r_y = radius of gyration about y-axis, in. (mm)
t_f = thickness of flange, in. (mm)

TABLE C3.6 (continued) Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Columns and Braces Subjected to Flexure with Axial Compression or Tension[c][d]

Axial Load Ratio and Section Compactness	Expected Deformation Capacity		
	Plastic Rotation Angle, rad		
	IO	LS	CP
Columns and Braces in Compression[a][b]			
4. Rectangular HSS and built-up box shapes[e] $a = 1.1(\lambda)^{-1.2}\left(1-\frac{P_G}{P_{ye}}\right)^{1.8} \le 0.05$ $b = 0.5(\lambda)^{-0.6}\left(1-\frac{P_G}{P_{ye}}\right)^{1.2} - 0.01 \le 0.08$ $c = 0.25$	$0.5a$	$0.75b$	b
Columns and Braces in Tension[a][b]			
1. For $\frac{\lvert P_G \rvert}{P_{ye}} < 0.2$ $a = 9\theta_y$ $b = 11\theta_y$ $c = 0.6$	θ_y	a	b
2. For $\frac{\lvert P_G \rvert}{P_{ye}} \ge 0.2$ $a = 13.5\left(1-\frac{5}{3}\frac{\lvert P_G \rvert}{P_{ye}}\right)\theta_y \ge 0$ $b = 16.5\left(1-\frac{5}{3}\frac{\lvert P_G \rvert}{P_{ye}}\right)\theta_y \ge 0$ $c = 0.6\left(1-\frac{5}{3}\frac{\lvert P_G \rvert}{P_{ye}}\right)+0.2 \ge 0.2$	$0.25\theta_y$	$13.5\left(1-\frac{5}{3}\frac{\lvert P_G \rvert}{P_{ye}}\right)\theta_y \ge 0$	$16.5\left(1-\frac{5}{3}\frac{\lvert P_G \rvert}{P_{ye}}\right)\theta_y \ge 0$
3. Other: Linear interpolation between the values on lines 1. and 2. for both flange slenderness and web slenderness shall be performed, and the lowest resulting value shall be used.			

[a]θ_y for the purpose of computing a plastic rotation angle is determined using Equation C3-15.
[b]The limiting width-to-thickness ratios, λ_{hd} and λ_{md}, are defined in AISC *Seismic Provisions* Table D1.1, with R_yF_y replaced by F_{ye} and $\alpha_s P_r$ replaced with P_G. λ shall be compared to λ_{hd} and λ_{md} for each element of the cross section and the element producing the lowest permissible deformation shall be used.
[c]Values are applicable for flexure-controlled beams with $L_v \ge 2.6M_{CE}/V_{CE}$. Linearly interpolate values to 0.0 when $L_v \le 1.6M_{CE}/V_{CE}$.
[d]For beams in concentrically braced frames with V- or inverted V-bracing, the permissible performance parameters are permitted to be multiplied by 1.25.
[e]For steel columns with built-up box shapes, the values shall be multiplied by 0.75.

where
L = laterally unbraced length of member, in. (mm)
P_G = axial force component of the gravity load as determined by ASCE/SEI 41, Equation 7-3, kips (N)
b_f = width of flange, in. (mm)
d = full nominal depth of member, in. (mm)
h = for rolled shapes, the clear distance between flanges less the fillet or corner radii;
= for built-up welded sections, the clear distance between flanges;
= for built-up bolted sections, the distance between fastener lines; and
= for tees, the overall depth, in. (mm)
r_y = radius of gyration about y-axis, in. (mm)
t_f = thickness of flange, in. (mm)

3. Shear Actions Concurrent with Axial Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior is considered deformation-controlled, the shear behavior strength shall be evaluated in accordance with this section. If the out-of-plane shear, V_{UDx} or V_{UDy}, is less than 0.15 times the out-of-plane expected plastic shear strength, V_{pcex} or V_{pcey}, then the shear behavior shall be designated as in-plane-controlled and it is permitted to neglect the effects of the out-of-plane shear demand in Equation C3-16.

$$\frac{\left|V_{UDx}\right|}{m_x V_{pcex}} + \frac{\left|V_{UDy}\right|}{m_y V_{pcey}} \le \kappa \tag{C3-16}$$

where

V_{UDx} = shear along the x-axis determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)

V_{UDy} = shear along the y-axis determined as a deformation-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)

V_{pcex}, the expected plastic shear strength of the section along the major principal axis (x-axis) at P_{UF}, kips (N), is determined as follows:

(1) When $\dfrac{\left|P_{UF}\right|}{P_{ye}} < 0.2\kappa$

$$V_{pcex} = V_{CE} = V_{pex} \tag{C3-17}$$

(2) When $\dfrac{\left|P_{UF}\right|}{P_{ye}} \ge 0.2\kappa$

$$V_{pcex} = V_{CE} = V_{pex}\sqrt{1-\left(\frac{\left|P_{UF}\right|}{P_{ye}}\right)^2} \tag{C3-18}$$

V_{pcey} = expected plastic shear strength of the section along the minor principal axis (y-axis) at P_{UF}, kips (N), determined using Equations C3-17 and C3-18, except that V_{pey} shall be substituted for V_{pex}, kips (N)

V_{pex} = expected plastic shear strength along the x-axis in the absence of axial force, determined in accordance with Section C2.3a.2, kips (N)

V_{pey} = expected plastic shear strength along the y-axis in the absence of axial force, determined in accordance with Section C2.3a.2, kips (N)

m_x = component capacity modification factor, m, for shear along the x-axis at P_{UF} in accordance with Table C3.7

m_y = component capacity modification factor, m, for shear along the y-axis at P_{UF} in accordance with Table C3.7

TABLE C3.7
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Columns and Braces Subjected to Shear[a] [b] [c] [d]

Axial Load Ratio and Member Length	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
For $\frac{\lvert P_{UF}\rvert}{P_{ye}} \le 0.2$					
$L_v \le \frac{1.6\,M_{CE}}{V_{CE}}$ (shear-controlled)	1.5	9	13	13	15
For $\frac{\lvert P_{UF}\rvert}{P_{ye}} > 0.2$					
$L_v \le \frac{1.6\,M_{CE}}{V_{CE}}$ (shear-controlled)	$0.75\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$ ≥ 1	$12\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$ ≥ 1	$18\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$ ≥ 1	$18\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$ ≥ 1	$21\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$ ≥ 1

[a]Values are applicable for shear-controlled beams with three or more web stiffeners. If there are no stiffeners, divide values for shear-controlled beams by 2.0, but values need not be taken less than 1.25. Linear interpolation is permitted for one or two stiffeners.
[b]Assumes ductile detailing for beam in the shear yielding zone in accordance with the *Seismic Provisions*.
[c]Regardless of the modifiers applied, m need not be taken as less than 1.0.
[d]Values of m shall be 1.0 when $L_v \ge 2.6M_{CE}/V_{CE}$. For $1.6M_{CE}/V_{CE} < L_v < 2.6M_{CE}/V_{CE}$, m shall be linearly interpolated between the tabulated values and 1.0.

b. Nonlinear Analysis Procedures

When constructing the nonlinear force-deformation model for use in the nonlinear analysis procedures, the generalized force-deformation curve for shear behavior shown in Figure C1.1, with modeling parameters a, b, and c as given in Table C2.4, shall be used for column or braces. Alternatively, these relationships may be derived from testing or analysis. For columns or braces, it is permitted to take α_h for shear action as 6% of the elastic slope. Further modification of the curve is permitted if a greater value for α_h is justified by testing or analysis. When the shear behavior is considered deformation-controlled, the plastic shear deformation demand, γ_p, predicted by analysis shall be not greater than the permissible plastic shear deformation provided in Table C3.8 for a given performance level. The yield shear deformation of a column or brace, γ_y, shall be determined from Equation C3-19:

$$\gamma_y = \frac{V_{CE}}{K_e L_v} \tag{C3-19}$$

TABLE C3.8
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Columns and Braces Subjected to Shear with Axial Compression or Tension[a][b][c][d]

Axial Load Ratio and Member Length	Modeling Parameters			Expected Deformation Capacity		
	Plastic Rotation Deformation, rad		Residual Strength Ratio	Plastic Shear Deformation, rad		
	a	*b*	*c*	IO	LS	CP
For $\frac{\lvert P_{UF}\rvert}{P_{ye}} \le 0.2$						
$L_v \le \frac{1.6\,M_{CE}}{V_{CE}}$ (shear-controlled)	0.15	0.17	0.8	0.005	0.14	0.16
For $\frac{\lvert P_{UF}\rvert}{P_{ye}} > 0.2$						
$L_v \le \frac{1.6\,M_{CE}}{V_{CE}}$ (shear-controlled)	$0.225\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right) > 0$	$0.225\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right) > 0$	$1.2\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right) > 0$	$0.0075\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right) > 0$	$0.21\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right) > 0$	$0.24\left(1-\frac{5}{3}\frac{\lvert P_{UF}\rvert}{P_{ye}}\right) > 0$

[a]Deformation is the rotation angle between the beam and column or portion of beam outside the shear yielding zone.
[b]Values are applicable for shear-controlled beams with three or more web stiffeners. If no stiffeners, divide values for shear-controlled beams by 2.0. Linear interpolation is permitted for one or two stiffeners.
[c]Assumes ductile detailing for beam in the shear yielding zone in accordance with the *Seismic Provisions*.
[d]Values shall be taken as 0.0 when $L_v \ge 2.6M_{CE}/V_{CE}$. For $1.6M_{CE}/V_{CE} < L_v < 2.6M_{CE}/V_{CE}$, values shall be linearly interpolated between the tabulated values and 0.0.

where

K_e = elastic shear stiffness, determined in accordance with Section C3.2, kip/in. (N/mm)

V_{CE} = expected shear strength of the column or brace, determined in accordance with Section C3.3a.3, kips (N)

Columns or braces classified as deformation-controlled for shear shall also satisfy Equation C3-16, except that values of m_x and m_y shall be taken as unity.

4b. Force-Controlled Actions

1. Axial Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the axial behavior of a column or brace is considered force-controlled, the lower-bound axial strength, $Q_{CL} = P_{CL}$, shall be determined in accordance with Section C3.3b.1.

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the axial behavior of a column or brace is considered force-controlled, the lower-bound axial strength, $Q_{CL} = P_{CL}$, shall be determined in accordance with Section C3.3b.1.

2. Flexural Actions Concurrent with Axial Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the flexural behavior is considered force-controlled, the column or brace shall satisfy Equations C3-9, C3-10, and C3-11 for a given performance level, computing all flexural strengths as lower-bound strengths with the lower-bound axial yield strength, $P_{yL} = A_g F_{yL}$, substituted for P_{ye} and the values for m_x and m_y taken as unity.

Columns or braces classified as force-controlled for flexure shall also satisfy Equation C3-4 for a given performance level, except that M_{UDx} and M_{UDy} shall be taken as M_{UFx} and M_{UFy}, respectively; the values for m_x and m_y shall be taken as unity; M_{pex} and M_{pey} shall be taken as the lower-bound plastic flexural strength about the x- and y-axis, M_{pLx} and M_{pLy}, respectively, with F_{yL} substituted for F_{ye}; and P_{ye} shall be taken as P_{yL}.

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the flexural behavior is considered force-controlled, the column or brace shall satisfy Equations C3-9, C3-10, and C3-11, when the column is in compression, except that m_x and m_y shall be taken as unity.

Columns or braces classified as force-controlled for flexure shall also satisfy Equation C3-4 for a given performance level, except that the values for m_x and m_y shall be taken as unity; M_{pex} and M_{pey} shall be taken as the lower-bound plastic flexural strength about the x- and y-axis, M_{pLx} and M_{pLy}, respectively, with F_{yL} substituted for F_{ye}; and P_{ye} shall be taken as P_{yL}.

3. Shear Actions Concurrent with Axial Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior is considered force-controlled, the column or brace shall satisfy Equation C3-16 for a given performance level, computing all shear strengths as lower-bound strengths with the lower-bound axial yield strength, P_{yL}, substituted for P_{ye}, and the values for m_x and m_y taken as unity.

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the shear behavior is considered force-controlled, the column or brace shall satisfy Equation C3-16, except that the values for m_x and m_y shall be taken as unity; V_{pex} and V_{pey} shall be taken as the lower-bound plastic shear strength along the x- and y-axis, V_{pLx} and V_{pLy}, respectively, with F_{yL} substituted for F_{ye}; and P_{ye} shall be taken as P_{yL}.

C4. PANEL ZONES

1. General

The component characteristics of panel zones at moment connections, subject to seismic forces or deformations from shear action with or without concurrent axial action, shall be determined in accordance with this section.

The shear behavior of a panel zone shall be designated as either deformation-controlled or force-controlled in accordance with Chapters D through I.

2. Stiffness

The stiffness of panel zones shall be based on principles of structural mechanics and as specified in the *Specification* unless superseded by supplemental provisions of this section or system-specific sections in Chapters D through I.

The force-deformation model for a panel zone shall account for all significant sources of deformation that affect its behavior, including those from axial, flexural, and shear actions.

Panel-zone flexibility shall be included in an analytical model by adding a panel zone at the beam-to-column joint. Alternatively, adjustment of the beam flexural stiffness to account for panel-zone flexibility is permitted. Where the expected shear strength of a panel zone exceeds the flexural strength of the adjacent beams (converted to applied shear on the panel zone) at a beam-to-column connection and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone is not required. In such cases, rigid offsets from the center of the column are permitted to represent the effective span of the beam. Otherwise, use of partially rigid offsets or centerline analyses is permitted.

2a. Flexural Stiffness

There are no additional requirements beyond those specified in Section C4.2.

2b. Axial Stiffness

There are no additional requirements beyond those specified in Section C4.2.

2c. Shear Stiffness

If the panel zone includes concrete encasement, then the shear stiffness is permitted to be determined using full composite action, including the effects of cracking, provided a mechanism exists for the transfer and distribution of forces to the surrounding components.

3. Strength

The shear strength of a panel zone shall be determined in accordance with this section.

The shear strength of the concrete encasement can be included in the shear strength of the panel zone provided a transfer mechanism exists that provides full composite action and distribution of forces to the surrounding components beyond the anticipated plastic deformations. Otherwise, the shear strength of a composite panel zone shall neglect the effect of the concrete.

3a. Deformation-Controlled Actions

The expected shear strength, V_{CE}, shall be determined using the equations for the nominal strength, R_n, determined from *Specification* Section J10.6(a), except that F_{ye} shall be substituted for F_y; P_{UF} shall be substituted for αP_r; t_p shall be substituted for t_w; and $Q_{CE} = V_{CE} = Q_y = V_{ye}$, where V_{ye} is the expected shear yield strength. P_{UF} shall be computed in accordance with Section C3 and dependent on the analysis type selected,

where

P_r = required axial strength using load and resistance factor design (LRFD) or allowable strength design (ASD) load combinations, kips (N)

t_p = total thickness of panel zone, including doubler plates, in. (mm)

t_w = thickness of column web, in. (mm)

3b. Force-Controlled Actions

The lower-bound shear strength, V_{CL}, shall be determined using the equations for the nominal strength, R_n, given in *Specification* Section J10.6(a), except that F_{yL} shall be substituted for F_y; P_{UF} shall be substituted for αP_r; t_p shall be substituted for t_w; and $Q_{CL} = V_{CL}$.

4. Permissible Performance Parameters

Permissible strengths and deformations for shear actions in a panel zone shall be computed in accordance with this section.

4a. Deformation-Controlled Actions

1. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior of a panel zone is considered deformation-controlled, the expected shear strength, $Q_{CE} = V_{CE}$, shall be determined in accordance with Section C4.3a and m shall be taken from Table C4.1 and modified by this section. The axial load, P_{UF}, shall be determined in accordance with Section C3.

The component capacity modification factor, m, of the panel zone in Table C4.1 for the LS and CP performance levels shall be multiplied by 2 when all of the following conditions are met:

(a) $V_{PZ}/V_{ye} < 1.10$, where the panel-zone shear, V_{PZ}, is determined from Equation C5-21 and V_{ye} is determined in accordance with Section C4.3a

(b) The beam flange-to-column flange connection is made with complete-joint-penetration (CJP) groove welds that satisfy the requirements of *Seismic Provisions* Section A3.4

(c) Beam flange-to-column flange connection welds are not located where column flanges are susceptible to local inelastic deformation

TABLE C4.1
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Panel Zones Subjected to Shear[a]

Axial Load	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
For $\frac{\lvert P_{UF}\rvert}{P_{ye}} \le 0.4$	1.5	4	5.5	6	6
For $\frac{\lvert P_{UF}\rvert}{P_{ye}} > 0.4$	$\left(\frac{2.5}{3}\right)\left(1-\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$	$\left(\frac{15}{3}\right)\left(1-\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$	$\left(\frac{22.5}{3}\right)\left(1-\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$	$\left(\frac{25}{3}\right)\left(1-\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$	$\left(\frac{25}{3}\right)\left(1-\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)+1$

CP = collapse prevention performance level as defined in ASCE/SEI 41, Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41, Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41, Chapter 2
[a]Regardless of the modifiers applied, *m* need not be taken as less than 1.0.

2. Nonlinear Analysis Procedures

When constructing the nonlinear force-deformation model for use in the nonlinear analysis procedures, the generalized force-deformation curve for shear behavior shown in Figure C1.1, with modeling parameters a, b, and c as given in Table C4.2, shall be used for panel zones. For Point B, V_{CE} shall be the shear strength associated with γ_y. In addition, an intermediate point, B*, between Points B and C shall be included where the shear strength associated with a total shear deformation of $4\gamma_y$ is determined using the equations for the nominal strength, R_n, determined from *Specification* Section J10.6(b), except that F_{ye} shall be substituted for F_y, P_{UF} shall be substituted for αP_r, and t_p shall be substituted for t_w. α_h for shear action beyond Point B* shall be taken as zero unless a greater value is justified by testing or analysis. Alternatively, these relationships may be derived from testing or analysis.

When the shear strength of a panel zone is considered deformation-controlled, the plastic shear deformation demand, γ_p, predicted by analysis shall be not greater than the permissible plastic shear deformation provided in Table C4.2 for a given performance level. The yield shear deformation, γ_y, shall be determined from Equation C4-1:

$$\gamma_y = \frac{F_{ye}}{G\sqrt{3}}\sqrt{1-\left(\frac{\lvert P_{UF}\rvert}{P_{ye}}\right)^2} \tag{C4-1}$$

Where the beam flanges are welded to the column flange, the permissible plastic shear deformation of the panel zone, $\gamma_{p,pz}$, shall be determined from Equations C4-2 or C4-3, when both of the following two conditions are met:

(1) $V_{PZ}/V_{ye} > 1.10$, where V_{PZ} is determined from Equation C5-21 and V_{ye} is determined in accordance with Section C4.3a.

TABLE C4.2
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Panel Zones Subjected to Shear

Axial Load	Modeling Parameters			Expected Deformation Capacity		
	Plastic Shear Deformation, rad		Residual Strength Ratio	Plastic Shear Deformation, rad		
	a	b	c	IO	LS	CP
For $\frac{\lvert P_{UF}\rvert}{P_{ye}} \le 0.4$	$\gamma_{p,pz}$	$\gamma_{u,pz}$	0.20[a]	$1\gamma_y$	$0.75\gamma_{u,pz}$	$\gamma_{u,pz}$
For $\frac{\lvert P_{UF}\rvert}{P_{ye}} > 0.4$	$\left(\frac{5}{3}\right)\left(1-\frac{\lvert P_{UF}\rvert}{P_{ye}}\right) \times \gamma_{p,pz}$	$\gamma_{u,pz}$	0.20[a]	$\left(\frac{5}{3}\right)\left(1-\frac{\lvert P_{UF}\rvert}{P_{ye}}\right) \times \gamma_y$	$0.75\gamma_{u,pz}$	$\gamma_{u,pz}$

[a]In lieu of using 20% of the panel zone shear strength, it is permitted to use 20% of the flexural strength of the beams framing into the joint.

(2) Beam flange-to-column flange connection welds are located where column flanges are susceptible to local inelastic deformation.

(a) For connections where the beam flange-to-column flange connection is made with CJP groove welds that do not meet the requirements of *Seismic Provisions* Section A3.4

$$\gamma_{p,pz} = \frac{0.092F_y}{G}\left(\alpha + \frac{3.45}{\alpha}\right)\left[1-\left(\frac{\lvert P_{UF}\rvert}{2P_{ye,cf}}\right)^2\right] \le 6\gamma_y \tag{C4-2}$$

(b) For connections where the beam flange-to-column flange connection is made with CJP groove welds that meet the requirements of *Seismic Provisions* Section A3.4

$$\gamma_{p,pz} = \frac{0.183F_y}{G}\left(\alpha + \frac{3.45}{\alpha}\right)\left[1-\left(\frac{\lvert P_{UF}\rvert}{2P_{ye,cf}}\right)^2\right] \le 10\gamma_y \tag{C4-3}$$

where

F_y = specified minimum yield stress of the column web, ksi (MPa)

$P_{ye,cf}$ = expected axial yield strength of the column flange, kips (N)
$= A_{cf}F_{ye}$

A_{cf} = area of column flange, in.^{2} (mm^2)
$= b_{cf}t_{cf}$

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

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

$\alpha = \dfrac{d_b}{t_{cf}}$

d_b = smallest depth of the connecting beams at a panel zone, in. (mm)

For all other cases, $\gamma_{p,pz}$ is permitted to be determined according to Equation C4-3.

Where the beam flanges are welded to the column flange, the permissible ultimate shear deformation of the panel zone, $\gamma_{u,pz}$, shall be based on the ultimate plastic rotation of the beam-column connection; the *b* parameter shown in Figure C1.1 and given in Table C5.5.

4b. Force-Controlled Actions

1. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior of a panel zone is considered force-controlled, the lower-bound shear strength, $Q_{CL} = V_{CL}$, shall be determined in accordance with Section C4.3b.

2. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the shear behavior of a panel zone is considered force-controlled, the lower-bound shear strength, $Q_{CL} = V_{CL}$, shall be determined in accordance with Section C4.3b.

C5. BEAM AND COLUMN CONNECTIONS

1. General

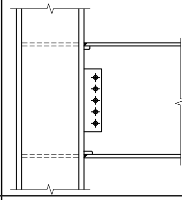
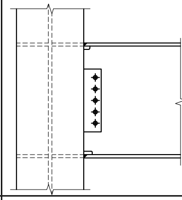
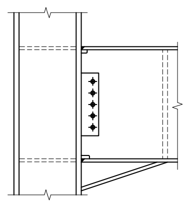
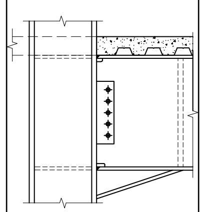
This section addresses the component characteristics of steel and composite steel-concrete beam-to-column connections, column-to-base connections, and column and beam splices subject to seismic forces and deformations.

The axial, flexural, and shear behavior of a connection shall be designated as either deformation-controlled or force-controlled in accordance with Chapters D through I.

1a. Fully Restrained Beam-to-Column Moment Connections

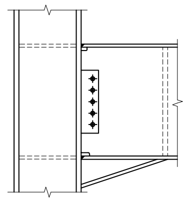
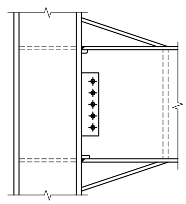
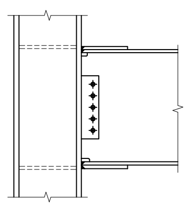
The connection shall be classified as fully restrained (FR) if the connection deformations, not including panel-zone deformations, do not contribute more than 10% to the total lateral deflection of the frame and the connection is at least as strong as the weaker of the two members being joined. Table C5.1 shall be used to identify the various FR connections for which permissible performance parameters are provided in Section C5. Connections described in Table C5.1 are permitted to be classified as FR connections without checking their contributions to the total lateral deflection of the frame. Modeling procedures, permissible performance parameters, and retrofit measures for moment frames with FR beam-to-column connections shall be as determined in Chapter D.

TABLE C5.1
Beam-to-Column Fully Restrained Connections

Connection	Description[a]	
Welded unreinforced flange (WUF) (pre-1995), strong-axis connection	Complete-joint-penetration (CJP) groove welds between beam flanges and column flanges. The beam web is either bolted, with or without supplemental welds, or welded to a shear plate attached to the column, or directly welded to the column. The existing flange welds are made with any filler metal (i.e., with or without specified minimum notch toughness requirements). A composite slab is or is not present.	
Welded unreinforced flange (WUF) (pre-1995), weak axis connection	CJP groove welds between beam flanges and continuity plates placed between the column flanges. The beam web is either bolted, with or without supplemental welds, or welded to a shear plate attached to the column, or directly welded to the column. The existing flange welds are made with any filler metal (i.e., with or without specified minimum notch toughness requirements). A composite slab is or is not present.	
Bottom haunch with top flange weld conforming to *Seismic Provisions* Section A3.4 Note: top flange connection is direct, or uses a top plate.	The top beam flange is connected to the column flange either (a) directly by a CJP groove weld or (b) by a welded top plate that is connected to the column with a CJP groove weld and to the beam such that the ultimate strength of the plate is fully developed. The beam web is either bolted, with or without supplemental welds, or welded to a single-plate shear connection, or welded directly to the column flange. A composite slab is or is not present. The existing weld connecting the top flange or top plate to the column flange shall meet the requirements of *Seismic Provisions* Section A3.4a. Alternatively, the existing weld connecting the top flange or top plate to the column flange shall be removed and replaced with a CJP groove weld meeting the requirements of *Seismic Provisions* Section A3.4b. The bottom haunch shall be connected to both the beam flange and the column flange with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4b. The existing bottom flange weld is made with any filler metal (i.e., with or without specified minimum notch toughness requirements).	
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and composite slab	The top beam flange is connected to the column flange with an existing CJP groove weld, made with any filler metal (i.e., with or without specified minimum notch toughness requirements). The beam web is either bolted, with or without supplemental welds, or welded to a single-plate shear connection, or welded directly to the column. A composite slab is present with minimum reinforcement ratio of 0.007 and connection between the beam and slab producing at least 12% composite action. The bottom haunch shall be connected to both the beam flange and the column flange with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4b. If the beam web is not welded directly to the column or the shear plate not welded to both the beam and column, the shear plate shall be retrofitted by welding it to the beam web. A reinforcing fillet shall be added to the backing bar under the top flange weld.	

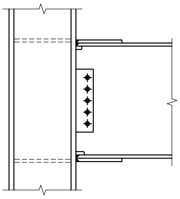
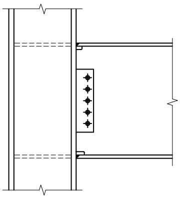
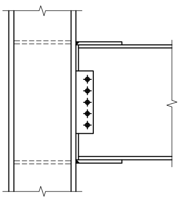
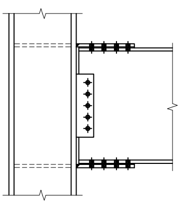
[a]Unless noted otherwise, existing web welds are permitted that are made with weld metal that does not have specified minimum notch toughness requirements. Existing web welds are permitted to remain unaltered, unless otherwise noted.

TABLE C5.1 (continued)
Beam-to-Column Fully Restrained Connections

Connection	Description[a]	
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and a non-composite slab or no slab	The top beam flange is connected to the column flange with existing CJP groove weld, made with any filler metal (i.e., with or without specified minimum notch toughness requirements). The beam web is either bolted, with or without supplemental welds, or welded to a single-plate shear connection, or welded directly to the column. If a slab is present, it has a minimum reinforcement ratio less than 0.007 or connection between the slab produces less than 12% composite action. The bottom haunch shall be connected to both the beam flange and the column flange with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4b. The existing top and bottom flange welds do not need to conform to *Seismic Provisions* Section A3.4 and are permitted to remain unaltered.	
Welded top and bottom haunches	The beam flanges are connected to the column flanges with existing CJP groove welds. The existing beam flange to column flange welds are made with any filler metal (i.e., with or without specified minimum notch toughness requirements). The beam web is either bolted, with or without supplemental welds, or welded to a single-plate shear connection, or directly welded to the column flange. A composite slab is or is not present. If the haunches are existing, they shall be connected to both the beam flanges and the column flange with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4a. Alternatively, the existing CJP groove welds, for the haunches, shall be removed and replaced with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4b. If the haunches are added as part of a retrofit, they shall be connected to both the beam flanges and the column flange with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4b. The existing beam flange to column flange welds are permitted to remain unaltered.	
Welded cover plate in WUF with existing flange weld remaining	The beam flanges are connected to the column flanges with existing CJP groove welds. The beam web is either bolted, with or without supplemental welds, or welded to a single-plate shear connection or welded directly to the column flange. A composite slab is or is not present. The existing beam flange welds need not be altered. The new cover plates shall be connected to the column flange with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4b. The cover plates are permitted to be welded or bolted to the beam.	

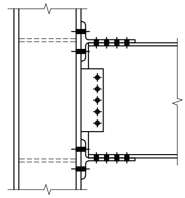
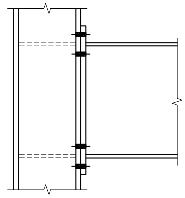
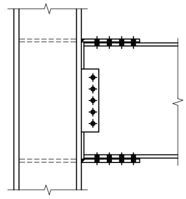
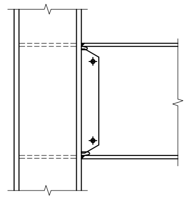
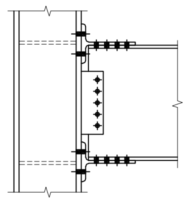
[a]Unless noted otherwise, existing web welds are permitted that are made with weld metal that does not have specified minimum notch toughness requirements. Existing web welds are permitted to remain unaltered, unless otherwise noted.

TABLE C5.1 (continued)
Beam-to-Column Fully Restrained Connections

Connection	Description[a]	
Welded cover-plated flanges	The beam flanges are connected to the column flanges with existing CJP groove welds. The beam web is either bolted, with or without supplemental welds, or welded to a single-plate shear connection or welded directly to the column flange. A composite slab is or is not present. The existing beam flange welds shall meet the requirements of *Seismic Provisions* Section A3.4a. Alternatively, the existing beam flange welds shall be removed and replaced with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4b. The new cover plates shall be connected to the column flange with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4b. The cover plates are permitted to be welded or bolted to the beam.	
Improved WUF—bolted web (IWUF-B)	The beam flanges are connected to the column flanges with existing CJP groove welds. The beam web is bolted, with or without supplemental welds, to a single-plate shear connection, or directly welded to the column flange. A composite slab is or is not present. The existing CJP groove welds, connecting the beam flanges to the column, shall meet the requirements of *Seismic Provisions* Section A3.4a. Alternatively, the existing CJP groove welds shall be removed and replaced with CJP groove welds meeting the requirements of *Seismic Provisions* Section A3.4b.	
Welded flange plates	The beam flanges are connected to the flange plates with fillet welds; the beam flanges are not directly connected to the column. The flange plates are welded to the column with CJP groove welds. The beam web is either bolted, with or without supplemental welds, or welded to a single-plate shear connection, or the beam web is welded directly to the column flange. A composite slab is or is not present. The existing CJP groove welds, connecting the flange plates to the column, shall meet the requirements of *Seismic Provisions* Section A3.4a. Alternatively, the existing CJP groove welds shall be removed and replaced with welds meeting the requirements of *Seismic Provisions* Section A3.4b. The existing fillet welds of the flange plates to the beam flanges are permitted to remain unaltered.	
Bolted flange plate	The beam flanges are connected to the flange plates with bolts; the beam flanges are not connected to the column. The flange plates are welded to the column with CJP groove welds. The beam web is either bolted, with or without supplemental welds, or welded to a single-plate shear connection. A composite slab is or is not present. The connection shall be considered FR if the connection is stronger than the expected plastic flexural strength, $F_{ye}Z$, of the beam.	

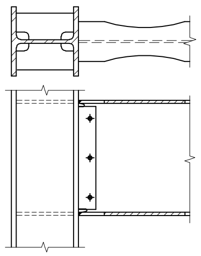
[a]Unless noted otherwise, existing web welds are permitted that are made with weld metal that does not have specified minimum notch toughness requirements. Existing web welds are permitted to remain unaltered, unless otherwise noted.

TABLE C5.1 (continued)
Beam-to-Column Fully Restrained Connections

Connection	Description[a]	
Double split-tee	The beam flanges are connected to the T-stubs with bolts; the beam flanges are not connected to the column. The T-stubs are bolted to the column. The beam web is either bolted, with or without supplemental welds, or welded to a single-plate shear connection. A composite slab is or is not present. This connection is permitted to be considered FR if it satisfies the strength and connection deformation requirements of Section C5.1a.	
Bolted end-plate moment connection in conformance with ANSI/AISC 358	This connection shall meet the requirements of ANSI/AISC 358, Chapter 6.	
Bolted flange plate (BFP) moment connection in conformance with ANSI/AISC 358	This connection shall meet the requirements of ANSI/AISC 358, Chapter 7.	
WUF—welded web (WUF-W) moment connection in conformance with ANSI/AISC 358	This connection shall meet the requirements of ANSI/AISC 358, Chapter 8.	
Double-tee moment connection in conformance with ANSI/AISC 358	This connection shall meet the requirements of ANSI/AISC 358, Chapter 13.	

[a]Unless noted otherwise, existing web welds are permitted that are made with weld metal that does not have specified minimum notch toughness requirements. Existing web welds are permitted to remain unaltered, unless otherwise noted.

TABLE C5.1 (continued)
Beam-to-Column Fully Restrained Connections

Connection	Description[a]	
Reduced beam section (RBS) moment connection in conformance with ANSI/AISC 358	This connection shall meet the requirements of ANSI/AISC 358, Chapter 5.	

[a]Unless noted otherwise, existing web welds are permitted that are made with weld metal that does not have specified minimum notch toughness requirements. Existing web welds are permitted to remain unaltered, unless otherwise noted.

User Note: Regardless of classification as fully restrained (FR) or partially restrained (PR), moment frames typically derive most of their inelastic lateral displacement through yielding of connection elements, including, for example, the beams near their joints with columns and column panel zones. The 10% contribution that serves as a measure of whether a connection is FR or PR does not consider such inelastic behavior. Rather, the 10% contribution applies to deformation during the elastic range of response, derived through such mechanisms as bending of tee flanges in double split-tee connections or bending of seat angles in top and bottom flange angle connections.

1b. Partially Restrained Beam-to-Column Moment Connections

Connections not meeting the requirements in Section C5.1a shall be classified as partially restrained (PR). Table C5.2 shall be used to identify the various connections for which permissible performance parameters are provided in Section C5. Modeling procedures, permissible performance parameters, and retrofit measures for moment frames with PR beam-to-column connections shall be as determined in Chapter D.

2. Stiffness

The stiffness of connections shall be based on principles of structural mechanics and as specified in the *Specification* unless superseded by supplemental provisions of this section or system-specific sections in Chapters D through I.

The force-deformation model for a connection shall account for all significant sources of deformation that affect its behavior, including those from axial, flexural, and shear actions.

TABLE C5.2
Beam-to-Column Partially Restrained Connections

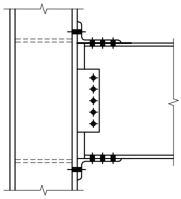
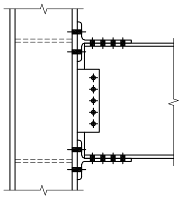
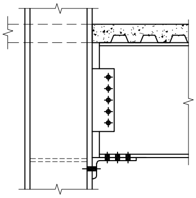
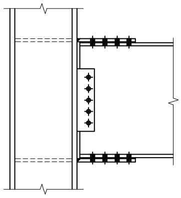
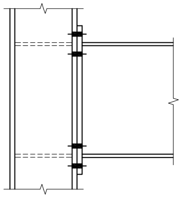
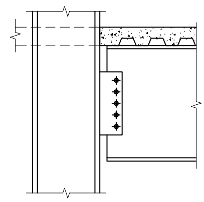
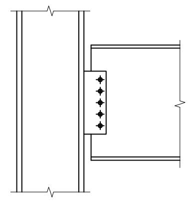
Connection	Description	
Top and bottom flange angle	Flange angles bolted or riveted to beam flanges and column flange without a composite slab.	
Double split-tee	Split-tees bolted or riveted to beam flanges and column flange.	
Composite top and flange angle bottom	Flange angle bolted or riveted to column flange and beam bottom flange with composite slab.	
Bolted flange plate	Flange plate with CJP groove welds at column and bolted to beam flanges	
Bolted end plate	Stiffened or unstiffened end plate welded to beam and bolted to column flange.	

TABLE C5.2 (continued)
Beam-to-Column Partially Restrained Connections

Connection	Description	
Shear connection with slab	Single-plate shear connection, composite slab.	
Shear connection without slab	Single-plate shear connection, no composite slab.	

User Note: Not all connections need to be explicitly or implicitly included in an analytical model. The engineer should use judgment based on principles of structural mechanics. For example, if the robustness of a column splice will prevent its behavior from contributing to the response of the adjacent columns, then it can be neglected and the column can be modeled as a continuous component from joint to joint.

2a. Beam-to-Column Connections

User Note: The provisions for rotationally restrained connections do not supersede provisions for panel zones, if applicable, provided in Section C4.

1. Fully Restrained (FR) Connections

Modeling of connection rotational stiffness for FR connections shall not be required except for connections that are intentionally reinforced to force formation of plastic hinges within the beam span, remote from the column face. For such connections, rigid elements shall be used between the column and the beam to represent the effective span of the beam.

2. Partially Restrained (PR) Connections

The moment-rotation behavior of each PR connection for use in modeling shall be determined by testing or analysis using the principles of structural mechanics. The deformation of the connection shall be included when calculating frame displacements.

In lieu of explicit connection modeling, it is permitted in linear analyses to adjust the flexural stiffness of a beam with PR connections, EI_b, to account for the flexibility of the end connections, as follows:

$$\left(EI_b\right)_{adjusted} = \frac{1}{\dfrac{6}{L_{CL}K_\theta} + \dfrac{1}{EI_b}} \tag{C5-1}$$

where

I_b = moment of inertia of the beam about the axis of bending, in.^{4} (mm^4)

K_θ = elastic stiffness of the partially restrained connection, kip-in./rad (N-mm/rad)

L_{CL} = centerline length of the beam taken between joints, in. (mm)

Where Equation C5-1 is used, the adjusted beam stiffness shall be used in frame analysis with FR connections, and the rotation of the connection shall be taken as the chord rotation of the beam.

2b. Column-to-Base Connections

The rotational stiffness, K_θ, of each base connection for use in modeling shall be determined by testing or analysis. The deformation of the column-to-base connection shall be included when determining frame displacements.

3. Strength

The axial, flexural, and shear strengths of a connection shall be determined in accordance with this section.

The strength of a connection shall be based on the controlling limit state considering all potential modes of failure.

The strength of bolts, rivets, and welds used in steel connections for a given deformation-controlled or force-controlled action shall be taken as the nominal strength for that action given in *Specification* Chapter J.

3a. Deformation-Controlled Actions

1. FR Beam-to-Column Moment Connections

The expected strengths for all applicable limit states for FR moment connections shall be determined in accordance with the procedures specified in the *Specification* or *Seismic Provisions*, testing, principles of structural mechanics, or the requirements of this section. Calculated expected strengths shall use expected material properties. Unless otherwise indicated in this section, the expected flexural strength, M_{CE}, of FR connections shall be determined as follows:

$$Q_{CE} = M_{CE} = F_{ye}Z_b \tag{C5-2}$$

where

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

(a) For welded unreinforced flange (WUF) (pre-1995) connections with beam depths of W24 (W610) and greater, M_{CE} shall be determined as follows:

$$Q_{CE} = M_{CE} = F_{ye} S_b \tag{C5-3}$$

where

S_b = elastic section modulus of beam, in.^{3} (mm^3)

(b) For reduced beam section connections, M_{CE} shall be determined in accordance with ANSI/AISC 358.

2. PR Beam-to-Column Moment Connections

The expected strengths for all applicable limit states for PR moment connections shall be determined in accordance with the procedures specified in the *Specification* or *Seismic Provisions*, testing, principles of structural mechanics, or the requirements of this section. Calculated expected strengths shall use expected material properties.

(a) For top and bottom flange angle connections

The expected flexural strength of a riveted or bolted flange angle connection, as shown in Figure C5.1, shall be the smallest value of M_{CE} based on the following limit states:

(i) If the expected shear strength of the rivet or bolt group connecting the horizontal leg of the flange angle to the beam flange controls, the expected flexural strength of the connection, M_{CE}, shall be determined as follows:

$$Q_{CE} = M_{CE} = \left(F_{ve} A_b N_b\right) d_b \tag{C5-4}$$

where

A_b = gross area of rivet or bolt, in.^{2} (mm^2)

F_{ve} = expected shear stress of bolt or driven rivet, taken as F_{nv}, ksi (MPa)

F_{nv} = nominal shear stress of bolt or driven rivet for bearing-type connections, given in *Specification* Section J3.7 or *Specification* Appendix 5, Section 5.3, ksi (MPa)

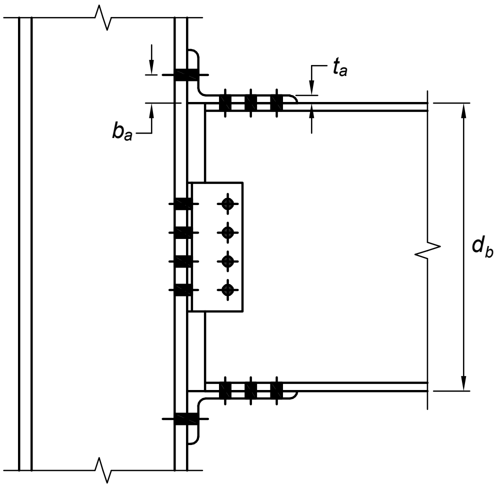


Fig. C5.1. Top and bottom flange angle connection.

N_b = least number of bolts or rivets connecting the top or bottom angle to the beam flange
d_b = depth of beam, in. (mm)

(ii) If the expected tensile strength of the horizontal leg of the flange angle controls, the expected flexural strength of the connection, M_{CE}, shall be determined as follows:

$$Q_{CE} = M_{CE} = P_{CE}\left(d_b + t_a\right) \tag{C5-5}$$

where P_{CE} is the expected tensile strength of the horizontal leg, governed by the gross or net section area, and shall be taken as the smaller value determined from Equations C5-6 and C5-7:

$$P_{CE} = F_{ye}A_g \tag{C5-6}$$

$$P_{CE} = F_{ue}A_e \tag{C5-7}$$

where
A_e = effective net area of horizontal angle leg, in.^{2} (mm^2)
A_g = gross area of horizontal angle leg, in.^{2} (mm^2)
t_a = thickness of angle, in. (mm)

(iii) If the expected tensile strength of the rivet or bolt group connecting the vertical leg of the flange angle to the column flange controls, the expected flexural strength, M_{CE}, of the connection shall be determined as follows:

$$Q_{CE} = M_{CE} = \left(F_{te}A_bN_b\right)\left(d_b + b_a\right) \tag{C5-8}$$

where
F_{te} = expected tensile stress of bolt or rivet, taken as F_{nt}, ksi (MPa)
F_{nt} = nominal tensile stress of bolt or driven rivet, given in *Specification* Section J3.7 or *Specification* Appendix 5, Section 5.3, ksi (MPa)
b_a = distance from the exterior flange face to the resultant tensile force of the bolt or rivet group, as shown in Figure C5.1, in. (mm)

The effect of prying on the rivet or bolt group connecting the vertical leg of the angle to the column flange shall also be considered.

(iv) If the expected flexural yielding of the flange angle controls, the expected flexural strength of the connection, M_{CE}, shall be determined as follows:

$$Q_{CE} = M_{CE} = \frac{wt_a{}^2F_{ye}}{2\left(b_a - \dfrac{t_a}{2}\right)}\left(d_b + b_a\right) \tag{C5-9}$$

where
w = length of flange angle, in. (mm)

(b) For double split-tee connections

The expected flexural strength of the double split-tee (T-stub) connection, as shown in Figure C5.2, shall be the smallest value of M_{CE} based on the following limit states:

(i) If the expected shear strength of the rivet or bolt group connecting the web of the split-tee to the beam flange controls, the expected flexural strength, M_{CE}, of the connection shall be determined using Equation C5-4.

(ii) If the expected tensile strength of the rivet or bolt group connecting the flange of the split-tee to the column flange controls, the expected flexural strength, M_{CE}, of the connection shall be determined as follows:

$$Q_{CE} = M_{CE} = \left(F_{te} A_b N_b\right)\left(d_b + t_s\right) \tag{C5-10}$$

where

N_b = least number of bolts or rivets connecting the flange of the top or bottom split-tee to the column flange

t_s = thickness of split-tee stem, in. (mm)

The effect of prying on the rivet or bolt group connecting the flange of the split-tee to the column flange shall also be considered.

(iii) If expected tensile strength of the stem of the split-tee controls, the expected flexural strength, M_{CE}, of the connection shall be determined using Equation C5-5, where t_s is substituted for t_a, and A_g and A_e are taken as the gross area and effective net area of the split-tee stem, respectively, in Equations C5-6 and C5-7.

(iv) If the expected flexural yielding of the flange of the split-tee controls, the expected flexural strength, M_{CE}, of the connection shall be determined as follows:

$$Q_{CE} = M_{CE} = \frac{w t_f^{\,2} F_{ye}}{\left(b_t - \dfrac{t_s}{2}\right)}\left(d_b + t_s\right) \tag{C5-11}$$

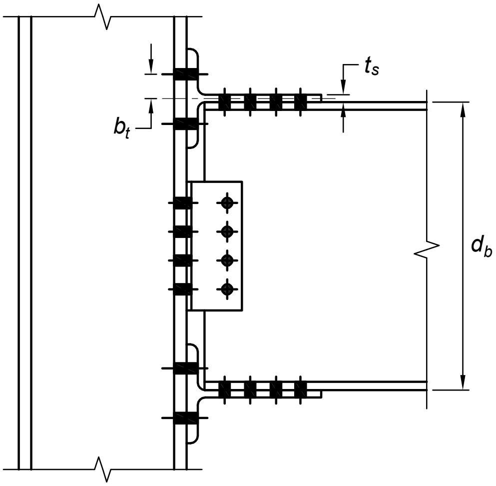


Fig. C5.2. Double split-tee connection.

where

b_t = distance between the nearest row of fasteners in the flange of the split-tee and the centerline of the split-tee stem, as shown in Figure C5.2, in. (mm)

t_f = thickness of flange of the split-tee, in. (mm)

w = length of split-tee, in. (mm)

(c) For bolted flange-plate connections

For bolted flange-plate connections, as shown in Figure C5.3, the flange plate shall be welded to the column and welded or bolted to the beam flange. This connection shall be considered fully restrained if its expected flexural strength equals or exceeds the expected flexural strength of the connected beam.

If the expected tensile strength of the flange plate controls, the expected flexural strength, M_{CE}, of the connection shall be determined using Equation C5-5, where the thickness of flange plate, t_p, is substituted for t_a, and A_g and A_e are taken as the gross area and effective net area of flange plate, respectively.

Similar to top and bottom flange angle and double split-tee connections, the expected flexural strength of the connection shall be determined when the welds or bolt group connecting the flange plate to the beam flange control over the tensile strength of the flange plate. The expected strength of the welds shall be taken as the nominal stress of the weld metal, F_{nw}, given in *Specification* Section J2.

(d) For bolted end-plate connections

Bolted end-plate connections, as shown in Figure C5.4, shall be considered fully restrained if the expected flexural strength equals or exceeds the expected flexural strength of the connected beam.

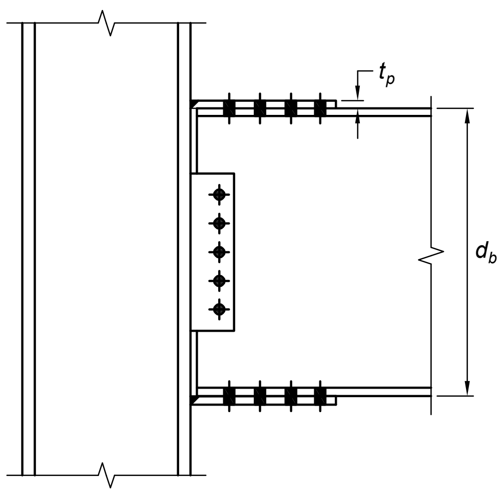


Fig. C5.3. Bolted flange-plate connection.

Applicable limit states for bolted end-plate connections shall be determined in accordance with the procedures of the *Specification*, *Seismic Provisions*, or by another procedure approved by the authority having jurisdiction (AHJ).

The expected flexural strength, M_{CE}, shall be determined for the limit state of flexural yielding of the end plate or the limit state of bolt rupture, subject to combined tension and shear actions.

(e) For composite PR connections

The expected strength for composite PR connections shall be derived from testing or analysis in accordance with ASCE/SEI 41, Section 7.6.

3. Column-to-Base Connections

The expected strengths for all applicable limit states for column-to-base connections shall be determined in accordance with the procedures specified in the *Specification* or *Seismic Provisions*, testing, principles of structural mechanics, or the requirements of this section. Calculated expected strengths shall use expected material properties.

3b. Force-Controlled Actions

The lower-bound strength of applicable limit states controlled by bolt, rivet, or weld failure computed using only the nominal strength of the bolt, rivet, or weld given in *Specification* Chapter J shall be multiplied by 0.85.

1. FR and PR Beam-to-Column Moment Connections

The lower-bound strengths for all applicable limit states for FR and PR moment connections shall be based on procedures listed in Sections C5.3a.1 and C5.3a.2 using lower-bound material properties instead of expected material properties.

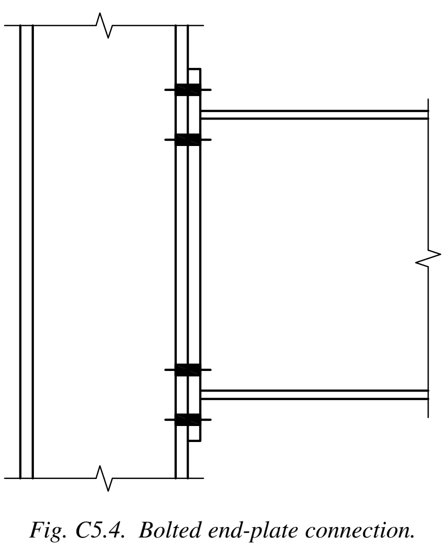


Fig. C5.4. Bolted end-plate connection.

2. Column-to-Base Connections

The lower-bound strengths for all applicable limit states for column-to-base connections shall be based on procedures listed in Section C5.3a.3 using lower-bound material properties instead of expected material properties.

3. Column and Beam Splices

The lower-bound strengths for all applicable limit states for column and beam splices shall be determined in accordance with procedures specified in the *Specification* or *Seismic Provisions*, testing, principles of structural mechanics, or the requirements of this section. Calculated lower-bound strengths shall use lower-bound material properties.

Yielding of the base metal shall be considered a deformation-controlled action based on permissible strengths or deformations given for the beam or column gross section.

Actions on groove welds in column or beam splices shall be considered force-controlled actions.

a. With Complete-Joint-Penetration Groove Welded Splices

The lower-bound tensile strength of splices made with CJP groove welds for a given action shall be determined in accordance with procedures given in the *Specification* for nominal strength, except that F_{yL} shall be substituted for F_y.

b. With Partial-Joint-Penetration Groove Welded Splices

The lower-bound tensile strength of splices made with partial-joint-penetration (PJP) groove welds, σ_{cr}, for a given action shall be determined in accordance with Equation C5-12. Weld stress demand on the splice, σ_{UF}, shall be determined as the maximum stress in the smaller section at the end of the PJP groove weld or in accordance with Equation C5-14. The demand, σ_{UF}, shall not exceed the lower-bound strength, σ_{cr}, determined as follows:

$$\sigma_{cr} = \frac{K_{IC}}{\left[F\left(\frac{a_0}{t_{f,u}}\right)\right]\sqrt{\pi a_0}} \le F_{ue}\left(1 - \frac{a_0}{t_{f,u}}\right) \le F_{ye} \tag{C5-12}$$

where

$$F\left(\frac{a_0}{t_{f,u}}\right) = \left(2.3 - 1.6\frac{a_0}{t_{f,u}}\right)\left(4.6\frac{a_0}{t_{f,u}}\right) \tag{C5-13}$$

K_{IC} = fracture toughness parameter in accordance with Table C5.3 or by other approved methods, ksi$\sqrt{\text{in.}}$ (MPa$\sqrt{\text{mm}}$). If the Charpy V-notch toughness is not known, it is permitted to use the value for 7 ft-lb (9.5 J).

a_0 = dimension of the smaller flange or web thickness that is not welded, including any applicable loss, in. (mm)

$t_{f,u}$ = thickness of the smaller flange or web, in. (mm)

TABLE C5.3
Fracture Toughness Parameter,
K_{IC}, for Steel

Charpy V-Notch at Lowest Anticipated Service Temperature (LAST), ft-lb (J)	K_{IC}, ksi$\sqrt{\text{in.}}$ $\left(\text{MPa}\sqrt{\text{mm}}\right)$
5 (6.8)	50 (1 700)
10 (14)	100 (3 500)
20 (27)	185 (6 400)
40 (54)	300 (10 000)

$$\sigma_{UF} = \left(\frac{P_{UF}}{A_g}\right) \pm \left(\frac{M_{UF,x}}{S_x}\right) \pm \left(\frac{M_{UF,y}}{S_y}\right) \tag{C5-14}$$

where

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

S_x = elastic section modulus of the smaller member taken about the x-axis, in.^{3} (mm^3)

S_y = elastic section modulus of the smaller member taken about the y-axis, in.^{3} (mm^3)

c. Bolted Splices

Actions on bolted splices shall be considered force-controlled. The lower-bound strengths of the splice shall be determined in accordance with procedures given in the *Specification* for nominal strengths using lower-bound material properties.

4. Permissible Performance Parameters

Permissible strengths and deformations for axial, flexural, and shear actions in a connection shall be computed in accordance with this section.

4a. Deformation-Controlled Actions

1. Beam-to-Column Connections

a. FR Beam-to-Column Moment Connections

1. Linear Analysis Procedures

For linear analysis procedures, flexural behavior of FR connections identified in Table C5.4 (Table C5.4M) shall be considered deformation-controlled. The expected flexural strength, $Q_{CE} = M_{CE}$, shall be determined in accordance with Section C5.3a.1 and m shall be taken from Table C5.4 (Table C5.4M) as modified in this section. Actions for limit states for which no values for m are provided shall be considered force-controlled.

TABLE C5.4
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—FR Beam-to-Column Connections Subjected to Flexure[a]

Component[c]	IO	Primary Component LS	Primary Component CP	Secondary Component LS	Secondary Component CP
WUF (pre-1995)[b]	1.0	$3.3 - 0.06d_b$	$4.4 - 0.08d_b$	$d_b < 24$ in., 2.8 $d_b \geq 24$ in., 2.0	$d_b < 24$ in., 3.7 $d_b \geq 24$ in., 2.6
Bottom haunch with weld at top flange conforming to *Seismic Provisions* Section A3.4	1.5	2.1	2.9	3.2	4.3
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and composite slab	1.5	2.0	2.7	2.5	3.4
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and a noncomposite slab or no slab	1.1	1.6	2.1	2.1	2.9
Welded top and bottom haunch	2.4	3.1	3.9	4.7	6.0
Welded cover plate in WUF with existing flange weld remaining[b]	$3.9 - 0.059d_b$	$4.3 - 0.067d_b$	$5.4 - 0.090d_b$	$5.4 - 0.090d_b$	$6.9 - 0.118d_b$
Welded cover-plated flanges	2.5	2.8	3.4	3.4	4.2
Improved WUF—bolted web[b]	$2.0 - 0.016d_b$	$2.3 - 0.021d_b$	$3.1 - 0.032d_b$	$4.9 - 0.048d_b$	$6.2 - 0.065d_b$
Welded Flange Plates					
1. Flange plate net section	2.5	3.3	4.1	5.7	7.3
2. Other limit states	Force-controlled				
All ANSI/AISC 358 conforming connections, with the exception of the RBS moment connection[b][d]	$38X_1 \leq 2.3$	$56X_1 \leq 3.4$	$75X_1 \leq 4.5$	4.5	6.0
RBS moment connection in conformance with ANSI/AISC 358[b][e]	$38X_2 \leq 2.3$	$56X_2 \leq 3.4$	$75X_2 \leq 4.5$	4.5	6.0

CP = collapse prevention performance level as defined in ASCE/SEI 41, Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41, Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41, Chapter 2

[a]Tabulated values shall be modified as indicated in Section C5.4a.1.a.1(a through d).
[b]Where values of m are a function of d_b, they need not be taken as less than 1.0.
[c]Refer to Table C5.1 for description of the connection.

[d] $$X_1 = 0.3\left(\frac{h}{t_w}\right)^{-0.3}\left(\frac{b_f}{2t_f}\right)^{-1.7}\left(\frac{L_b}{r_y}\right)^{-0.2}\left(\frac{L}{d}\right)^{1.1}$$

[e] $$X_2 = 0.55\left(\frac{h}{t_w}\right)^{-0.5}\left(\frac{b_f}{2t_f}\right)^{-0.7}\left(\frac{L_b}{r_y}\right)^{-0.5}\left(\frac{L}{d}\right)^{0.8}$$

where
L = length of span, in.
L_b = length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in.
d_b = depth of beam, in.
The values for L, L_b, b_f, d, h, t_f, t_w, and r_y all pertain to the connected beam.

TABLE C5.4M
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—FR Beam-to-Column Connections Subjected to Flexure[a]

Component[c]	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
WUF (pre-1995)[b]	1.0	$3.3 - 0.0024d_b$	$4.4 - 0.0031d_b$	$d_b < 600$mm, 2.8 $d_b \geq 600$mm, 2.0	$d_b < 600$mm, 3.7 $d_b \geq 600$mm, 2.6
Bottom haunch with weld at top flange conforming to *Seismic Provisions* Section A3.4	1.5	2.1	2.9	3.2	4.3
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and composite slab	1.5	2.0	2.7	2.5	3.4
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and a noncomposite slab or no slab	1.1	1.6	2.1	2.1	2.9
Welded top and bottom haunch	2.4	3.1	3.9	4.7	6.0
Welded cover plate in WUF with existing flange weld remaining[b]	$3.9 - 0.0023d_b$	$4.3 - 0.0026d_b$	$5.4 - 0.0035d_b$	$5.4 - 0.0035d_b$	$6.9 - 0.0046d_b$
Welded cover-plated flanges	2.5	2.8	3.4	3.4	4.2
Improved WUF—bolted web[b]	$2.0 - 0.00063d_b$	$2.3 - 0.00083d_b$	$3.1 - 0.0013d_b$	$4.9 - 0.0019d_b$	$6.2 - 0.0026d_b$
Welded flange plates					
1. Flange plate net section	2.5	3.3	4.1	5.7	7.3
2. Other limit states	Force-controlled				
All ANSI/AISC 358 conforming connections, with the exception of the RBS moment connection[b][d]	$38X_1 \leq 2.3$	$56X_1 \leq 3.4$	$75X_1 \leq 4.5$	4.5	6.0
RBS moment connection in conformance with ANSI/AISC 358[b] [e]	$38X_2 \leq 2.3$	$56X_2 \leq 3.4$	$75X_2 \leq 4.5$	4.5	6.0

[a]Tabulated values shall be modified as indicated in Section C5.4a.1.a.1(a through d).
[b]Where values of m are a function of d_b, they need not be taken as less than 1.0.
[c]Refer to Table C5.1 for description of the connection.

[d] $$X_1 = 0.3\left(\frac{h}{t_w}\right)^{-0.3}\left(\frac{b_f}{2t_f}\right)^{-1.7}\left(\frac{L_b}{r_y}\right)^{-0.2}\left(\frac{L}{d}\right)^{1.1}$$

[e] $$X_2 = 0.55\left(\frac{h}{t_w}\right)^{-0.5}\left(\frac{b_f}{2t_f}\right)^{-0.7}\left(\frac{L_b}{r_y}\right)^{-0.5}\left(\frac{L}{d}\right)^{0.8}$$

where
L = length of span, mm
L_b = length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm
d_b = depth of beam, mm
The values for L, L_b, b_f, d, h, t_f, t_w, and r_y all pertain to the connected beam.

Tabulated values for m in Table C5.4 (Table C5.4M) shall be modified as determined by the following conditions. The modifications shall be cumulative, but the resulting value for m need not be taken as less than 1.0.

User Note: The permissible flexural strength of FR beam-to-column moment connections is dependent on the detailing of continuity plates, the strength of the panel zone, the beam span-to-depth ratio, and the slenderness of the beam web and flanges.

(a) If the connection does not satisfy at least one of the following three conditions, the tabulated value for m in Table C5.4 (Table C5.4M) shall be multiplied by 0.8.

(i)
$$t_{cf} \geq \frac{b_{bf}}{5.2} \tag{C5-15}$$

or

(ii)
$$\frac{b_{bf}}{7} \leq t_{cf} < \frac{b_{bf}}{5.2} \tag{C5-16}$$

and

$$t \geq \frac{t_{bf}}{2} \tag{C5-17}$$

or

(iii)
$$t_{cf} < \frac{b_{bf}}{7} \tag{C5-18}$$

and

$$t > t_{bf} \tag{C5-19}$$

where

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

t = thickness of continuity plate, in. (mm)

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

(b) If the following condition is not met, the tabulated value for m in Table C5.4 (Table C5.4M) shall be multiplied by 0.8. V_{ye} shall be determined in accordance with Section C4.3a.

$$0.6 \leq \frac{V_{PZ}}{V_{ye}} \leq 0.9 \tag{C5-20}$$

where

V_{PZ} = panel-zone shear at the development of a hinge (expected first yield) at the critical location of the connection, kips (N)

For M_y at the face of the column, V_{PZ} is permitted to be estimated using Equation C5-21:

$$V_{PZ} = \frac{\sum M_{y(beam)}}{d_b}\left(\frac{L_{CL}}{L_{CL} - d_c}\right)\left(\frac{h_{avg} - d_b}{h_{avg}}\right) \tag{C5-21}$$

where

$M_{y(beam)}$ = expected first yield moment of the beam, kip-in. (N-mm)
= SF_{ye}

d_c = depth of column, in. (mm)

h_{avg} = average story height of columns above and below panel zone, in. (mm)

(c) If the beam flange and web slenderness satisfy the following conditions, the tabulated value for m in Table C5.4 (Table C5.4M) need not be modified.

$$\frac{b_f}{2t_f} \le 0.31\sqrt{\frac{E}{F_{ye}}} \tag{C5-22}$$

and

$$\frac{h}{t_w} \le 2.45\sqrt{\frac{E}{F_{ye}}} \tag{C5-23}$$

If the beam flange or web slenderness satisfy the following conditions, the tabulated value for m in Table C5.4 (Table C5.4M) shall be multiplied by 0.5.

$$\frac{b_f}{2t_f} > 0.38\sqrt{\frac{E}{F_{ye}}} \tag{C5-24}$$

or

$$\frac{h}{t_w} > 3.76\sqrt{\frac{E}{F_{ye}}} \tag{C5-25}$$

Straight-line interpolation, based on the case that results in the lower modifier, shall be used for intermediate values of beam flange or web slenderness,

where

b_f = width of flange, in. (mm)

h = for rolled shapes, the clear distance between flanges less the fillet or corner radii; for built-up welded sections, the clear distance between flanges; for built-up bolted sections, the distance between fastener lines, in. (mm)

t_f = thickness of flange, in. (mm)

t_w = thickness of web, in. (mm)

(d) If the clear span-to-depth ratio, L_{cf}/d_b, is less than 8, the tabulated value for m in Table C5.4 (Table C5.4M) shall be multiplied by the factor

$$0.5^{\left[\frac{(8-L_{cf}/d_b)}{3}\right]}$$

where

L_{cf} = length of beam taken as the clear span between column flanges, in. (mm).

FR connections designed to promote yielding of the beam remote from the column face shall be considered force-controlled for flexure and shall satisfy Equation C5-26:

$$M_{CLc} \geq M_{peb} \tag{C5-26}$$

where

M_{CLc} = lower-bound flexural strength of connection at the face of the column, determined in accordance with Section C5.3b.1, kip-in. (N-mm)

M_{peb} = expected plastic flexural strength of beam, determined in accordance with Section C2.3a at the plastic hinge location, projected to the face of column, kip-in. (N-mm)

2. Nonlinear Analysis Procedures

For nonlinear analysis procedures, flexural behavior of FR connections identified in Table C5.5 (Table C5.5M) shall be considered deformation-controlled, and the plastic rotation angle, θ_p, predicted by analysis shall be not greater than the permissible plastic rotation angle given in Table C5.5 (Table C5.5M) as modified in this section.

Tabulated deformations in Table C5.5 (Table C5.5M) shall be modified in accordance with the conditions set forth in Sections C5.4a.1.a.1(a), (b), (c), and (d). The modifications shall be cumulative.

> **User Note:** The permissible flexural strength of FR beam-to-column moment connections is dependent on the detailing of continuity plates, the strength of the panel zone, the beam span-to-depth ratio, and the slenderness of the beam web and flanges.

FR connections designed to promote yielding of the beam remote from the column face shall be considered force-controlled for flexure and shall satisfy Equation C5-26.

b. PR Beam-to-Column Moment Connections

1. Linear Analysis Procedures

For linear analysis procedures, the flexural behavior of PR connections identified in Table C5.6 (Table C5.6M) shall be considered deformation-controlled. The expected flexural strength, $Q_{CE} = M_{CE}$, shall be determined in accordance with Section C5.3a.2 and m shall be taken from Table C5.6 (Table C5.6M). Actions for limit states for which no values for m are provided shall be considered force-controlled.

2. Nonlinear Analysis Procedures

For nonlinear analysis procedures, the flexural behavior of PR connections identified in Table C5.7 (Table C5.7M) shall be considered deformation-controlled, and the plastic rotation angle, θ_p, predicted by analysis shall be not greater than the permissible plastic rotation angle given in Table C5.7 (Table C5.7M).

TABLE C5.5
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—FR Beam-to-Column Connections Subjected to Flexure[a]

Component[c]	Modeling Parameters			Expected Deformation Capacity		
	Plastic Rotation Angle, rad		Residual Strength Ratio	Plastic Rotation Angle, rad		
	a	*b*	*c*	IO	LS	CP
WUF (pre-1995)[b]	$0.048 - 0.0011d_b$	$d_b < 24$ in., 0.04 $d_b \geq 24$ in., 0.025	0.2	$0.024 - 0.0006d_b \geq 0$	$d_b < 24$ in., 0.03 $d_b \geq 24$ in., 0.019	$d_b < 24$ in., 0.04 $d_b \geq 24$ in., 0.025
Bottom haunch with weld at top flange conforming to *Seismic Provisions* Section A3.4	0.028	0.047	0.2	0.014	0.035	0.047
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and composite slab	0.025	0.035	0.2	0.012	0.026	0.035
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and a noncomposite slab or no slab	0.018	0.028	0.2	0.09	0.021	0.028
Welded top and bottom haunches	0.028	0.048	0.2	0.014	0.0360	0.048
Welded cover plate in WUF with existing flange weld remaining[b]	$0.056 - 0.0011d_b$	$0.056 - 0.0011d_b$	0.2	$0.028 - 0.00055d_b$	$0.0420 - 0.00083d_b$	$0.056 - 0.0011d_b$
Welded cover-plated flanges	0.031	0.031	0.2	0.016	0.0233	0.031
Improved WUF—bolted web[b]	$0.021 - 0.00030d_b$	$0.050 - 0.00060d_b$	0.2	$0.010 - 0.00015d_b$	$0.0375 - 0.00045d_b$	$0.050 - 0.00060d_b$

[a]Values are applicable at the column face. Tabulated values shall be modified as indicated in Section C5.4a.1.a.1(a through d).
[b]Where plastic rotations are a function of d_b, they need not be taken as less than 0.0.
[c]Refer to Table C5.1 for description of the connection.

TABLE C5.5 (continued)
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—FR Beam-to-Column Connections Subjected to Flexure[a]

Component[c]	Modeling Parameters			Expected Deformation Capacity		
	Plastic Rotation Angle, rad		Residual Strength Ratio	Plastic Rotation Angle, rad		
	a	*b*	*c*	IO	LS	CP
Welded flange plates						
1. Flange plate net section	0.03	0.06	0.2	0.015	0.0450	0.06
2. Other limit states	Force-controlled					
All ANSI/AISC 358 conforming connections, with the exception of the RBS moment connection[b][d]	$X_1 \le 0.07$	0.07	0.3	0.5*a*	0.75*b*	*b*
RBS moment connection in conformance with ANSI/AISC 358[b][e]	$X_2 \le 0.07$	0.07	0.3	0.5*a*	0.75*b*	*b*

[a]Values are applicable at the column face. Tabulated values shall be modified as indicated in Section C5.4a.1.a.1(a through d).
[b]Where plastic rotations are a function of d_b, they need not be taken as less than 0.0.
[c]Refer to Table C5.1 for description of the connection.

[d]$X_1 = 0.3\left(\frac{h}{t_w}\right)^{-0.3}\left(\frac{b_f}{2t_f}\right)^{-1.7}\left(\frac{L_b}{r_y}\right)^{-0.2}\left(\frac{L}{d}\right)^{1.1}$

[e]$X_2 = 0.55\left(\frac{h}{t_w}\right)^{-0.5}\left(\frac{b_f}{2t_f}\right)^{-0.7}\left(\frac{L_b}{r_y}\right)^{-0.5}\left(\frac{L}{d}\right)^{0.8}$

4b. Force-Controlled Actions

1. All Connections

a. Linear Analysis Procedures

When linear analysis procedures are used and the flexural, shear, or axial behavior of a connection is considered force-controlled, the lower-bound component strength, Q_{CL}, of the connection shall be determined in accordance with Section C5.3b.

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the flexural, shear, or axial behavior of a connection is considered force-controlled, Q_{CL} of the connection shall be determined in accordance with Section C5.3b.

TABLE C5.5M
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—FR Beam-to-Column Connections Subjected to Flexure[a]

Component[c]	Modeling Parameters			Expected Deformation Capacity		
	Plastic Rotation Angle, rad		Residual Strength Ratio	Plastic Rotation Angle, rad		
	a	*b*	*c*	IO	LS	CP
WUF (pre-1995)[b]	$0.048 - 0.000043d_b$	$d_b < 600$ mm, 0.04 $d_b \geq 600$ mm, 0.025	0.2	$0.024 - 0.000024d_b \geq 0$	$d_b < 600$ mm, 0.03 $d_b \geq 600$ mm, 0.019	$d_b < 600$ mm, 0.04 $d_b \geq 600$ mm, 0.025
Bottom haunch with weld at top flange conforming to *Seismic Provisions* Section A3.4	0.028	0.047	0.2	0.014	0.035	0.047
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and composite slab	0.025	0.035	0.2	0.012	0.026	0.035
Bottom haunch with welds at top and bottom flanges that do not conform to *Seismic Provisions* Section A3.4 and a non-composite slab or no slab	0.018	0.028	0.2	0.09	0.021	0.028
Welded top and bottom haunches	0.028	0.048	0.2	0.014	0.0360	0.048
Welded cover plate in WUF with existing flange weld remaining[b]	$0.056 - 0.000043d_b$	$0.056 - 0.000043d_b$	0.2	$0.028 - 0.000022d_b$	$0.0420 - 0.000033d_b$	$0.056 - 0.000043d_b$
Welded cover-plated flanges	0.031	0.031	0.2	0.016	0.0233	0.031
Improved WUF—bolted web[b]	$0.021 - 0.000012d_b$	$0.050 - 0.000024d_b$	0.2	$0.010 - 0.0000059d_b$	$0.0375 - 0.000018d_b$	$0.050 - 0.000024d_b$

[a] Values are applicable at the column face. Tabulated values shall be modified as indicated in Section C5.4a.1.a.1 (a through d).
[b] Where plastic rotation angles are a function of d_b, they need not be taken as less than 0.0.
[c] Refer to Table C5.1 for description of the connection.

TABLE C5.5M (continued) Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—FR Beam-to-Column Connections Subjected to Flexure[a]

	Modeling Parameters			Expected Deformation Capacity		
	Plastic Rotation Angle, rad		Residual Strength Ratio	Plastic Rotation Angle, rad		
Component[c]	***a***	***b***	***c***	**IO**	**LS**	**CP**
Welded flange plates						
1. Flange plate net section	0.03	0.06	0.2	0.015	0.0450	0.06
2. Other limit states	Force-controlled					
All ANSI/AISC 358 conforming connections, with the exception of the RBS moment connection[b][d]	$X_1 \le 0.07$	0.07	0.3	0.5*a*	0.75*b*	*b*
RBS moment connection in conformance with ANSI/AISC 358[b][e]	$X_2 \le 0.07$	0.07	0.3	0.5*a*	0.75*b*	*b*

[a]Values are applicable at the column face. Tabulated values shall be modified as indicated in Section C5.4a.1.a.1 (a through d).
[b]Where plastic rotation angles are a function of d_b, they need not be taken as less than 0.0.
[c]Refer to Table C5.1 for description of the connection.
[d]$X_1 = 0.3\left(\frac{h}{t_w}\right)^{-0.3}\left(\frac{b_f}{2t_f}\right)^{-1.7}\left(\frac{L_b}{r_y}\right)^{-0.2}\left(\frac{L}{d}\right)^{1.1}$
[e]$X_2 = 0.55\left(\frac{h}{t_w}\right)^{-0.5}\left(\frac{b_f}{2t_f}\right)^{-0.7}\left(\frac{L_b}{r_y}\right)^{-0.5}\left(\frac{L}{d}\right)^{0.8}$

2. FR and PR Beam-to-Column Moment Connections

FR and PR connections shall meet the requirements of Section C5.4b.1.

The upper-bound beam flexural strength shall be used to determine the required connection strength. The upper-bound flexural strength shall be taken as M_{pe}.

3. Column-to-Base Connections

Column-to-base connections shall meet the requirements of Section C5.4b.1.

The upper-bound flexural strength of column-to-base connections shall be included under the range of conditions considered. The upper-bound flexural strength of column-to-base connections shall include the effective flexural resistance resulting from the compressive load in the column, and shall assume the development of full tension strength of the anchor rods.

TABLE C5.6
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—PR Beam-to-Column Connections Subjected to Flexure

Component	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
Top and Bottom Flange Angle[b]					
1. Shear failure of rivet or bolt (limit state 1)[c]	1.5	4	6	6	8
2. Tension failure of horizontal leg of angle (limit state 2)	1.25	1.5	2	1.5	2
3. Tension failure of rivet or bolt (limit state 3)[c]	1.25	1.5	2.5	4	4
4. Flexural failure of angle (limit state 4)	2	5	7	7	14
Double Split-Tee[b]					
1. Shear failure of rivet or bolt (limit state 1)[c]	1.5	4	6	6	8
2. Tension failure of rivet or bolt (limit state 2)[c]	1.25	1.5	2.5	4	4
3. Tension failure of split-tee stem (limit state 3)	1.25	1.5	2	1.5	2
4. Flexural failure of split-tee (limit state 4)	2	5	7	7	14
Bolted Flange Plate[b]					
1. Failure in net section of flange plate or shear failure of bolts or rivets[c]	1.5	4	5	4	5
2. Weld failure or tension failure on gross section of plate	1.25	1.5	2	1.5	2
Bolted End Plate					
1. Yield of end plate	2	5.5	7	7	7
2. Yield of bolts	1.5	2	3	4	4
3. Failure of weld	1.25	1.5	2	3	3

[a] d_{bg} = depth of bolt group, in.
[b] Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If $d_b > 18$ in. multiply m by $18/d_b$, but values need not be less than 1.0.
[c] For high-strength bolts, divide values by 2.0, but values need not be less than 1.25.

TABLE C5.6 (continued)
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—PR Beam-to-Column Connections Subjected to Flexure

Component	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
Composite Top with Bottom Flange Angle[b]					
1. Failure of deck reinforcement	1.25	2	3	4	6
2. Local flange yielding and web crippling of column	1.5	4	6	5	7
3. Yield of bottom flange angle	1.5	4	6	6	7
4. Tensile yield of rivets or bolts at column flange	1.25	1.5	2.5	2.5	3.5
5. Shear yield of beam flange connections	1.25	2.5	3.5	3.5	4.5
Shear Connection with Slab[a]	$2.4 - 0.011d_{bg}$	–	–	$13.0 - 0.290d_{bg}$	$17.0 - 0.387d_{bg}$
Shear Connection without Slab[a]	$8.9 - 0.193d_{bg}$	–	–	$13.0 - 0.290d_{bg}$	$17.0 - 0.387d_{bg}$

[a]d_{bg} = depth of bolt group, in.
[b]Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If $d_b > 18$ in. multiply m by $18/d_b$, but values need not be less than 1.0.
[c]For high-strength bolts, divide values by 2.0, but values need not be less than 1.25.

5. Anchorage to Concrete

Connections of steel components to concrete components shall comply with the requirements of these Provisions and ASCE/SEI 41, Chapter 10, for classification of actions as deformation-controlled or force-controlled, and determination of associated strengths.

The capacity of connections between steel components and concrete components shall be the lowest value determined for the limit states of strength of the steel components, strength of connection plates, and strength of anchor rods and their embedment in the concrete.

The capacity of column-to-base connections shall be the lowest strength determined based on the following limit states: strength of welds or anchor rods, bearing strength of the concrete, and yield strength of the base plate.

TABLE C5.6M
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—PR Beam-to-Column Connections Subjected to Flexure

Component	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
Top and Bottom Flange Angle[b]					
1. Shear failure of rivet or bolt (limit state 1)[c]	1.5	4	6	6	8
2. Tension failure of horizontal leg of angle (limit state 2)	1.25	1.5	2	1.5	2
3. Tension failure of rivet or bolt (limit state 3)[c]	1.25	1.5	2.5	4	4
4. Flexural failure of angle (limit state 4)	2	5	7	7	14
Double Split-Tee[b]					
1. Shear failure of rivet or bolt (limit state 1)[c]	1.5	4	6	6	8
2. Tension failure of rivet or bolt (limit state 2)[c]	1.25	1.5	2.5	4	4
3. Tension failure of split-tee stem (limit state 3)	1.25	1.5	2	1.5	2
Bolted Flange Plate[b]					
1. Failure in net section of flange plate or shear failure of bolts or rivets[c]	1.5	4	5	4	5
2. Weld failure or tension failure on gross section of plate	1.25	1.5	2	1.5	2
Bolted End Plate					
1. Yield of end plate	2	5.5	7	7	7
2. Yield of bolts	1.5	2	3	4	4
3. Failure of weld	1.25	1.5	2	3	3

[a] d_{bg} = depth of bolt group, in.
[b] Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If $d_b > 450$ mm, multiply m by $450/d_b$, but values need not be less than 1.0.
[c] For high-strength bolts, divide values by 2.0, but values need not be less than 1.25.

TABLE C5.6M (continued)
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—PR Beam-to-Column Connections Subjected to Flexure

Component	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
Composite Top with Bottom Flange Angle[b]					
1. Failure of deck reinforcement	1.25	2	3	4	6
2. Local flange yielding and web crippling of column	1.5	4	6	5	7
3. Yield of bottom flange angle	1.5	4	6	6	7
4. Tensile yield of rivets or bolts at column flange	1.25	1.5	2.5	2.5	3.5
5. Shear yield of beam flange connections	1.25	2.5	3.5	3.5	4.5
Shear Connection with Slab[a]	$2.4 - 0.00043d_{bg}$	—	—	$13.0 - 0.011d_{bg}$	$17.0 - 0.015d_{bg}$
Shear Connection without Slab[a]	$8.9 - 0.0076d_{bg}$	—	—	$13.0 - 0.011d_{bg}$	$17.0 - 0.015d_{bg}$

[a]d_{bg} = depth of the bolt group (mm)
[b]Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If d_b > 450 mm, multiply m by $450/d_b$ but values need not be less than 1.0.
[c]For high-strength bolts, divide values by 2.0, but values need not be less than 1.25.

The capacity of anchor rod connections between column-to-base connections and concrete substrata shall be the lowest strength determined based on the following limit states: shear or tensile yield strength of the anchor rods, loss of bond between the anchor rods and the concrete, or failure of the concrete. Anchor rod strengths for each failure type or limit state shall be the nominal strengths determined in accordance with ACI 318 (or ACI 318M), or according to other procedures approved by the AHJ.

For base plate yielding, bolt yielding, and weld failure within a column-to-base connection, the value for m stipulated in this section based on the respective limit states for a PR end plate connection shall be used. Column-to-base connection limit states controlled by anchor rod failure modes governed by the concrete shall be considered a force-controlled action.

TABLE C5.7
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—PR Beam-to-Column Connections Subjected to Flexure

Component	Modeling Parameters			Expected Deformation Capacity		
	Plastic Rotation Angle, rad		Residual Strength Ratio	Plastic Rotation Angle, rad		
	a	b	c	IO	LS	CP
Top and Bottom Flange Angle[b]						
1. Shear failure of rivet or bolt (Limit State 1)[c]	0.036	0.048	0.200	0.008	0.030	0.040
2. Tension failure of horizontal leg of angle (Limit State 2)	0.012	0.018	0.800	0.003	0.010	0.015
3. Tension failure of rivet or bolt (Limit State 3)[c]	0.016	0.025	1.000	0.005	0.020	0.020
4. Flexural failure of angle (Limit State 4)	0.042	0.084	0.200	0.010	0.035	0.070
Double Split-Tee[b]						
1. Shear failure of rivet or bolt (Limit State 1)[c]	0.036	0.048	0.200	0.008	0.030	0.040
2. Tension failure of rivet or bolt (Limit State 2)[c]	0.016	0.024	0.800	0.005	0.020	0.020
3. Tension failure of split-tee stem (Limit State 3)	0.012	0.018	0.800	0.003	0.010	0.015
4. Flexural failure of split-tee (Limit State 4)	0.042	0.084	0.200	0.010	0.035	0.070
Bolted Flange Plate[b]						
1. Failure in net section of flange plate or shear failure of bolts or rivets[c]	0.030	0.030	0.800	0.008	0.020	0.025
2. Weld failure or tension failure on gross section of plate	0.012	0.018	0.800	0.003	0.010	0.015

[a] Where plastic rotations are a function of the depth of the bolt group, d_{bg}, they shall not be taken as less than 0.0.
[b] Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If beam depth, $d_b > 18$ in., multiply m by $18/d_b$.
[c] For high-strength bolts, divide values by 2.0.

TABLE C5.7 (continued)
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—PR Beam-to-Column Connections Subjected to Flexure

Component	Modeling Parameters			Expected Deformation Capacity		
	Plastic Rotation Angle, rad		Residual Strength Ratio	Plastic Rotation Angle, rad		
	a	*b*	*c*	IO	LS	CP
Bolted End Plate						
1. Yield of end plate	0.042	0.042	0.800	0.010	0.035	0.035
2. Yield of bolts	0.018	0.024	0.800	0.008	0.020	0.020
3. Failure of weld	0.012	0.018	0.800	0.003	0.015	0.015
Composite Top with Bottom Flange Angle[b]						
1. Failure of deck reinforcement	0.018	0.035	0.800	0.005	0.020	0.030
2. Local flange yielding and web crippling of column	0.036	0.042	0.400	0.008	0.025	0.035
3. Yield of bottom flange angle	0.036	0.042	0.200	0.008	0.025	0.035
4. Tensile yield of rivets or bolts at column flange	0.015	0.022	0.800	0.005	0.013	0.018
5. Shear yield of beam-flange connection	0.022	0.027	0.200	0.005	0.018	0.023
Shear Connection with Slab[a][d]	$0.029 - 0.00020d_{bg}$	$0.15 - 0.0036d_{bg}$	0.400	$0.014 - 0.00010d_{bg} \le \frac{g}{d_{max}} - 0.02$	$0.1125 - 0.0027d_{bg} \le \frac{g}{d_{max}} - 0.02$	$0.15 - 0.0036d_{bg} \le \frac{g}{d_{max}} - 0.02$
Shear Connection without Slab[a][d]	$0.15 - 0.0036d_{bg}$	$0.15 - 0.0036d_{bg}$	0.400	$0.075 - 0.0018d_{bg} \le \frac{g}{d_{max}} - 0.02$	$0.1125 - 0.0027d_{bg} \le \frac{g}{d_{max}} - 0.02$	$0.15 - 0.0036d_{bg} \le \frac{g}{d_{max}} - 0.02$

[a]Where plastic rotations are a function of the depth of the bolt group, d_{bg}, they shall not be taken as less than 0.0.
[b]Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If beam depth, $d_b > 18$ in., multiply m by $18/d_b$.
[c]For high-strength bolts, divide values by 2.0.
[d]d_{max} = larger of d_1 and d_2, in.
where
d_1 = vertical distance from center of bolt group to top of beam, in.
d_2 = vertical distance from center of bolt group to bottom of beam, in.
g = gap distance between the end of beam and face of column, in.

TABLE C5.7M
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—PR Beam-to-Column Connections Subjected to Flexure

Component	Modeling Parameters			Expected Deformation Capacity		
	Plastic Rotation Angle, rad		Residual Strength Ratio	Plastic Rotation Angle, rad		
	a	*b*	*c*	IO	LS	CP
Top and Bottom Flange Angle[b]						
1. Shear failure of rivet or bolt (Limit State 1)[c]	0.036	0.048	0.200	0.008	0.030	0.040
2. Tension failure of horizontal leg of angle (Limit State 2)	0.012	0.018	0.800	0.003	0.010	0.015
3. Tension failure of rivet or bolt (Limit State 3)[c]	0.016	0.025	1.000	0.005	0.020	0.020
4. Flexural failure of angle (Limit State 4)	0.042	0.084	0.200	0.010	0.035	0.070
Double Split-Tee[b]						
1. Shear failure of rivet or bolt (Limit State 1)[c]	0.036	0.048	0.200	0.008	0.030	0.040
2. Tension failure of rivet or bolt (Limit State 2)[c]	0.016	0.024	0.800	0.005	0.020	0.020
3. Tension failure of split-tee stem (Limit State 3)	0.012	0.018	0.800	0.003	0.010	0.015
4. Flexural failure of split-tee (Limit State 4)	0.042	0.084	0.200	0.010	0.035	0.070
Bolted Flange Plate[b]						
1. Failure in net section of flange plate or shear failure of bolts or rivets[c]	0.030	0.030	0.800	0.008	0.020	0.025
2. Weld failure or tension failure on gross section of plate	0.012	0.018	0.800	0.003	0.010	0.015

[a]Where plastic rotations are a function of the depth of the bolt group, d_{bg}, they shall not be taken as less than 0.0.
[b]Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If beam depth, $d_b > 450$ mm, multiply m by $450/d_b$.
[c]For high-strength bolts, divide values by 2.0.

TABLE C5.7M (continued) Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—PR Beam-to-Column Connections Subjected to Flexure

Component	Modeling Parameters			Expected Deformation Capacity		
	Plastic Rotation Angle, rad		Residual Strength Ratio	Plastic Rotation Angle, rad		
	a	*b*	*c*	IO	LS	CP
Bolted End Plate						
1. Yield of end plate	0.042	0.042	0.800	0.010	0.035	0.035
2. Yield of bolts	0.018	0.024	0.800	0.008	0.020	0.020
3. Failure of weld	0.012	0.018	0.800	0.003	0.015	0.015
Composite Top with Bottom Flange Angle[b]						
1. Failure of deck reinforcement	0.018	0.035	0.800	0.005	0.020	0.030
2. Local flange yielding and web crippling of column	0.036	0.042	0.400	0.008	0.025	0.035
3. Yield of bottom flange angle	0.036	0.042	0.200	0.008	0.025	0.035
4. Tensile yield of rivets or bolts at column flange	0.015	0.022	0.800	0.005	0.013	0.018
5. Shear yield of beam-flange connection	0.022	0.027	0.200	0.005	0.018	0.023
Shear Connection with Slab[a][d]	$0.029 - 0.0000079d_{bg}$	$0.15 - 0.00014d_{bg}$	0.400	$0.014 - 0.0000039d_{bg} \le \frac{g}{d_{max}} - 0.02$	$0.1125 - 0.00011d_{bg} \le \frac{g}{d_{max}} - 0.02$	$0.15 - 0.00014d_{bg} \le \frac{g}{d_{max}} - 0.02$
Shear Connection without Slab[a][d]	$0.15 - 0.00014d_{bg}$	$0.15 - 0.00014d_{bg}$	0.400	$0.075 - 0.000071d_{bg} \le \frac{g}{d_{max}} - 0.02$	$0.1125 - 0.00011d_{bg} \le \frac{g}{d_{max}} - 0.02$	$0.15 - 0.00014d_{bg} \le \frac{g}{d_{max}} - 0.02$

[a]Where plastic rotations are a function of the depth of the bolt group, d_{bg}, they shall not be taken as less than 0.0.
[b]Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If beam depth, $d_b > 450$ mm, multiply m by $450/d_b$.
[c]For high-strength bolts, divide values by 2.0.
[d]d_{max} = larger of d_1 and d_2 (mm)
where
d_1 = vertical distance from center of bolt group to the top of beam (mm)
d_2 = vertical distance from center of bolt group to the bottom of beam (mm)
g = gap distance between the end of beam and face of column (mm)

C6. STEEL PLATE USED AS SHEAR WALLS

1. General

The component characteristics of steel plate used as shear walls subject to seismic forces or deformations from shear action, with no concurrent axial action, shall be determined in accordance with this section. This section applies to steel plate shear walls, with web plates sufficiently thick or stiffened to prevent shear buckling, that primarily resist loads or deformations through shear strength and stiffness. This section does not apply to shear walls subject to shear buckling.

The shear behavior of a steel plate shear wall shall be designated as either deformation-controlled or force-controlled in accordance with Chapters D through I.

2. Stiffness

The stiffness of steel plate shear walls shall be based on principles of structural mechanics and as specified in the *Specification* unless superseded by supplemental provisions of this section or system-specific sections in Chapters D through I.

The force-deformation model for a steel plate shear wall shall account for all significant sources of deformation that affect its behavior, including those from axial, flexural, and shear actions.

2a. Flexural Stiffness

There are no additional requirements beyond those specified in Section C6.2.

2b. Axial Stiffness

There are no additional requirements beyond those specified in Section C6.2.

2c. Shear Stiffness

If the steel plate wall includes concrete encasement or backing, then the shear stiffness of the plate wall shall be determined using full composite action, including the effects of cracking, provided a mechanism exists that provides sufficient transfer and distribution of forces to the surrounding boundary elements.

It is permitted to analyze a steel plate shear wall using plane stress finite elements with beams and columns as horizontal and vertical boundary elements, respectively. The elastic shear stiffness of a steel plate wall with a web plate that is sufficiently thick or stiffened to avoid shear buckling, K_w, shall be determined in accordance with Equation C6-1 unless another method based on principles of structural mechanics is used:

$$K_w = \frac{Gat_w}{h} \tag{C6-1}$$

where

a = clear width of wall between vertical boundary elements, in. (mm)
h = clear height of wall between horizontal boundary elements, in. (mm)
t_w = thickness of steel plate shear wall, in. (mm)

User Note: Equation C6-1 does not account for the change in elastic stiffness for shear buckling of an unstiffened plate wall prior to achieving shear yielding, nor does it capture composite action with concrete. The equivalent elastic stiffness of a buckled wall or composite wall at yield should be determined based on principles of structural mechanics or analysis.

3. Strength

The shear strength of steel plate shear walls shall be determined in accordance with this section.

The shear strength of the concrete encasement or backing is permitted to be included in the shear strength of the steel plate wall provided a transfer mechanism exists that provides full composite action and distribution of forces to the surrounding boundary elements beyond the anticipated plastic deformations. Otherwise, the shear strength of a composite plate wall shall neglect the effect of the concrete.

3a. Deformation-Controlled Actions

The expected shear strength, V_{CE}, of a steel plate wall shall be determined using equations for nominal shear strength, V_n, given in *Specification* Chapter G, except that F_{ye} shall be substituted for F_y, and $Q_{CE} = V_{CE}$.

Alternatively, for an unstiffened plate wall, it is permitted to determine the expected shear strength using equations for nominal shear strength, V_n, given in *Seismic Provisions* Section F5, except that F_{ye} shall be substituted for F_y.

For plate walls expected to experience inelastic action through shear yielding, the wall shall have a web plate that is sufficiently thick or stiffened to prevent shear buckling, and $Q_{CE} = Q_y = V_{CE}$. Stiffener strength, stiffness, and spacing shall be in accordance with the requirements for beams or girders given in *Specification* Chapter G. In lieu of providing stiffeners, it is permitted to encase or back the plate wall in concrete; the expected shear strength is permitted to be computed taking h/t_w in *Specification* Chapter G equal to zero.

3b. Force-Controlled Actions

The lower-bound shear strength, V_{CL}, of a steel plate wall shall be determined using equations for nominal shear strength, V_n, given in *Specification* Chapter G, except that F_{yL} shall be substituted for F_y, and $Q_{CL} = V_{CL}$.

Alternatively, for an unstiffened plate wall, it is permitted to determine the lower-bound shear strength using equations for nominal shear strength, V_n, given in *Seismic Provisions* Section F5, except that F_{yL} shall be substituted for F_y.

4. Permissible Performance Parameters

Permissible strengths and deformations for shear actions in a steel plate shear wall shall be computed in accordance with this section. Values provided are applicable if the web plate is sufficiently thick, or if stiffeners, concrete encasement, or backing are provided to prevent shear buckling.

4a. Deformation-Controlled Actions

1. Shear Actions

a. Linear Analysis Procedures

When the linear analysis procedures are used and the shear behavior of a steel plate shear wall is considered deformation-controlled, the expected shear strength, $Q_{CE} = V_{CE}$, shall be determined in accordance with Section C6.3a and m taken from Table C6.1.

b. Nonlinear Analysis Procedures

When constructing the nonlinear force-deformation model for use in the nonlinear analysis procedures, the generalized force-deformation curve for shear behavior shown in Figure C1.1, with modeling parameters a, b, and c as given in Table C6.2, shall be used for steel plate shear walls. Alternatively, these relationships are permitted to be derived from testing or analysis. For steel plate shear walls, it is permitted to take α_h for shear action as 6% of the elastic slope. Further modification of the curve is permitted if a greater value of α_h is justified by testing or analysis.

When the shear strength of a steel plate shear wall is considered deformation-controlled, the plastic shear deformation demand, γ_p, predicted by analysis shall not be greater than the permissible plastic shear deformations provided in Table C6.2 for a given performance level. The yield shear deformation, γ_p, of a steel plate shear wall shall be determined from Equation C6-2:

$$\gamma_y = \frac{V_{CE}}{K_w h} \tag{C6-2}$$

where

V_{CE} = expected shear strength of the steel plate shear wall determined in accordance with Section C6.3a, kips (N)

4b. Force-Controlled Actions

1. Shear Actions

a. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior of a steel plate shear wall is considered force-controlled, the lower-bound shear strength, $Q_{CL} = V_{CL}$, shall be determined in accordance with Section C6.3b.

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the shear behavior of a steel plate shear wall is considered force-controlled, the lower-bound shear strength, $Q_{CL} = V_{CL}$, shall be determined in accordance with Section C6.3b.

TABLE C6.1
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Steel Plate Shear Walls Subjected to Shear

Component	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
Steel plate shear walls[a]	1.5	8	12	12	14

CP = collapse prevention performance level as defined in ASCE/SEI 41, Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41, Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41, Chapter 2
[a]Applicable if the web plate is sufficiently thick or if stiffeners, concrete encasement, or backing are provided to prevent shear buckling.

TABLE C6.2
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Steel Plate Shear Walls Subjected to Shear

Component	Modeling Parameters			Expected Deformation Capacity		
	Plastic Shear Deformation, rad		Residual Strength Ratio	Plastic Shear Deformation, rad		
	a	*b*	*c*	IO	LS	CP
Steel plate shear walls[a]	$14\gamma_y$	$16\gamma_y$	0.7	$0.5\gamma_y$	$13\gamma_y$	$15\gamma_y$

[a]Applicable if the web plate is sufficiently thick or if stiffeners, concrete encasement, or backing are provided to prevent shear buckling.

C7. BRACED FRAME CONNECTIONS

1. General

For the purposes of this section, braced frame connections join one or more braces to beams and columns that resist seismic forces and deformations. Three types of braced frames are considered: concentrically braced frames, eccentrically braced frames, and buckling-restrained braced frames, as discussed in Chapter E. In these elements, braced frame connections are located at the brace-beam-column, brace-column, or brace-beam intersection, depending on the bracing configuration. For evaluation, the connection shall be designated as configured to either restrain or accommodate end rotation of the brace as defined in *Seismic Provisions* Section F2.6c.3. Individual component limit states of a braced frame connection shall be designated as either deformation-controlled or force-controlled actions as described in Section C7.3.

Braced frame connections shall be evaluated for the combined loading conditions resulting from (1) axial tension or compression of the brace(s); (2) flexural demands of the braces that are expected to buckle; and (3) axial, shear, and flexural demands resulting from restraint of the adjacent beams and/or columns. For deformation-controlled connections, accurate modeling approaches, as discussed in Chapter E, are required to determine the deformation demands. Stiffness, as well as force and deformation capacities, of braced frame connections are provided herein.

2. Stiffness

The stiffness of braces, beams, and columns within the extents of the brace end connection shall be based on principles of structural mechanics and as specified in the *Specification* unless superseded by supplemental provisions of this section or system-specific sections in Chapters D through I.

2a. Rotation-Restrained Connections at Brace Ends

The rotational stiffness of a connection that restrains end rotation of the brace is permitted to be evaluated as rigid.

2b. Rotation-Accommodating Connections at Brace Ends

When a buckling brace is implicitly modeled with an axial element that models the nonlinear behavior of the braced frame using the parameters defined in Section C3.2 and described in Section E1.2b, the rotational stiffness of a connection that accommodates end rotation of the brace is permitted to be evaluated as rigid, except to account for effects on adjacent members as required by Section C7.2c.

When a buckling brace is explicitly modeled using two-dimensional nonlinear beam-column elements, rotational stiffness of a rotation-accommodating connection, such as a gusset plate or knife plate, shall be computed based on the effective gusset plate width, B_w, and the average unrestrained length of the gusset plate, L_{avg}.

User Note: Braced-end connections may conservatively be modeled as pinned. Where assessments using such simplified modeling indicate the need for retrofit, the engineer should consider using the more accurate connection model for a more accurate assessment.

User Note: The effective gusset plate width, B_w, may conservatively be determined using a 37° projection, with that projection limited by any unconnected edge of the gusset. The effective width of a knife plate may be determined similarly but is often restricted by the gross width of the plate.

The elastic rotational spring stiffness, K_θ, in the plane of brace buckling shall be determined as follows:

$$K_\theta = \frac{EA_g t_p^2}{3L_{avg}} \tag{C7-1}$$

where

A_g = gross area of gusset plate, in.^{2} (mm^2)
= $B_w t_p$
B_w = effective gusset plate width, in. (mm)
L_{avg} = average unrestrained length of gusset plate, in. (mm)
t_p = thickness of gusset plate, in. (mm)

2c. Modeling of Beam-to-Column Joint

Where gusset plates join beams and columns and the gusset-plate thickness is greater than or equal to $0.75t_w$, where t_w is the greater web thickness of both the beam and the column, the beam-to-column connection shall be modeled as fully restrained unless justified otherwise by analysis. For gusset plates welded directly to beams or columns, rigid elements or offsets extending the full gusset-plate length in columns and 75% of the gusset-plate length in beams shall be used, as shown in Figure C7.1, unless justified otherwise by analysis.

User Note: Where a gusset plate frames into the web of a column oriented for weak-axis bending, stiffeners joining the top and bottom of the gusset to the column flange are required for the connection to be considered fully restrained.

Where gusset plates join beams and columns and the gusset-plate thickness is less than $0.75t_w$ of either the beam or the column, the beam-to-column connection shall be modeled as partially restrained using the provisions of Section C5 or as simply supported, unless justified otherwise by testing or analysis.

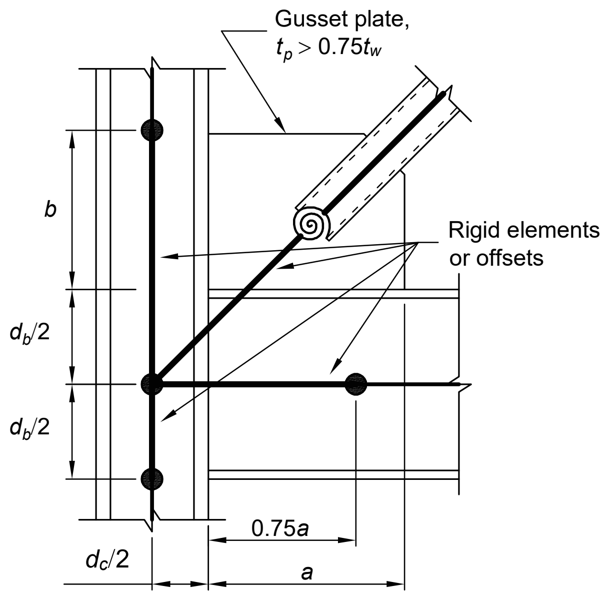


Fig. C7.1. Rigid-element or offset dimensions for welded gusset plates.

3. Strength

Braced frame connection strength shall be based on principles of structural mechanics and as specified in the *Seismic Provisions* and *Specification*, except that default material properties shall be substituted for specified minimum material properties, unless superseded by provisions of this section.

3a. Deformation-Controlled Actions

Welds connecting rotation-accommodating gusset plate connections shall be designated as gusset-plate interface welds.

(a) The strength of gusset-plate interface welds made with complete-joint-penetration (CJP) groove welds meeting the requirements of the *Specification* need not be evaluated.

(b) The strength of an interface weld group consisting of one or two (parallel) fillet weld lines (typically on either side of a single edge of the gusset plate) shall be evaluated based on the strength of the filler metal in accordance with *Specification* Section J2.4.

(c) The strength of gusset-plate interface welds made with partial-joint-penetration groove welds meeting the requirements of the *Specification* shall be evaluated based on their tensile strength.

When brace buckling is explicitly modeled using nonlinear beam-column elements, the flexural strength of the gusset plate shall be determined as follows:

$$Q_{CE} = M_{CE} = F_{ye}\left(\frac{B_w t_p^2}{6}\right) \tag{C7-2}$$

3b. Force-Controlled Actions

Unless modified elsewhere in this section, limit states of brace connections shall be evaluated using nominal strengths determined from the *Specification*, except that F_{yL} and the lower-bound tensile strength determined in accordance with Chapter A, F_{uL}, shall be substituted for F_y and F_u, respectively. For welded connections made with filler metal that does not meet the toughness requirements of *Seismic Provisions* Section A3.4a, limit states shall be evaluated such that the lower-bound strength is equal to 75% of the nominal strength. If a capacity-based analysis approach in accordance with *Seismic Provisions* Section A3.2 is used, F_{ye} and F_{ue} are permitted to be substituted for F_y and F_u, respectively, for evaluating connection limit states, and the force-controlled action caused by gravity loads and earthquake forces, Q_{UF}, is the expected brace capacity (Method 1 of ASCE/SEI 41, Section 7.5.2.1.2). Applicable limit states are not limited to those described in this section.

User Note: The effective gusset plate width, for the purposes of axial strength checks, may conservatively be determined using a 37° projection, with that projection limited by any unconnected edge of the gusset. The effective width of a knife plate may be determined similarly, but is often restricted by the gross width of the plate.

1. Tensile Yielding in the Gross Section

Yield strength, T_{CL}, of a connecting plate shall be determined using the nominal axial strength, P_n, determined from *Specification* Equation D2-1, except that F_{yL} shall be substituted for F_y.

2. Tensile Rupture in the Net Section

Rupture strength, T_{CL}, of a brace or connecting plate shall be determined using the nominal axial strength, P_n, determined from *Specification* Equation D2-2, except that F_{uL} shall be substituted for the specified minimum tensile strength, F_u. Reinforcement used to reinforce the net section in the slotted region of the brace shall conform to the requirements of *Seismic Provisions* Section F2.5b(c).

3. Block Shear

Block shear rupture strength, T_{CL}, of a connecting plate shall be determined using the nominal strength, R_n, determined from *Specification* Equation J4-5, except that the lower-bound properties, F_{yL} and F_{uL}, shall be substituted for F_y and F_u, respectively.

4. Compressive Strength

Lower-bound compressive strength, P_{CL}, of a connecting plate shall be evaluated in the direction of the brace as the minimum of the flexural buckling and gross section yielding resistances in accordance with Equation C7-3 and the lower-bound strength, $Q_{CL} = P_{CL}$:

$$P_{CL} = F_{crL} A_g \leq F_{yL} A_g \tag{C7-3}$$

where

F_{crL} = critical stress of the plate computed using F_{yL}, ksi (MPa)

The critical stress, F_{cr}, shall be computed using the nominal stress, F_n, determined in accordance with *Specification* Section E3, except that F_{yL} shall be substituted for F_y, where the effective length, L_c, is permitted to be computed as KL_{avg}. Unless justified otherwise by analysis, the effective length factor, K, shall be equal to 0.65 for corner gusset plates at the brace-beam-column intersection and 1.2 for midspan gusset plates at the brace-beam intersection.

5. Bearing and Tearout Strength at Bolt Holes

The lower-bound strength, Q_{CL}, of connected material at bolt holes for bearing and tearout shall be determined in accordance with *Specification* Section J3.11. For bolt groups that are not the sole load-transfer mechanism between the brace and frame, the strength of the connected material is permitted to be evaluated using *Specification* Section J3.11 when deformation at the bolt hole at service load is not a design consideration.

4. Permissible Performance Parameters

Component permissible performance parameters shall be determined in accordance with this section.

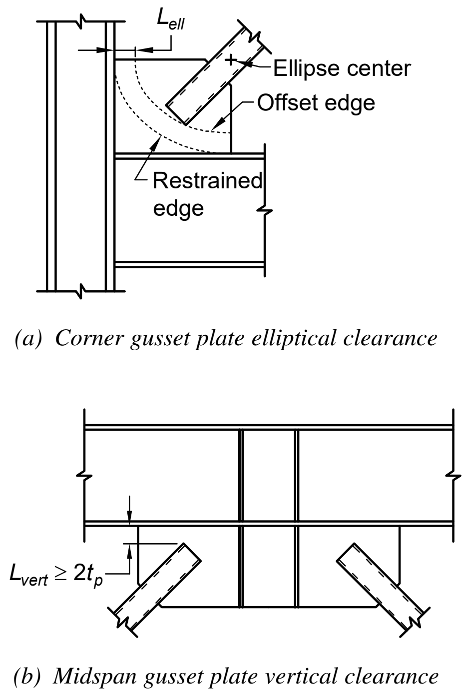
All actions acting on braced frame connections that are designated as rotation-restrained shall be considered force-controlled.

4a. Deformation-Controlled Actions

Welded gusset plate rotation capacity, θ_{gp}, shall be determined based on (i) type of interface weld; (ii) compliance with the toughness requirements of the *Seismic Provisions* Section A3.4a; (iii) ratio of the yield strength of the gusset plate, f_{tUD}, to the tensile strength of the weld group, f_{tCE}, with the limit on the ratio, f_{tUD}/f_{tCE}, defined in the following; and (iv) rotational clearance, L_{ell}, defined in Figure C7.2(a) relative to the plate thickness, t_p. For midspan gusset-plate connections, L_{ell} is permitted to be taken as the vertical clearance between the brace end and beam flange, L_{vert}, provided that L_{vert} is greater than or equal to $2t_p$, as shown in Figure C7.2(b). If the elliptical clearance is not determinable or the vertical clearance is less than $2t_p$ in midspan gusset plates, the connection shall be evaluated as rotation restrained. The yield strength of the gusset plate, in kip/in. (N/mm), is as follows:

$$f_{tUD} = F_{ye} t_p \tag{C7-4}$$

(a) The rotational capacity of gusset plates with interface welds conforming to the requirements of *Seismic Provisions* Section F2.6c.4 and with toughness requirements of *Seismic Provisions* Section A3.4a need not be evaluated.



(a) Corner gusset plate elliptical clearance

(b) Midspan gusset plate vertical clearance

Fig. C7.2. Gusset plate clearance models.

(b) The rotational capacity of gusset plates with interface welds made with CJP groove welds that meet the requirements of the *Specification* but do not meet the toughness requirements of *Seismic Provisions* Section A3.4a shall be determined from Table C7.1 using $f_{tUD}/f_{tCE} = 0.75$.

(c) The rotational capacity of gusset plates with interface welds made with fillet welds that do not meet the toughness requirements of *Seismic Provisions* Section A3.4a shall be determined from Table C7.1.

1. Linear Analysis Procedures

When the rotational capacity of the gusset plate is a consideration, m for computing permissible performance parameters of the brace in both tension and compression shall be determined from Table C3.2, such that $n \leq n_p$, where the modification factor for connection robustness, n_p, is determined from Equation C7-5:

$$n_p = \frac{L_c \theta_{gp}^2}{2\Delta_C} \geq 1 \tag{C7-5}$$

where

L_c = effective length, defined in Section C3.3a.1, in. (mm)

Δ_C = axial deformation at expected compressive buckling strength, in. (mm), determined using Equation C3-2

θ_{gp} = welded gusset plate rotation capacity, equal to d as computed from Table C7.1 and based on Section C7.4a, rad

2. Nonlinear Analysis Procedures

When brace buckling is modeled using a concentrated spring and the parameters defined in Section C3.2, the modeling parameters and permissible deformations of the brace, determined from Table C3.4, shall be modified such that $n\Delta_y \leq L_c \theta_{gp}^2/2$, where Δ_y is equal to Δ_T or Δ_C for braces in tension or compression, respectively.

When brace buckling is modeled using nonlinear beam-column elements capable of simulating member buckling, the modeling parameter of the gusset plate in flexure shall be determined from Table C7.1. When the rotational capacity of the gusset plate is a consideration, the permissible performance parameters shall be determined from Table C7.1.

4b. Force-Controlled Actions

1. Linear Analysis Procedures

When linear analysis procedures are used and the behavior of a braced frame connection is considered force-controlled, the lower-bound strength, Q_{CL}, of the connection shall be determined in accordance with Section C7.3b.

The following exceptions for evaluation of force-controlled actions shall apply:

(a) For rotation-accommodating connections, the demand-to-capacity ratio for the gusset-plate axial yielding limit state in tension, as specified in Section C7.3b.1, is permitted to be a maximum of 1.2.

TABLE C7.1
Modeling Parameters and Permissible Performance Parameters for Nonlinear Analysis Procedures—Braced Frame Connections

Component/Action	Modeling Parameters	Permissible Performance Parameters		
	Strength Ratio at Fracture	Total Rotation, rad		
	f[b]	IO	LS	CP
Welded Gusset-Plate Rotation[a] $d = 0.11\left(\dfrac{L_{ell}}{t_p}\right)^{0.33}\left(\dfrac{f_{tUD}}{f_{tCE}}\right)^{-0.57}$	1.0	$\dfrac{1.5M_{CE}}{K_\theta} \le 0.7d$	$0.7d$	d

CP = collapse prevention performance level as defined in ASCE/SEI 41, Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41, Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41, Chapter 2
[a]For computing d, $\dfrac{L_{ell}}{t_p} \le 8$ and $\dfrac{f_{tUD}}{f_{tCE}} \ge 0.75$.
[b]f = resistance immediately prior to fracture (see Figure C3.1)

(b) For bolt groups loaded in shear that are not the sole load transfer mechanism from the brace to the beam and/or column and where the connected material bearing and tearout resistance does not exceed 1.2 times the bolt fracture resistance, the demand-to-capacity ratio for bolt fracture in shear is permitted to be a maximum of 1.3.

(c) For bolt groups loaded in shear that are not the sole load transfer mechanism between the brace and frame, the demand-capacity ratio for bearing and tearout at bolt holes in the connected material is permitted to be a maximum of 1.1.

2. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the behavior of a braced frame connection is considered force-controlled, the lower-bound strength, Q_{CL}, of the connection shall be determined in accordance with Section C7.3b.

The following exceptions for evaluation of force-controlled actions shall apply:

(a) For rotation-accommodating connections, the demand-capacity ratio for the gusset-plate axial yielding limit state in tension, as specified in Section C7.3b.1, is permitted to be a maximum of 1.2; the brace axial strength determined in Section C3.3a.1 shall be limited by the lower-bound component strength, Q_{CL}, for this limit state.

(b) For bolt groups loaded in shear that are not the sole load transfer mechanism between the brace and frame and where the connected material bearing and tearout resistance does not exceed 1.2 times the bolt fracture resistance, the demand-to-capacity ratio for bolt fracture in shear is permitted to be a maximum of 1.3.

(c) For bolt groups loaded in shear that are not the sole load transfer mechanism between the brace and frame, the demand-to-capacity ratio for bearing and tearout at bolt holes in the connected material is permitted to be a maximum of 1.1.

CHAPTER D
STRUCTURAL STEEL MOMENT FRAMES

Steel moment frames develop their seismic resistance primarily through bending of beams and columns and moment-resisting beam-to-column connections. This chapter describes requirements for the primary and secondary structural steel components of moment frames. Unless otherwise noted in this chapter, these requirements are in addition to any requirement prescribed in Chapter C.

The chapter is organized as follows:

D1. General
D2. Stiffness
D3. Strength
D4. Permissible Performance Parameters
D5. Retrofit Measures

D1. GENERAL

Moment frames shall consist of beams and columns connected by one or more of the connections defined in Table C5.1 or Table C5.2. Modeling procedures, permissible performance parameters, and retrofit measures for moment frames with fully restrained (FR) and partially restrained (PR) beam-to-column connections shall be as determined in Sections D2 through D5.

D2. STIFFNESS

1. Linear Analysis Procedures

If linear analysis procedures are used, the following criteria shall apply:

(a) Moment frames shall be composed of columns, beams, connections, and panel zones. Elastic axial stiffness, shear stiffness, and flexural stiffness of each component shall be determined as specified for each component in Chapter C.

(b) FR and PR beam-to-column connections shall be modeled as specified in Section C5.

(c) Panel zones shall be modeled as specified in Section C4.

(d) Column-to-base connections shall be modeled as specified in Section C5.

2. Nonlinear Static Procedure

If the nonlinear static procedure is used, the following criteria shall apply:

(a) Elastic stiffness properties of components shall be modeled as specified in Section D2.1.

(b) Inelastic action in components shall be represented in the analytical model by nonlinear force-deformation relationships, incorporating multi-force interaction effects where needed, derived from testing or analysis; and

(c) Behavior specific to a component not addressed in this section that can influence the stiffness of a component by more than 5% shall be considered in the analytical model.

User Note: Examples of behavior that can influence the stiffness of a component by more than 5% include panel-zone shear deformations, bolt slippage, composite action, and base anchorage flexibility.

3. Nonlinear Dynamic Procedure

If the nonlinear dynamic procedure is used, in addition to the requirements in Section D2.2, the complete hysteretic behavior of each component shall be determined in accordance with Section B2 and Chapter C.

D3. STRENGTH

Component strengths shall be determined in accordance with Section B2 and Chapter C. Classification of component actions as deformation-controlled or force-controlled shall be in accordance with Section D4.

D4. PERMISSIBLE PERFORMANCE PARAMETERS

1. General

Component permissible strengths and deformations shall be determined in accordance with Section B2 and this section.

The following criteria shall apply:

(a) Flexure actions in FR and PR beam-to-column moment connections listed in Tables C5.1 and C5.2 shall be considered deformation-controlled actions.

(b) Flexural actions in beams and columns shall be considered deformation-controlled.

(c) Axial compression action is force-controlled for all components. Axial tension action is deformation-controlled for all components.

(d) Shear actions in panel zones and beams are considered deformation-controlled; shear actions in columns and FR and PR beam-to-column connections are considered force-controlled.

2. Linear Analysis Procedures

For linear analysis procedures, calculated component actions shall be compared with permissible strengths in accordance with ASCE/SEI 41, Section 7.5.2. The component capacity modification factors, m, for computing the permissible strengths for structural steel components shall be determined from Chapter C. Limit states for which no values for m are provided for a component in Chapter C shall be considered force-controlled.

3. Nonlinear Analysis Procedures

For nonlinear analysis procedures, calculated component actions shall satisfy the requirements of ASCE/SEI 41, Section 7.5.3. Permissible deformations for structural steel components shall be determined from Chapter C.

D5. RETROFIT MEASURES

Seismic retrofit measures for moment frames shall satisfy the requirements of this chapter, Section B3, and the provisions of ASCE/SEI 41.

CHAPTER E

STRUCTURAL STEEL BRACED FRAME AND STEEL PLATE SHEAR WALL REQUIREMENTS

Steel braced frames and steel plate shear walls are those elements that develop seismic resistance primarily through either axial forces in the bracing components or shear forces in the shear wall components, respectively. This chapter describes the element-specific requirements for the primary and secondary structural steel components of steel braced frames or steel plate shear walls. Unless otherwise noted in this chapter, these requirements are in addition to any requirement prescribed in Chapter C.

The chapter is organized as follows:

E1. Concentrically Braced Frames (CBF)
E2. Eccentrically Braced Frames (EBF)
E3. Buckling-Restrained Braced Frames (BRBF)
E4. Steel Plate Shear Walls

E1. CONCENTRICALLY BRACED FRAMES (CBF)

1. General

Concentrically braced frames (CBF) are braced frames where component work lines intersect at a single point at a connection, or at multiple points with the distance between points of intersection being the eccentricity. Bending caused by such eccentricities shall be considered in the modeling and evaluation of the components.

Strength and deformation limits of CBF meeting all requirements of *Seismic Provisions* Section F2 shall be defined employing this section and Section C3. The strength and deformation limits of all other CBF shall be defined by the lowest strength and deformation capacity permitted by the combination of Sections C3 and C7.

2. Stiffness

2a. Linear Analysis Procedures

If linear analysis procedures are used, the following criteria shall apply:

(a) Elastic axial stiffness, shear stiffness, and flexural stiffness of all components shall be determined in accordance with Chapter C.

(b) Fully restrained (FR) and partially restrained (PR) beam-to-column moment connections shall be modeled as specified in Sections C5 and D1. Beam-column connections with corner gusset plates shall be modeled as specified in Section C7. Panel zones, if applicable, shall be modeled as specified in Section C4.

(c) Column-to-base connections shall be modeled as specified in Section C5.

2b. Nonlinear Analysis Procedures

Nonlinear analysis shall be performed using the generalized force-deformation relationship, inelastic beam-column elements with fiber-discretized cross sections, nonlinear lumped plasticity elements, or other rational method. The modeling approach shall be verified by comparison of computed to measured braced frame response or be calibrated to accurately simulate the analytical force-deformation relation given in Figure C3.1 and consistent with values from Table C3.4 for the CBF brace and configuration of the element. The computed behavior for all elements in the CBF shall be evaluated by the limits provided in Chapter B, and Sections C2, C3, C5, and C7. It is permitted to account for the nonlinear response of beams and beam connections in V-type, inverted V-type, and multi-story X-type braced frames. It is permitted to account for the nonlinear response of columns in frames with brace-column intersections not coincident with beam-column joints.

> **User Note:** The commentary provides modeling methods that have been documented to provide acceptable accuracy for the generalized force-deformation relationship and inelastic beam-column elements with fiber-discretized cross-section methods.

3. Strength

Component strengths of CBF shall be determined in accordance with Section B2, Chapter C, and the additional requirements of this section. Classification of component actions as deformation-controlled or force-controlled shall be in accordance with Section E1.4.

Connections that meet the requirements of the *Seismic Provisions* need not be evaluated as force-controlled elements under the requirements of Section C7.

For hollow structural section (HSS) braces filled with normal weight concrete such that the concrete engages the end connections of the brace, the composite strength of the brace shall be considered to compute the capacity-limited brace force when evaluating other component actions, including beams, columns, and connections in Sections C3 and C7. The bare steel strength of HSS braces filled with normal weight concrete shall be used to evaluate brace actions when the concrete fill does not contact or engage the brace end connections. Braces filled with lightweight or other concrete shall be experimentally evaluated in accordance with ASCE/SEI 41, Section 7.6.

4. Permissible Performance Parameters

4a. General

Component permissible strengths and deformations for CBF shall be determined in accordance with Section B2 and the requirements of this section.

The following criteria shall apply for assessment of CBF:

(a) Axial tension and compression actions in braces shall be considered deformation-controlled.

(b) Flexural actions in beams and columns shall be considered deformation-controlled, unless noted otherwise in Section E1.4b.

(c) Axial compression action in columns shall be considered force-controlled.

(d) Shear actions in panel zones shall be considered deformation-controlled; shear actions in beams, columns, and FR and PR beam-to-column moment connections shall be considered force-controlled.

(e) Unless otherwise defined in Section C7, compression, tension, shear, and flexural actions in brace connection components, including gusset plates, bolts, welds, and other connectors, shall be considered force-controlled unless connections are explicitly modeled and test results indicate that connection performance is ductile and stable while the desired brace ductility is achieved.

4b. Linear Analysis Procedures

For linear analysis procedures, calculated component actions shall be compared with permissible strengths in accordance with ASCE/SEI 41, Section 7.5.2. The component capacity modification factors, m, for computing the permissible strengths for structural steel components shall be determined from Chapter C. Limit states for which no values for m are provided for a component in Chapter C shall be considered force-controlled.

Actions in components other than buckling braces in V-braced, inverted V-braced, and multi-story X-braced frames shall be evaluated as force-controlled to resist the unbalanced load effects in combination with gravity loads specified in ASCE/SEI 41, Section 7.2.2. The unbalanced load effects shall be determined using the following conditions:

(a) The expected yield strength of the brace in tension with 30% of the expected compressive strength of the adjacent brace in compression, and

(b) The expected yield strength of the brace in tension with 100% of the expected compressive strength of the adjacent brace in compression, where the expected brace strengths are defined in Section C3.

Exception: It is permitted to classify flexural actions in beams of V-braced and inverted V-braced frames as deformation-controlled to resist the unbalanced load effects using the criteria of Section C3.4a.2.a with the following component capacity modification factors:

(a) m = 2.5 for the collapse prevention and life safety performance levels and m = 1.0 for the immediate occupancy performance level when the beam is classified as a compact section in accordance with *Specification* Section B4.1 and the top flange is laterally braced at a spacing not exceeding the limiting laterally unbraced length for the limit state of yielding, L_p, as defined in *Specification* Chapter F.

(b) m = 1 for all performance levels when the beam is classified as a slender section in accordance with *Specification* Section B4.1 and the top flange is laterally braced at a spacing exceeding limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, L_r, as defined in *Specification* Chapter F.

(c) Linearly interpolated between (a) and (b) for intermediate compactness and lateral bracing conditions.

Actions in columns in braced frames with brace-column joints not coincident with beam-column joints shall be evaluated as force-controlled to resist the unbalanced load effects in combination with gravity loads specified in ASCE/SEI 41, Section 7.2.2. The unbalanced load effects shall be determined using the load conditions specified in this section.

4c. Nonlinear Analysis Procedures

For nonlinear analysis procedures, calculated component actions shall satisfy the requirements of ASCE/SEI 41, Section 7.5.3. Permissible deformation for structural steel components shall be selected from Chapter C.

5. Retrofit Measures

Seismic retrofit measures for CBF shall satisfy the requirements of this section, Section B3, and applicable provisions of ASCE/SEI 41.

E2. ECCENTRICALLY BRACED FRAMES (EBF)

1. General

Eccentrically braced frames (EBF) are braced frames where component work lines do not intersect at a single point and the distance between points of intersection, or eccentricity, exceeds the depth of the smallest member joined at the connection. The component between these points, referred to as the link beam, has a span equal to the eccentricity. Component properties for a link beam shall be taken from Section C2 or C3, depending on the axial force in the link, using the length of the beam or column equal to the eccentricity.

2. Stiffness

2a. Linear Analysis Procedures

If linear analysis procedures are used, the following criteria shall apply:

(a) EBF shall be composed of braces, columns, beams, connections, and panel zones, as applicable. Elastic axial stiffness, shear stiffness, and flexural stiffness of each component shall be calculated as specified for each component in Chapter C.

(b) FR and PR beam-to-column moment connections shall be modeled as specified in Sections C5 and D1. Panel zones, if needed, shall be modeled as specified in Section C4.

(c) Columns and braces shall be modeled as specified in Section C3.

(d) The region of gusset boundary to the beam, column, and brace shall be modeled as rigid unless a more detailed model is available. Brace connections shall be modeled as specified in Section C5.

(e) Column-to-base connections shall be modeled as specified in Section C5.

2b. Nonlinear Static Procedure

If the nonlinear static procedure is used, the following criteria apply:

(a) The elastic properties of components shall be modeled as specified in Section E2.2a.

(b) The nonlinear force-deformation behavior of components to represent yielding or buckling, post-yielding or post-buckling, peak strength, strength reduction after peak strength, and residual strength shall be modeled as specified for each component in Chapter C.

2c. Nonlinear Dynamic Procedure

If the nonlinear dynamic procedure is used, in addition to the requirements in Section E2.2b, the complete hysteretic behavior of each component shall be determined in accordance with Section B2 and Chapter C.

3. Strength

Component strengths of EBF shall be determined in accordance with Section B2 and Chapter C. Classification of component actions as deformation-controlled or force-controlled shall be in accordance with Section E2.4.

4. Permissible Performance Parameters

4a. General

Component permissible strengths and deformations shall be determined in accordance with Section B2 and this section.

The following criteria shall apply for assessment of EBF:

(a) Shear and flexure actions in link beams shall be considered deformation-controlled.

(b) All other actions in link beams and actions in other EBF components shall be considered force-controlled.

(c) Compression, tension, shear, and flexure actions on brace connections, including gusset plates, bolts, welds, and other connectors, shall be considered force-controlled.

4b. Linear Analysis Procedures

For linear analysis procedures, calculated component actions shall be compared with permissible strengths in accordance with ASCE/SEI 41, Section 7.5.2. The component capacity modification factors, m, for computing the permissible strengths for structural steel components shall be selected from Chapter C. Limit states for which no values for m are provided for a component in Chapter C shall be considered force-controlled.

All components in an EBF except the link beams shall be assessed or designed for 1.25 times the lesser of the expected flexural or shear strength of the link beams to ensure link yielding without brace, beam, or column buckling. Where the link beam is attached to the column flange with complete-joint-penetration groove welds, the requirements for these connections shall be the same as for FR beam-to-column moment connections in Section C5.

A link beam that exhibits inelastic shear yielding with an axial load ratio, P_{UF}/P_{ye}, greater than 0.6 shall remain elastic for shear actions and m_x and m_y in Section C3.4a.3 shall reduce to unity, where P_{UF} is the axial force (compression or tension) determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, $P_{ye} = A_g F_{ye}$, A_g is the gross area of the link beam, and F_{ye} is determined in accordance with Chapter A. This provision is applicable for both compression and tension axial force.

4c. Nonlinear Analysis Procedures

For nonlinear analysis procedures, calculated component actions shall satisfy the requirements of ASCE/SEI 41, Section 7.5.3. Permissible deformations for structural steel components shall be selected from Chapter C.

Shear yielding link beams with an axial load ratio, P_{UF}/P_{ye}, greater than 0.6 shall remain elastic for all actions and the permissible plastic shear deformations for shear action in Section C3.4a.3 will reduce to zero. This provision is applicable for both compression and tension axial forces.

5. Retrofit Measures

Seismic retrofit measures for EBF shall satisfy the requirements of this section, Section B3, and applicable provisions of ASCE/SEI 41.

E3. BUCKLING-RESTRAINED BRACED FRAMES (BRBF)

1. General

Buckling-restrained braced frames (BRBF) are concentrically braced frame systems with buckling-restrained braces (BRB) that are composed of a steel core and a casing system that restrains the core from buckling.

2. Stiffness

2a. Linear Analysis Procedures

If linear analysis procedures are used, the following criteria shall apply:

(a) BRBF shall be composed of buckling-restrained braces, columns, beams, connections, and panel zones, as applicable. Elastic axial stiffness, shear stiffness, and flexural stiffness of each component shall be calculated as specified for each component in Chapter C.

(b) FR and PR beam-to-column moment connections shall be modeled as specified in Sections C5 and D1. Panel zones, if applicable, shall be modeled as specified in Section C4.

(c) Columns and braces shall be modeled as specified in Section C3.

(d) The region of gusset boundary to beam, column, and brace shall be modeled as rigid unless a more detailed model is available. Brace connections shall be modeled as specified in Section C5.

(e) Column-to-base connections shall be modeled as specified in Section C5.

(f) Braces shall be modeled with the stiffness of the yielding core segments as specified in Section C3. The transition segments shall include the properties of the brace that is stiffened from the core to the gusset.

2b. Nonlinear Static Procedure

If the nonlinear static procedure is used, the following criteria shall apply:

(a) The elastic properties of components shall be modeled as specified in Section E3.2a.

(b) The nonlinear force-deformation behavior of components to represent yielding or buckling, post-yielding or post-buckling, peak strength, strength reduction after peak strength, and residual strength shall be modeled as specified for each component in Chapter C.

(c) The nonlinear axial force-deformation behavior of buckling-restrained braces is permitted to be modeled as shown in Figure C1.1 with parameters as defined in Section C3, or these relationships are permitted to be derived by testing or analysis. The parameter Δ_y defined in Section C3 shall represent the axial deformation at the expected brace yield strength, which occurs at Point B in the curve in Figure C1.1. The post-peak slope beyond modeling parameter b from Section C3 is permitted to match the negative yield stiffness down to a near zero residual strength.

2c. Nonlinear Dynamic Procedure

If the nonlinear dynamic procedure is used, in addition to the requirements in Section E3.2b, the complete hysteretic behavior of each component shall be determined in accordance with Section B2 and Chapter C.

3. Strength

Component strengths of BRBF shall be determined in accordance with Section B2, Chapter C, and the additional requirements of this section. Classification of component actions as deformation-controlled or force-controlled shall be in accordance with Section E3.4.

BRBF systems shall be evaluated and designed as capacity-based systems with the BRB casing system, connections, and adjoining members designed to resist the maximum forces that the steel core can develop in accordance with Section C3.3a.1.

4. Permissible Performance Parameters

4a. General

Component permissible strengths and deformations shall be determined in accordance with Section B2 and the requirements of this section.

The following criteria shall apply for assessment of BRBF:

(a) Axial tension and compression actions in braces shall be considered deformation-controlled.

(b) Flexure actions in beams and columns shall be considered deformation-controlled.

(c) Axial compression action in columns shall be considered force-controlled.

(d) Shear actions in panel zones shall be considered deformation-controlled, and shear actions in beams, columns, and FR and PR beam-to-column connections shall be considered force-controlled.

(e) Compression, tension, shear, and bending actions in brace connection components, including gusset plates, bolts, welds, and other connectors, shall be considered force-controlled, unless connections are explicitly modeled and test results indicate that connection performance is ductile and stable while the desired brace ductility is achieved.

The permissible strengths and deformations for a BRB in Section C3 are permitted if testing in accordance with *Seismic Provisions* Section K3, as a minimum, is submitted. The deformation term, Δ_{bm}, given in *Seismic Provisions* Section K3.4c shall be the maximum of 100% of the deformations at the BSE-1N seismic hazard level or 65% of the deformations at the BSE-2N seismic hazard level, as defined in ASCE/SEI 41, Chapter 2.

4b. Linear Analysis Procedures

For linear analysis procedures, calculated component actions shall be compared with permissible strengths in accordance with ASCE/SEI 41, Section 7.5.2. The component capacity modification factors, *m*, for computing the permissible strengths for structural steel components shall be selected from Chapter C. Limit states for which no values of *m* are provided for a component in Chapter C shall be considered force-controlled.

4c. Nonlinear Analysis Procedures

For nonlinear analysis procedures, calculated component actions shall satisfy the requirements of ASCE/SEI 41, Section 7.5.3. Permissible deformations for structural steel components shall be selected from Chapter C.

5. Retrofit Measures

Seismic retrofit measures for BRBF shall satisfy the requirements of this section, Section B3, and applicable provisions of ASCE/SEI 41.

In the case where additional seismic force-resisting elements are added in series with the BRBF to reduce the demands on the BRBF components, the relative stiffness for each component shall be incorporated into the analysis.

If the BRB component not meeting the permissible performance parameters is replaced with a larger capacity BRB component, the connections and adjoining members (beams and columns) shall be evaluated for the new expected brace strengths determined in Section E3.3.

If a BRBF is added as the retrofit element, the design shall be based on determining the nominal strengths according to the procedures in these Provisions and the *Seismic Provisions*, as applicable.

E4. STEEL PLATE SHEAR WALLS

1. General

Steel plate shear walls, with or without perforations, are connected to horizontal and vertical boundary elements on all four sides of the steel plate shear wall. These boundary elements shall be evaluated as beams or columns. Component properties for a steel plate shear wall shall be taken from Sections C1 through C6, as applicable.

2. Stiffness

2a. Linear Analysis Procedures

If linear analysis procedures are used, the following criteria shall apply:

(a) Steel plate shear walls are composed of plate walls, columns, beams, connections, and panel zones, as applicable. Elastic axial stiffness, shear stiffness, and flexural stiffness of each component shall be determined as specified for each component in Chapter C.

(b) FR and PR connections shall be modeled as specified in Sections C5 and D1. Panel zones, if needed, shall be modeled as specified in Section C4.

(c) Column-to-base connections shall be modeled as specified in Section C5.

(d) Steel plate used as shear walls, with web plates sufficiently thick or stiffened to prevent buckling, shall be modeled as specified in Section C6. Other methods for analyzing steel plate shear walls are permitted based on accepted principles of structural mechanics for this type of element.

2b. Nonlinear Static Procedure

If the nonlinear static procedure is used, the following criteria apply:

(a) The elastic properties of components shall be modeled as specified in Section E4.2a.

(b) The nonlinear force-deformation behavior of components to represent yielding or buckling, post-yielding or post-buckling, peak strength, strength reduction after peak strength, and residual strength shall be modeled as specified for each component in Chapter C.

2c. Nonlinear Dynamic Procedure

If the nonlinear dynamic procedure is used, in addition to the requirements in Section E4.2b, the complete hysteretic behavior of each component shall be determined in accordance with Section B2 and Chapter C.

3. Strength

Component strengths of steel plate shear walls shall be determined in accordance with Section B2 and Chapter C. Classification of component actions as deformation-controlled or force-controlled shall be in accordance with Section E4.4.

4. Permissible Performance Parameters

4a. General

Component permissible strengths and deformations of steel plate shear walls shall be determined in accordance with Section B2 and the requirements of this section.

The following criteria shall apply for assessment of steel plate shear walls not subject to shear buckling:

(a) Shear action in steel plate shear walls shall be considered deformation-controlled. The shear strength and stiffener requirements shall be determined in accordance with Section C6.

(b) Flexure actions in beams and columns shall be considered deformation-controlled.

(c) Axial compression action in columns shall be considered force-controlled.

(d) Shear actions in panel zones shall be considered deformation-controlled, and shear actions in beams, columns, and FR and PR beam-to-column moment connections shall be considered force-controlled.

(e) Compression, tension, shear, and bending actions on connections, including gusset plates, bolts, welds, and other connectors, shall be considered force-controlled, unless connections are explicitly modeled, and testing indicates that connection performance is ductile and stable while the desired plate wall ductility is achieved.

4b. Linear Analysis Procedures

For linear analysis procedures, calculated component actions shall be compared with permissible strengths in accordance with ASCE/SEI 41, Section 7.5.2. The component capacity modification factors, m, for computing the permissible strengths for the structural steel components shall be selected from Chapter C. Limit states for which no values of m are provided for a component in Chapter C shall be considered force-controlled.

4c. Nonlinear Analysis Procedures

For nonlinear analysis procedures, calculated component actions shall satisfy the requirements of ASCE/SEI 41, Section 7.5.3. Permissible deformation for structural steel components shall be selected from Chapter C.

5. Retrofit Measures

Seismic retrofit measures for steel plate shear walls shall satisfy the requirements of this section, Section B3, and applicable provisions of ASCE/SEI 41. Potential retrofit measures are permitted to include the addition of stiffeners, encasement in concrete, or the addition of concrete backing on steel plate shear walls.

CHAPTER F

STRUCTURAL STEEL FRAMES WITH INFILLS

F1. GENERAL

Structural steel frames with partial or complete infills of reinforced concrete or reinforced or unreinforced masonry shall be evaluated considering the combined stiffness of the steel frame and infill material.

The engineering properties and permissible performance parameters for the infill walls shall comply with the requirements in ASCE/SEI 41, Chapter 10, for concrete and ASCE/SEI 41, Chapter 11, for masonry. Infill walls and frames shall be considered to resist the seismic force in composite action, considering the relative stiffness of each element, until complete failure of the walls has occurred. The interaction between the structural steel frame and infill shall be considered using procedures specified in ASCE/SEI 41, Chapter 10, for concrete frames with infill. The analysis of each component shall be performed in stages, considering the effects of interaction between the elements and carried through each performance level. At the point where the infill has been deemed to fail, as determined by the permissible performance parameters specified in ASCE/SEI 41, Chapters 10 or 11, the wall shall be removed from the analytical model. The analysis shall be resumed on the bare structural steel frame, taking into consideration any vertical discontinuity created by the degraded wall. At this point, the engineering properties and permissible performance parameters for the frame components, as specified in Chapter C, shall apply.

F2. RETROFIT MEASURES

Seismic retrofit measures for structural steel frames with infills shall satisfy the requirements of Section B3, Section F1, and the provisions of ASCE/SEI 41.

CHAPTER G
DIAPHRAGMS

ASCE/SEI 41, Chapter 7, includes provisions for classification of diaphragms, mathematical modeling, diaphragm chords, diaphragm collectors, and diaphragm ties. Specific provisions for diaphragms considered in this chapter include steel deck diaphragms that are either (1) bare, (2) filled with reinforced structural concrete, or (3) filled with unreinforced or insulating (nonstructural) concrete topping. Additional requirements are provided for diaphragm elements, including steel truss diaphragms, archaic diaphragms, and chord and collector elements.

The chapter is organized as follows:

G1. Bare Steel Deck Diaphragms
G2. Steel Deck Diaphragms with Reinforced Concrete Structural Topping
G3. Steel Deck Diaphragms with Unreinforced Structural Concrete Topping or Lightweight Insulating Concrete
G4. Horizontal Steel Truss Diaphragms
G5. Archaic Diaphragms—Shallow Brick Arches Spanning Between Structural Steel Floor Beams
G6. Chord and Collector Elements

G1. BARE STEEL DECK DIAPHRAGMS

1. General

Steel deck diaphragms shall be composed of profiled steel panels. Panels (decking units) shall be attached to each other at side-laps by welds, crimping (such as button punching), or mechanical fasteners, and shall be attached to the structural steel supports by welds or by mechanical fasteners. Bare steel deck diaphragms are permitted to resist diaphragm seismic loads acting alone or in conjunction with supplementary horizontal steel truss diaphragms designed in accordance with the requirements of Section G4. Structural steel frame components, to which bare steel deck diaphragms are attached, shall be considered to be the chord and collector elements.

The criteria of this section shall apply to existing diaphragms and to stiffened, strengthened, or otherwise retrofitted diaphragms. Interaction of new and existing components of retrofitted diaphragms shall be evaluated to ensure strain compatibility. Load transfer mechanisms between new and existing diaphragm components shall be evaluated.

2. Stiffness

Bare steel deck diaphragms shall be classified as flexible, stiff, or rigid in accordance with ASCE/SEI 41, Section 7.2.9. The stiffness shall be determined in accordance with ANSI/AISI S310.

The force-deformation model for bare steel deck diaphragms shall include profile buckling and yielding, and local deformations at side-lap and structural (support) connectors.

3. Strength

Strength of bare steel deck diaphragms shall be determined in accordance with this section.

3a. Deformation-Controlled Actions

For strength based on deformation-controlled actions, the expected strength, Q_{CE}, for bare steel deck diaphragms shall be determined by modifying the nominal diaphragm strength, S_n, determined in accordance with ANSI/AISI S310. If the nominal strength is controlled by panel buckling, the expected strength shall be determined as $1.1S_{nb}$, where S_{nb} is the nominal shear strength per unit length of a diaphragm controlled by out-of-plane buckling. If the nominal strength is controlled by side-lap or structural connections, the expected strength depends on the connectors employed, as follows:

(a) If power actuated fasteners are used for the structural connections, the expected strength shall be determined as $1.2S_{nf}$, where S_{nf} is the nominal shear strength per unit length of diaphragm controlled by connections.

(b) For all other structural or side-lap connections within the scope of ANSI/AISI S310, the expected strength shall be determined as $1.0S_{nf}$.

3b. Force-Controlled Actions

For strength based on force-controlled actions, the lower-bound shear strength, Q_{CL}, for bare steel deck diaphragms shall be determined by modifying the nominal diaphragm strength, S_n, determined in accordance with ANSI/AISI S310. If the nominal strength is controlled by panel buckling, S_{nb}, the lower-bound strength shall be determined as $0.9S_{nb}$. If the nominal strength is controlled by side-lap or structural connections, S_{nf}, the expected strength depends on the connectors employed, as follows:

(a) If power actuated fasteners are used for the structural connections, the lower-bound strength shall be determined as $1.0S_{nf}$.

(b) If welds are used for the structural or side-lap connectors, the lower-bound strength shall be determined as $0.8S_{nf}$. For all other side-lap or structural connections within the scope of ANSI/AISI S310, the expected strength shall be determined as $0.9S_{nf}$.

4. Permissible Performance Parameters

For life safety or lower performance levels, bearing support or anchorage of the deck shall be maintained. For higher performance levels than life safety, the amount of damage to the connections shall not impair the load transfer between the diaphragm and the structural steel frame. Deformations shall not exceed the threshold of deflections that cause unacceptable damage to other components, either structural or nonstructural, at the target performance level(s). Permissible performance parameters for collectors shall be as specified in Section G6.4.

4a. Deformation-Controlled Actions

1. Linear Analysis Procedures

Bare steel deck is permitted to be designated as deformation-controlled. When the strength of a bare steel deck diaphragm is considered deformation-controlled, the expected component strength, Q_{CE}, shall be determined from Section G1.3a and m shall be taken from Table G1.1.

2. Nonlinear Analysis Procedures

The generalized force-deformation curve shown in Figure C1.1, with the modeling parameters d, e, and c as defined in Table G1.2, shall be used for bare steel deck diaphragms, or these relationships may be derived from testing or analysis.

When the shear strength of a bare steel deck diaphragm is considered deformation-controlled, the plastic shear deformation, γ_p, shall be no greater than the permissible plastic shear deformation provided in Table G1.2 for a given performance level. The yield shear deformation, γ_y, of a bare steel deck diaphragm shall be calculated as the expected component strength, Q_{CE}, divided by the initial stiffness as determined in Section G1.2.

4b. Force-Controlled Actions

1. Linear Analysis Procedures

When the shear strength of a bare steel deck diaphragm is considered force-controlled, the lower-bound shear strength, Q_{CL}, shall be determined in accordance with Section G1.3b.

2. Nonlinear Analysis Procedures

When the shear strength of a bare steel deck diaphragm is considered force-controlled, the total shear deformation, γ, of the diaphragm shall not exceed γ_y determined in accordance with Section G1.4a.2. The lower-bound shear strength, Q_{CL}, determined in accordance with Section G1.3b, shall not be less than the maximum force determined by ASCE/SEI 41, Section 7.5.3.2.3.

5. Retrofit Measures

Seismic retrofit measures for bare steel deck diaphragms shall satisfy the requirements of this section, Section B3, and ASCE/SEI 41.

TABLE G1.1
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Bare Steel Deck Diaphragm

Component or Action	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
Shear strength controlled by connectors[a]:					
support: PAF; side-lap: screw	1.0	1.5	2.0	2.0	3.0
support: weld; side-lap: screw	1.0	1.5	2.0	2.0	3.0
support: weld; side-lap: button punch	1.0	1.0	1.3	2.0	3.0
support: weld; side-lap: weld	1.0	1.3	1.6	2.0	3.0
Shear strength controlled by panel buckling	1.25	2.0	3.0	2.0	3.0

CP = collapse prevention performance level as defined in ASCE/SEI 41, Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41, Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41, Chapter 2
PAF = power actuated fasteners
[a]For panels with spans between supports with fasteners greater than 60 in. (1 500 mm), the spacing of side-lap connections between supports shall not exceed 36 in. (900 mm), and the spacing of edge fasteners between supports shall not exceed 36 in. (900 mm).

TABLE G1.2
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Bare Steel Deck Diaphragms

Component or Action	Modeling Parameters			Permissible Deformations		
	Plastic Shear Deformation, rad		Residual Strength Ratio	Plastic Shear Deformation, rad		
	d	*e*	*c*	IO	LS	CP
Shear strength controlled by connectors[a]:						
support: PAF; side-lap: screw	$2.8\gamma_y$	$4.0\gamma_y$	0.4	$1.4\gamma_y$	$2.8\gamma_y$	$4.0\gamma_y$
support: weld; side-lap: screw	$2.8\gamma_y$	$4.0\gamma_y$	0.05[b]	$1.4\gamma_y$	$2.8\gamma_y$	$4.0\gamma_y$
support: weld; side-lap: button punch	$1.7\gamma_y$	$3.1\gamma_y$	0.05[b]	$0.9\gamma_y$	$1.7\gamma_y$	$3.1\gamma_y$
support: weld; side-lap: weld	$2.3\gamma_y$	$3.6\gamma_y$	0.05[b]	$1.2\gamma_y$	$2.3\gamma_y$	$3.6\gamma_y$
Shear strength controlled by panel buckling	$3.6\gamma_y$	$5.6\gamma_y$	0.5	$1.8\gamma_y$	$3.7\gamma_y$	$6.0\gamma_y$

[a]For panels with spans between supports with fasteners greater than 60 in. (1 500 mm), the spacing of side-lap connections between supports shall not exceed 36 in. (900 mm), and the spacing of edge fasteners between supports shall not exceed 36 in. (900 mm).
[b]Structural connectors generally control residual strength. Value based on arc spot weld; for an arc seam weld, c = 0.15.

G2. STEEL DECK DIAPHRAGMS WITH REINFORCED CONCRETE STRUCTURAL TOPPING

1. General

Steel deck diaphragms with reinforced concrete structural topping, consisting of either composite or noncomposite construction, are permitted to resist seismic diaphragm loads. The concrete fill shall be either normal or lightweight structural concrete, with reinforcing composed of welded wire reinforcing or reinforcing bars. It is permitted in all instances to ignore the contributions of any reinforcing and apply the provisions of Section G3 in lieu of this section. Panels (decking units) shall be attached to each other at side-laps by welds, crimping, or mechanical fasteners and shall be attached to structural steel supports by welds or by steel headed stud anchors. The structural steel framing components to which the topped steel deck diaphragms are attached, or the reinforcing steel within the concrete structural topping, are permitted to be considered the chord and collector elements.

The criteria of this section shall apply to existing diaphragms and new and retrofitted diaphragms. Interaction of new and existing components of retrofitted diaphragms shall be evaluated for strain compatibility. Load transfer mechanisms between new and existing diaphragm components shall be considered in determining the flexibility of the diaphragm.

2. Stiffness

For existing steel deck diaphragms with reinforced concrete structural topping, a rigid diaphragm assumption is permitted if the span-to-depth ratio is not greater than 5:1. For greater span-to-depth ratios, and in cases with plan irregularities, diaphragm stiffness shall be explicitly included in the analysis in accordance with ASCE/SEI 41, Section 7.2.9. Diaphragm stiffness shall be determined using the cast-in-place concrete diaphragm provisions of ASCE/SEI 41, Section 10.10.2.2, for the slab above the top of the steel deck or another method with a representative concrete thickness approved by the authority having jurisdiction (AHJ).

Inelastic properties of diaphragms shall not be included in inelastic seismic analyses if the weak link in the diaphragm is connection failure.

3. Strength

The strength of steel deck diaphragms with reinforced concrete structural topping shall be determined in accordance with this section.

3a. Deformation-Controlled Actions

The expected component strength, Q_{CE}, of steel deck diaphragms with reinforced concrete structural topping shall be determined by ASCE/SEI 41, Section 10.10.2.3, considering the reinforced slab above the top of the steel deck or by another procedure approved by the AHJ. Expected component strengths, Q_{CE}, for steel headed stud anchors shall be equal to the nominal strengths specified in *Specification* Chapter I for steel headed stud anchors, except that the expected tensile strength,

F_{ue}, shall be substituted for the specified minimum tensile strength, F_u. F_{ue} shall be determined in accordance with Section A5.2.

Alternatively, the expected component strength, Q_{CE}, of steel deck diaphragms with reinforced concrete structural topping shall be taken as two times the allowable strength values specified in the applicable building code unless a larger value is justified by test data or manufacturer data.

3b. Force-Controlled Actions

The lower-bound component strength, Q_{CL}, of steel deck diaphragms with reinforced concrete structural topping shall be determined by ASCE/SEI 41, Section 10.10.2.3, considering the reinforced slab above the top of the steel deck or by another procedure approved by the AHJ. Lower-bound component strengths, Q_{CL}, for steel headed stud anchors shall be equal to the nominal strengths specified in *Specification* Chapter I for steel headed stud anchors, except that the lower-bound tensile strength, F_{uL}, shall be substituted for F_u. F_{uL} shall be determined in accordance with Section A5.2.

4. Permissible Performance Parameters

For life safety or lower performance levels, bearing support or anchorage shall be maintained. For higher performance levels than life safety, the amount of damage to the connections or cracking in concrete-filled slabs shall not impair the load transfer between the diaphragm and the structural steel frame. Deformations shall be limited to be below the threshold of deflections that cause damage to other components, either structural or nonstructural, at specified performance levels. Permissible performance parameters for collectors shall be as specified in Section G6.4.

Steel headed stud anchors for structural steel beams designed to act compositely with the slab shall have the design strength to transfer both diaphragm shears and composite beam shears. Where the beams are encased in concrete, use of bond between the structural steel and the concrete is permitted to transfer loads.

4a. Deformation-Controlled Actions

1. Linear Analysis Procedures

When the strength of a steel deck diaphragm with reinforced concrete structural topping is considered deformation-controlled, the expected component strength, Q_{CE}, shall be determined from Section G2.3a and m shall be taken from ASCE/SEI 41, Table 10-21 and Table 10-22, as specified in ASCE/SEI 41, Section 10.10.2.4.

2. Nonlinear Analysis Procedures

The generalized force-deformation curve shown in Figure C1.1, with the modeling parameters a, b, and c as defined in ASCE/SEI 41, Tables 10-19 and 10-20, and as specified in ASCE/SEI 41, Section 10.10.2.4, shall be used for steel deck diaphragms with reinforced concrete structural topping. Alternatively, these relationships may be derived from testing or analysis.

When the shear strength of a steel deck diaphragm with reinforced concrete structural topping is considered deformation-controlled, the total shear deformation, γ, shall be evaluated against the permissible shear deformations provided in ASCE/SEI 41, Table 10-19 and Table 10-20, as specified in ASCE/SEI 41, Section 10.10.2.4.

4b. Force-Controlled Actions

1. Linear Analysis Procedures

When the strength of a steel deck diaphragm with reinforced concrete structural topping is considered force-controlled, the lower-bound shear strength, Q_{CL}, shall be determined in accordance with Section G2.3b.

2. Nonlinear Analysis Procedures

When the shear strength of a steel deck diaphragm with reinforced concrete structural topping is considered force-controlled, the total shear deformation, γ, of the diaphragm shall not exceed Point B as defined in the generalized force-deformation curve of Figure C1.1 with initial stiffness defined in Section G2.2 and strength defined in Section G2.3. The lower-bound shear strength, Q_{CL}, determined in accordance with Section G2.3b, shall not be less than the maximum force determined by ASCE/SEI 41, Section 7.5.3.2.3.

5. Retrofit Measures

Seismic retrofit measures for steel deck diaphragms with reinforced concrete structural topping shall satisfy the requirements of this section, Section B3, and ASCE/SEI 41.

G3. STEEL DECK DIAPHRAGMS WITH UNREINFORCED STRUCTURAL CONCRETE TOPPING OR LIGHTWEIGHT INSULATING CONCRETE

1. General

Seismic diaphragm loads are permitted to be resisted by steel deck diaphragms with unreinforced concrete, concrete with temperature and shrinkage reinforcing with or without headed stud anchors, or lightweight insulating concrete as defined in ANSI/AISI S310. The provisions of this section apply to plain concrete or where the reinforcing qualifies as temperature and shrinkage reinforcement in accordance with either ANSI/SDI C, Section 2.4.B.15.a.1, for composite steel deck-slabs, or with ANSI/SDI NC, Section 2.4.B.2, for noncomposite steel deck with concrete. Panels (decking units) shall be attached to each other at side-laps by welds, crimping, or mechanical fasteners and shall be attached to structural steel supports by welds or by steel headed stud anchors. The structural steel frame components to which the topped steel deck diaphragm is attached shall be considered the chord and collector elements.

The criteria of this section shall apply to existing diaphragms and to stiffened, strengthened, or otherwise retrofitted diaphragms. Interaction of new and existing components of retrofitted diaphragms shall be evaluated to ensure strain compatibility. Load transfer mechanisms between new and existing diaphragm components shall be evaluated.

2. Stiffness

Steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete shall be classified as flexible, stiff, or rigid in accordance with ASCE/SEI 41, Section 7.2.9. The diaphragm stiffness shall be determined in accordance with ANSI/AISI S310.

3. Strength

The strength of steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete shall be determined in accordance with this section.

3a. Deformation-Controlled Actions

The expected component strength, Q_{CE}, for steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete shall be determined by modifying the nominal diaphragm strength, S_n, determined in accordance with ANSI/AISI S310. If the deck uses welds for the structural connectors, the expected strength shall be determined as $1.8S_{nf}$. If the deck uses welded steel headed stud anchors for the structural connectors, the expected strength shall be determined as $1.5S_{nf}$.

3b. Force-Controlled Actions

The lower-bound component strength, Q_{CL}, for steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete shall be determined by modifying the nominal diaphragm strength, S_n, determined in accordance with ANSI/AISI S310. If the deck uses welds for the structural connectors, the lower-bound strength shall be determined as $1.0S_{nf}$. If the deck uses welded steel headed stud anchors for the structural connectors, the lower-bound strength shall be determined as $1.0S_{nf}$.

4. Permissible Performance Parameters

For life safety or lower performance levels, bearing support or anchorage of the deck shall be maintained. For higher performance levels than life safety, the amount of damage to the connections shall not impair the load transfer between the diaphragm and the structural steel frame. Deformations shall not exceed the threshold of deflections that cause unacceptable damage to other components, either structural or nonstructural, at the target performance level(s). Permissible performance parameters for collectors shall be as specified in Section G6.4.

4a. Deformation-Controlled Actions

1. Linear Analysis Procedures

When the strength of a steel deck diaphragm with unreinforced structural concrete topping or lightweight insulating concrete is considered deformation-controlled, the expected component strength, Q_{CE}, shall be determined from Section G3.3a and m shall be taken from Table G3.1.

2. Nonlinear Analysis Procedures

The generalized force-deformation curve shown in Figure C1.1, with the parameters d, e, and c as defined in Table G3.2 shall be used for steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete, or these relationships may be derived from testing or analysis.

When the shear strength of steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete is considered deformation-controlled, the total shear deformation, γ, shall be no greater than the permissible shear deformations provided in Table G3.2 for a given performance level. The initial shear deformation, γ_i, of a steel deck diaphragm with unreinforced structural concrete topping or lightweight insulating concrete shall be calculated as the expected strength, Q_{CE}, divided by the initial stiffness as determined in Section G3.2.

4b. Force-Controlled Actions

1. Linear Analysis Procedures

When the shear strength of a steel deck diaphragm with unreinforced structural concrete topping or lightweight insulating concrete is considered force-controlled, the lower-bound shear strength, Q_{CL}, shall be determined in accordance with Section G3.3b.

2. Nonlinear Analysis Procedures

When the shear strength of a steel deck diaphragm with unreinforced structural concrete topping or lightweight insulating concrete is considered force-controlled, the total shear deformation, γ, of the diaphragm shall not exceed γ_i determined in accordance with Section G3.4a.2. The lower-bound shear strength, Q_{CL}, determined in accordance with Section G3.3b, shall not be less than the maximum force determined by ASCE/SEI 41, Section 7.5.3.2.3.

5. Retrofit Measures

Seismic retrofit measures for steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete shall satisfy the requirements of this section, Section B3, and ASCE/SEI 41.

TABLE G3.1
Component Capacity Modification Factor, *m*, for Linear Analysis Procedures—Steel Deck Diaphragm with Unreinforced Structural Concrete Topping or Lightweight Insulating Concrete

Component or Action	IO	Primary Component		Secondary Component	
		LS	CP	LS	CP
Shear strength of deck with unreinforced structural concrete topping or lightweight insulating concrete					
deck welded to support (arc spot or arc seam)	1.5	4.0	6.0	6.0	8.0
headed shear studs welded through deck to support	1.5	3.0	4.0	6.0	8.0

CP = collapse prevention performance level as defined in ASCE/SEI 41, Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41, Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41, Chapter 2

TABLE G3.2
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Steel Deck Diaphragm with Unreinforced Structural Concrete Topping or Lightweight Insulating Concrete

Component or Action	Modeling Parameters			Permissible Deformations		
	Shear Deformation, rad		Residual Strength Ratio	Shear Deformation, rad		
	d	*e*	*c*	IO	LS	CP
Shear strength of deck with unreinforced structural concrete topping or lightweight insulating concrete						
deck welded to support (arc spot or arc seam)	$8.0\gamma_i$	$10.0\gamma_i$	0.4	$2.0\gamma_i$	$8.0\gamma_i$	$10.0\gamma_i$
headed shear studs welded through deck to support	$8.0\gamma_i$	$10.0\gamma_i$	0.3	$2.0\gamma_i$	$8.0\gamma_i$	$10.0\gamma_i$

G4. HORIZONTAL STEEL TRUSS DIAPHRAGMS

1. General

Horizontal steel truss diaphragms are permitted to act as diaphragms independently or in conjunction with steel deck. Where either a bare steel deck roof or structural concrete fill over steel deck is provided, relative rigidities between the steel truss and the bare steel deck roof or structural concrete fill over steel deck shall be considered in the analysis.

The criteria of this section shall apply to existing truss diaphragms, retrofitted truss diaphragms, and new diaphragms added to an existing building.

Where steel truss diaphragms are added as part of a retrofit plan, interaction of new and existing components of strengthened diaphragm elements (stiffness compatibility) shall be evaluated, and the load transfer mechanisms between new and existing diaphragm components shall be considered in determining the stiffness of the strengthened diaphragm.

Load transfer mechanisms between new diaphragm components and existing frames shall be considered in determining the stiffness of the diaphragm or frame element.

2. Stiffness

2a. Linear Analysis Procedures

Steel truss diaphragm elements shall be modeled as horizontal truss components (similar to structural steel braced frames) where axial stiffness controls deflections. Connections are permitted to be modeled as pinned except where connections provide moment resistance or where eccentricities exist at the connections. In such cases, connection rigidities shall be modeled. Stiffness of truss diaphragms shall be explicitly considered in distribution of seismic forces to vertical components.

2b. Nonlinear Analysis Procedures

Inelastic models similar to those of structural steel braced frames shall be used for truss components where nonlinear behavior of truss components occurs. Elastic properties of truss diaphragms are permitted in the model for inelastic seismic analyses where nonlinear behavior of truss components does not occur.

3. Strength

The strength of truss diaphragm members shall be determined as specified for structural steel braced frame members in Chapter E and using the appropriate expected or lower-bound properties as provided in Chapter A. Lateral support of truss diaphragm members provided by steel deck, with or without concrete fill, shall be considered in the evaluation of truss diaphragm strengths. Gravity load effects shall be included in the required strength for those members that support gravity loads.

4. Permissible Performance Parameters

Permissible performance parameters for horizontal steel truss diaphragm components shall be as specified for concentrically braced frames in Section E1.4.

5. Retrofit Measures

Seismic retrofit measures for steel truss diaphragms shall meet the requirements of this section, Section B3, and ASCE/SEI 41.

G5. ARCHAIC DIAPHRAGMS—SHALLOW BRICK ARCHES SPANNING BETWEEN STRUCTURAL STEEL FLOOR BEAMS

1. General

Archaic diaphragms in structural steel buildings are those consisting of shallow masonry arches that span between structural steel or wrought iron beams, with the arches packed tightly between the floor beams to provide the necessary resistance to arch thrust.

2. Stiffness

2a. Linear Analysis Procedures

Existing archaic diaphragms shall be modeled as a horizontal diaphragm with the equivalent thickness of masonry arches and concrete fill. Modeling of the archaic diaphragm as a truss with structural steel or wrought iron beams as tension components and arches as compression components is permitted. The stiffness of archaic diaphragms shall be considered in determining the distribution of seismic forces to vertical components. Analysis results shall be evaluated to verify that diaphragm response remains elastic as assumed.

Interaction of new and existing components of strengthened diaphragms shall be evaluated by checking the strain compatibility of the two classes of components in cases where new structural components are added as part of a seismic retrofit. Load transfer mechanisms between new and existing diaphragm components shall be considered in determining the stiffness of the strengthened diaphragm.

2b. Nonlinear Analysis Procedures

Response of archaic diaphragms shall remain elastic unless otherwise approved by the AHJ.

3. Strength

Member strengths of archaic diaphragm components are permitted to be determined assuming that no tension strength exists for all components except for structural steel or wrought iron beams. Gravity load effects shall be included for components of these diaphragms. Force transfer mechanisms between the various components of the diaphragm, and between the diaphragm and the frame, shall be evaluated to verify the completion of the load path.

4. Permissible Performance Parameters

Archaic diaphragms shall be considered force-controlled. For life safety or lower performance levels, diaphragm deformations and displacements shall not lead to a loss of bearing support for the components of the arches. For higher performance levels than life safety, the deformation caused by diagonal tension shall not result in the loss of the load transfer mechanism. Deformations shall be limited below the threshold of deflections that cause damage to other components, either structural or nonstructural, at specified performance levels. These values shall be established in conjunction with those for structural steel or wrought iron frames.

5. Retrofit Measures

Seismic retrofit measures for archaic diaphragms shall satisfy the requirements of this section, Section B3, and ASCE/SEI 41.

G6. CHORD AND COLLECTOR ELEMENTS

1. General

Structural steel framing that supports the diaphragm and frames either the perimeter of the diaphragm, an interior opening, a discontinuity, or a reentrant corner, are permitted to be considered as chord elements. Structural steel framing that serves to transfer force between diaphragms and members of the lateral force-resisting system, or distributes forces within the diaphragm or seismic force-resisting system, are permitted to be considered to be collector elements. Where structural concrete is present, additional slab reinforcement is permitted to provide tensile strength while the slab carries chord or collector compression. The structural steel framing that transfers lateral loads shall be attached to the deck with spot welds by steel headed stud anchors or by other approved methods.

2. Stiffness

Modeling assumptions specified for equivalent structural steel frame members in these Provisions shall be used for chord and collector elements.

3. Strength

The strength of structural steel chords and collectors shall be as specified in Section C3 for members subjected to combined axial force and flexure, and using the appropriate expected or lower-bound properties as provided in Chapter A. Chord and collector connections shall be considered force-controlled. The strength of steel reinforcing bars embedded in concrete slabs acting as chords or collectors shall be determined in accordance with the requirements of ASCE/SEI 41, Chapter 10.

4. Permissible Performance Parameters

Inelastic action in chords and collectors is permitted if it is permitted in the diaphragm. Where such actions are permissible, chords and collectors shall be considered deformation-controlled. The component capacity modification factors, m, shall be taken from applicable components in Chapter C, and inelastic permissible performance parameters shall be taken from components of moment frames with fully restrained beam-to-column moment connections in Chapter D. Where inelastic action is not permitted, chords and collectors shall be considered force-controlled components. Where chord and collector elements are force-controlled, the force-controlled action caused by gravity loads and earthquake forces, Q_{UF}, need not exceed the total force that can be delivered to the component by the expected strength of the diaphragm or the vertical components resisting seismic forces. For life safety or lower performance levels, the deformations and displacements of chord and collector components shall not result in the loss of vertical support. For higher performance levels than life safety, the deformations and displacements of chords and collectors shall not impair the load path.

Welds and connectors joining the diaphragms to the chords and collectors shall be considered force-controlled. If all connections meet the permissible performance parameters, the diaphragm shall be considered to prevent buckling of the chord member within the plane of the diaphragm. Where chords or collectors carry gravity loads in combination with seismic loads, they shall be designed as members with combined axial load and flexure in accordance with Chapter D.

5. Retrofit Measures

Seismic retrofit measures for chord and collector elements shall satisfy the requirements of this section, Section B3, and ASCE/SEI 41.

CHAPTER H
STRUCTURAL STEEL PILE FOUNDATIONS

H1. GENERAL

A pile provides strength and stiffness to the foundation either by bearing directly on soil or rock, by friction along the pile length in contact with the soil, or by a combination of these mechanisms. Foundations shall be evaluated as specified in ASCE/SEI 41, Chapter 8. Concrete components of foundations shall be evaluated as specified in ASCE/SEI 41, Chapter 10. The evaluation and design of structural steel piles shall comply with the requirements of these Provisions.

H2. STIFFNESS

If the pile cap is below grade, the foundation stiffness from the pile cap bearing against the soil is permitted to be represented by equivalent soil springs derived as specified in ASCE/SEI 41, Chapter 8. Additional stiffness of the piles is permitted to be derived through bending and bearing against the soil. For piles in a group, the reduction in each pile's contribution to the total foundation stiffness and strength shall be made to account for group effects. Additional requirements for determining the stiffness shall be as specified in ASCE/SEI 41, Chapter 8.

H3. STRENGTH

Except in sites subject to liquefaction of soils, it is permitted to neglect buckling of portions of piles embedded in the ground. Flexural demands in piles shall be determined either by nonlinear methods or by elastic methods for which the pile is treated as a cantilever column above a calculated point of fixity.

H4. PERMISSIBLE PERFORMANCE PARAMETERS

The permissible performance parameters for the axial force and maximum moments on the pile shall be as specified for a structural steel column in Section C3.4, where the lower-bound axial compression and flexural strengths shall be computed for an unbraced length equal to zero for those portions of piles that are embedded in nonliquefiable soils.

Connections between structural steel piles and pile caps shall be considered force-controlled.

H5. RETROFIT MEASURES

Seismic retrofit measures for structural steel pile foundations shall meet the requirements of this chapter, Section B3, and ASCE/SEI 41.

CHAPTER I

CAST AND WROUGHT IRON

I1. GENERAL

Framing that includes existing components of cast iron, wrought iron, or both is permitted to participate in resisting seismic forces in combination with concrete or masonry walls. Subject to the limitations of this chapter, existing wrought iron and cast iron components of structural framing are permitted to be assessed and designed to resist seismic forces or deformations as primary or secondary components.

User Note: The historical gray cast iron covered by these Provisions can be highly susceptible to tensile failures, although it is capable of providing significant compressive strength. The historical wrought iron covered by these Provisions is capable of developing yield strength and ductility in tension, although through-thickness tensile properties of wrought iron are noticeably reduced as compared to its tensile properties in the longitudinal (rolling) direction. The Commentary provides further discussion.

I2. STIFFNESS

The stiffness of cast and wrought iron components shall be calculated using elastic section properties and a modulus of elasticity of 20,000 ksi (138 000 MPa) for cast iron and 25,000 ksi (170 000 MPa) for wrought iron, unless a different value is obtained by testing or other methods approved by the authority having jurisdiction.

I3. STRENGTH

Component strengths shall be determined in accordance with Section B2 and the requirements of this section.

1. Cast Iron

Cast iron components shall not be used to resist tensile stresses from axial or flexural actions.

User Note: Because of the metallurgical nature of historical cast iron, beams made from historical cast iron are believed to have little or no seismic toughness and as a result should not be used to resist seismic actions. Due to similar concerns, the limitation on tensile stresses in historical cast iron columns is applicable to any tensile stress, whether arising from axial actions, flexural actions, or combined axial and flexural actions.

The lower-bound compressive strength, $Q_{CL} = P_{CL}$, of a cast iron column shall be determined from Equation I3-1:

$$P_{CL} = A_g F_{cr} \tag{I3-1}$$

The critical stress, F_{cr}, is determined as follows:

(a) When $\dfrac{L_c}{r} \leq 108$

$$F_{cr} = 17 \text{ ksi} \tag{I3-2}$$

$$F_{cr} = 117 \text{ MPa} \tag{I3-2M}$$

(b) When $\dfrac{L_c}{r} > 108$

$$F_{cr} = F_e \tag{I3-3}$$

where

A_g = gross area of the cross section, in.^{2} (mm^2)

F_e = elastic buckling stress, ksi (MPa)

$$= \frac{\pi^2 E_{ci}}{\left(\dfrac{L_c}{r}\right)^2} \tag{I3-4}$$

E_{ci} = modulus of elasticity of cast iron
= 20,000 ksi (138 000 MPa)

L_c = laterally unbraced length of column, in. (mm)

r = radius of gyration, in. (mm)

2. Wrought Iron

Lower-bound strength of a wrought iron component is permitted to be determined by considering the applicable provisions of the *Specification*, where the properties of wrought iron are substituted for the properties of structural steel. Lower-bound yield stress, F_{yL}, and lower-bound tensile strength, F_{uL}, shall be taken from Table A5.3, unless determined by testing in accordance with Section A5, and the modulus of elasticity shall be taken as 25,000 ksi (170 000 MPa).

I4. PERMISSIBLE PERFORMANCE PARAMETERS

Component permissible performance parameters shall be determined in accordance with Section B2 and the requirements of this section.

1. Cast Iron

Actions on cast iron components shall be force-controlled.

The ability of cast iron components to resist the deformations at the selected seismic hazard level shall be evaluated. In this evaluation, cast iron components are not permitted to resist tensile stresses.

2. Wrought Iron

Actions on wrought iron components shall be force-controlled.

The ability of wrought iron components to resist the deformations at the selected seismic hazard level shall be evaluated.

I5. RETROFIT MEASURES

Seismic retrofit measures for structural frames including cast iron components, wrought iron components, or both, shall satisfy the requirements of this chapter, Section B3, and ASCE/SEI 41.

COMMENTARY
on the Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings

August 1, 2022

(The Commentary is not a part of AISC 342-22, *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*, but is included for informational purposes only.)

INTRODUCTION

The Provisions are intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations, and limits of the Provisions.

The Provisions and Commentary are intended for use by design professionals with demonstrated engineering competence.

COMMENTARY SYMBOLS

The Commentary uses the following symbols in addition to the symbols defined in the Provisions. The section number in the righthand column refers to the Commentary section where the symbol is first used.

Symbol	Definition	Section
C_b	Lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the segment are braced	C3.4a.2.a.2
F_{avg}	Average of test values, ksi (MPa)	A5.3b
F_{min}	Equivalent specified minimum strength, ksi (MPa)	A5.3b
K_θ	Equivalent rotational spring stiffness, kip-in./rad (N-mm/rad)	C5.2a.2
L	Length of beam, in. (mm)	C1
L_1, L_3	Unrestrained gusset plate lengths to the nearest adjacent member at Whitmore width ends, in. (mm)	C7.2
L_2	Unrestrained gusset plate lengths to the nearest adjacent member at Whitmore width center, in. (mm)	C7.2
M	Bending moment, kip-in. (N-mm)	C3.4a.2.a.1
M_{CE}	Expected flexural strength of connection, kip-in. (N-mm)	C5.2a.2
M_p	Plastic bending moment, kip-in. (N-mm)	C3.4a.2.a.1
M_{pc}	Plastic flexural strength of the cross section in the presence of axial force (compression or tension), kip-in. (N-mm)	C3.4a.2.a.1
M_{pce}	Expected plastic flexural strength of the cross section in the presence of an axial force, P, kip-in. (N-mm)	D2.2
M_{pcx}	Plastic flexural strength of the cross section about the major principal axis (x-axis) in the presence of axial force (compression or tension), kip-in. (N-mm)	C3.4a.2.a.1
M_{pcy}	Plastic flexural strength of the cross section about the minor principal axis (y-axis) in the presence of axial force (compression or tension), kip-in. (N-mm)	C3.4a.2.a.1
M_{pe}	Flexural yield strength of the cross section, kip-in. (N-mm)	C1
M_{pe}	Expected plastic flexural strength of the cross section in the absence of axial force, kip-in. (N-mm)	D2.2
M_{px}	Plastic flexural strength of the cross section about the x-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.1
M_{py}	Plastic flexural strength of the cross section about the y-axis in the absence of axial force, kip-in. (N-mm)	C3.4a.2.a.1
M_x	Bending moment about the x-axis, kip-in. (N-mm)	C3.4a.2.a.1
M_y	Bending moment about the y-axis, kip-in. (N-mm)	C3.4a.2.a.1
P	Axial force (compression or tension), kips (N)	C3.4a.2.a.1
P_{CLx}	Lower-bound compressive strength in the plane of bending, kips (N)	C3.4a.2.a.2
P_{CLy}	Lower-bound compressive strength out of the plane of bending, kips (N)	C3.4a.2.a.2
P_E	Elastic critical buckling strength of a member in the plane of bending, kips (N)	D2.2
P_y	Axial yield strength, kips (N)	C3.4a.2.a.1

Z	Plastic section modulus taken about the axis of bending, in.^{3} (mm^3)	C3.4a.2.a.1
e	Length of EBF link, in. (mm)	C2.1
k	Lower tolerance limit factor, a function of n, p, and γ	A5.3b
n	Number of samples (statistical sample size)	A5.3b
p	Proportion of test data falling above the lower limit	A5.3b
Δ	Total displacement, in. (mm)	C1
Δ_y	Yield displacement, in. (mm)	C1
α	Exponent for nonlinear yield surface	C3.4a.2a.1
β	Exponent for nonlinear yield surface	C3.4a.2a.1
γ	Confidence interval	A5.3b
γ	Angular shear deformation, rad	C1
γ_y	Angular shear yield deformation, rad	C1
σ_{test}	Standard deviation of the sample of test values	A5.3b

CHAPTER A
GENERAL PROVISIONS

A1. SCOPE

The *International Existing Building Code* (IEBC) (ICC, 2021a) outlines the requirements for seismic evaluation and retrofit (or rehabilitation) of an existing building to improve its seismic lateral-force resistance. The IEBC provides a performance-based seismic design (PBSD) approach that explicitly references the evaluation and retrofit procedures prescribed in *Seismic Evaluation and Retrofit of Existing Buildings*, ASCE/SEI 41 (ASCE, 2017), hereafter referred to as ASCE/SEI 41. ASCE/SEI 41, Chapter 9, *Steel and Iron*, prescribes the modeling requirements and parameters, and the permissible strengths and deformations for primary and secondary structural steel, composite, wrought iron, and cast iron components subject to seismic forces and deformations.

These Provisions are intended to be the reference provisions to be used in the development of Chapter 9 in ASCE/SEI 41 for buildings with structural steel, composite, cast iron, and wrought iron components. Therefore, these Provisions are intended to be used in conjunction with ASCE/SEI 41 and not as stand-alone provisions. If the seismic retrofit (or rehabilitation) work is required by the IEBC or the *International Building Code* (IBC) (ICC, 2021b) to satisfy the provisions for a new building, these Provisions are not to be used unless approved by the Authority Having Jurisdiction.

These Provisions and associated commentary were developed from the provisions of ASCE/SEI 41, Chapter 9, *Steel and Iron*. The provisions from ASCE/SEI 41, Chapter 9, have been reorganized and formatted to be consistent with other AISC ANSI-approved standards. In addition to reorganization, editorial changes have been made to improve the user-friendliness of these Provisions. The intent of the technical provisions of ASCE/SEI 41, Chapter 9, has remained unchanged. It is the expectation that the next version of ASCE/SEI 41 will incorporate these Provisions as the portion of Chapter 9 dealing with structural steel, composite, wrought iron, and cast iron components.

There are some instances where the provisions in ASCE/SEI 41, and hence these Provisions, differ from other AISC standards. It is the expectation that in time these differences will either be rectified or commentary provided where needed to explain and clarify any differences for the user of these documents.

Techniques for repair of earthquake-damaged components are not included in ASCE/SEI 41 or in these Provisions. The design professional is referred to SAC Joint Venture publications FEMA 350 (FEMA, 2000a), FEMA 351 (FEMA, 2000b), FEMA 352 (FEMA, 2000c), and FEMA 353 (FEMA, 2000d) for information on design, evaluation, and repair of damaged steel moment-resisting frame structures.

Great care should be exercised in selecting the appropriate retrofit approaches and techniques for application to historic buildings to preserve their unique characteristics.

These Provisions are not intended for new construction projects, nor for the retrofit of structures not designed to resist seismic loading.

A2. REFERENCED SPECIFICATIONS, CODES, AND STANDARDS

These Provisions cite ASTM standard specifications that have been withdrawn, meaning the standard specification is considered obsolete and is no longer maintained by ASTM. Withdrawn standard specifications may be available from ASTM; however, availability may be limited. As an alternative, AISC Design Guide 15, *Rehabilitation and Retrofit* (Brockenbrough and Schuster, 2018), provides historical summaries of ASTM standard specifications for structural steel, including specified minimum yield stress and tensile strength values. Similarly, these Provisions cite past versions of the AISC *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341 (AISC, 2022a), hereafter referred to as the *Seismic Provisions*, copies of which may be obtained from AISC.

A4. DOCUMENT REVIEW AND CONDITION ASSESSMENT

1. General

ASCE/SEI 41, Section 6.2, includes provisions for determining a required level of knowledge as one of three categories: Minimum, Usual, or Comprehensive. As summarized in Table C-A4.1, the required level of knowledge then determines the data collection requirements for the condition assessment specified in Section A4, the material properties specified in Section A5, and other collected data as specified by ASCE/SEI 41, Section 6.2, and elsewhere in ASCE/SEI 41. The existence or absence of construction documents, such as design drawings, among other documents, also influences the degree of data collection. Detailed requirements and additional guidance for data collection can be found in ASCE/SEI 41, Sections 3.2 and 6.2, and the associated ASCE/SEI 41 Commentary sections.

Primary and secondary components of buildings include columns, beams, braces, connections, and link beams. Components may also appear in diaphragms. Columns, beams, and braces may be built up from plates, angles, and channels, used in various combinations, and connected together with rivets, bolts, or welds. Connections are considered to be a component in these Provisions; connections are composed of connectors, such as rivets, bolts, and welds, and connecting elements, such as plates and angles.

The extent of condition assessment that these Provisions require to be accomplished is related to availability and accuracy of construction and as-built records, the quality of materials used, the quality of construction performed, and the physical condition of the structure. As envisioned by these Provisions, the condition assessment necessarily includes a search for construction and as-built records and a detailed review of those records that are located. If original structural design drawings are not included among the available construction documents, or if the available documents show incomplete information, ASCE/SEI 41, Section 6.2, requires that field survey

TABLE C-A4.1
Selected Data Collection Requirements Excerpted from ASCE/SEI 41

Category of Data to Be Collected	Required Level of Knowledge					
	Minimum		**Usual**		**Comprehensive**	
Testing	No tests		Usual testing		Comprehensive testing	
Drawings	Design drawings	Field survey drawings prepared in absence of design drawings	Design drawings	Field survey drawings prepared in absence of design drawings	Design drawings	Field survey drawings prepared in absence of design drawings
Condition Assessment (Section A4)	Visual (Section A4.2)	Comprehensive (Section A4.3)	Visual (Section A4.2)	Comprehensive (Section A4.3)	Visual (Section A4.2)	Comprehensive (Section A4.3)
Material Properties (Section A5)	From design drawings (or other design documents)	From default values (Section A5.2)	From design drawings (or other design documents) and tests	From usual tests (Section A5.4b)	From design drawings (or other design documents) and tests	From comprehensive tests (Section A5.4c)

Note: This table is a partial excerpt of ASCE/SEI 41, Table 6-1. These Provisions provide data collection requirements for only some categories of data. Refer to ASCE/SEI 41, Section 6.2, for complete data collection requirements for all categories of data, including additional categories not shown here. Where these Provisions provide requirements for data collection, ASCE/SEI 41 may impose additional requirements.

drawings be prepared. The design professional is encouraged to research and acquire all available records from the original construction.

Direct visual inspection provides the most valuable information because it can be used to identify configuration issues, it allows measurement of component dimensions, and it identifies the presence of degradation. The continuity of load paths may be established by viewing components, including connections in particular. From visual inspection, the need for other test methods to quantify the presence and degree of degradation may be established.

Accessibility constraints may necessitate the use of instruments, such as a fiberscope or video probe, to help avoid damage to covering materials. The knowledge and insight gained from the condition assessment is invaluable for understanding load paths and the ability of components to resist and transfer loads. The degree of assessment performed also affects the knowledge factor, which is discussed in Section B1.2.

These Provisions require that the physical condition of existing components be examined for degradation, including connectors and connecting elements. Degradation may include environmental effects (such as corrosion, fire damage, and chemical attack) or past or current loading effects (such as overload, damage from past earthquakes, fatigue, and fracture). The condition assessment should also examine for configurational problems observed in recent earthquakes, including effects of discontinuous components, improper bolting and welding, and poor fit-up. Often, unfinished interior areas, such as mechanical rooms, interstitial spaces, attics, and basements, provide suitable access to structural components and can give a general indication of the condition of the general structure. Invasive inspection of critical components, particularly connectors and connecting elements within connections, is typically necessary, if not explicitly required by these Provisions.

Component orientation, plumbness, and physical dimensions are to be confirmed during an assessment. Connections between components are critical to the performance of structural systems and as a result, require special consideration and evaluation. The load path for the system is to be determined, and each connection in each load path is to be evaluated. This evaluation includes diaphragm-to-component and component-to-component connections. FEMA 351 (FEMA, 2000b) provides recommendations for inspection of welded structural steel moment frames. The strength and deformation capacity of connections are to be checked where the connection is attached to one or more components that are expected to experience significant inelastic response. Detailed inspections are required for anchorages between exterior walls and the roof and floor diaphragms that are used to resist out-of-plane loading.

The condition assessment also affords an opportunity to review other conditions that may influence structural elements and systems and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the structural system in question, including infills, neighboring buildings, and equipment attachments. Limitations to assessment that are posed by existing coverings, wall and ceiling finishes, insulation, infills, and other conditions should also be defined such that prudent retrofit measures may be planned.

For structural steel and wrought iron elements encased in concrete, it may be more cost-effective to provide an entirely new seismic force-resisting system than to undertake a visual inspection by removal and subsequent repair of the concrete encasement.

The physical condition of components may also dictate the use of certain destructive and nondestructive test methods. If structural elements are covered by well-bonded fireproofing materials or are encased in durable concrete, where the covering or encasing material is confirmed to not include significant amounts of constituents that might promote corrosion, it is likely that their condition is suitable. However, local removal of these materials at connections should be performed as part of the assessment. The scope of this removal effort is dictated by the component and element design. For example, in a braced frame, exposure of several key connections may suffice if the physical condition is acceptable and the configuration matches the design drawings. However, for moment frames, it may be necessary to expose more connection locations because of varying designs and the critical nature of the connections. Refer to FEMA 351 (FEMA, 2000b) for inspection of welded moment frames. For instances where no construction documents exist, it is necessary to expose or indirectly view all primary connections for documentation of actual connection details.

2. Visual Condition Assessment

The method of connecting the various components of the structural system is critical for the performance of the system. The type and character of the connections are to be determined by a review of the structural design drawings and a field verification of the connections and their condition. Connections of the same connection type are characterized by similar limit states and similar modes of nonlinear behavior. The engineer's judgment is required to determine how many different connection type groups are suitable for a given building.

2a. Buildings Previously Subjected to Ground Shaking

It is important to inspect, as part of a seismic evaluation or retrofit project, existing structural steel buildings of any configuration that have been subjected to significant ground shaking in the past. Experience from the 1994 Northridge Earthquake showed that steel buildings can sustain significant damage that is not evident from a cursory building walkthrough. Numerous buildings experienced welded connection fractures (FEMA, 2000e), but these fractures could be observed only after removing finishes and fireproofing. While moment frame buildings were the focus of most of the damage in Northridge, damage was also observed in braced frame buildings (Kelly et al., 2000). Several buildings were found to have similar damage following the 1989 Loma Prieta Earthquake, but the damage was not discovered until several years after the earthquake (FEMA, 2000e).

A threshold of 0.2g peak ground acceleration (PGA), where g is the acceleration of gravity = 386 in./s^2 (9 810 mm/s^2), is used to identify when a building has been subjected to strong ground shaking. This value is based on correlations between an intensity of IV on the modified Mercalli intensity (MMI) scale, where structural damage occurs, and PGA (FEMA, 2000c) and has been correlated with the presence of steel building damage in past earthquakes, primarily the Northridge Earthquake (FEMA, 2000e). If there is documentation of past earthquake(s) affecting the region, but the PGA is not known, the user should consider alternate metrics, like MMI, to determine whether to perform an assessment for past earthquake damage. Additionally, the user may wish to consider inspection if there has been a past earthquake, but the PGA is less than 0.2g. Catalog records of past seismic events are maintained by the United States Geological Survey and regional organizations that track seismic activity.

Typically, it will not be practical to inspect every member and connection to determine if there has been damage. A subset of the members and connections should be determined based on the prevalence of the members and connection details. FEMA 352 (FEMA, 2000c) is one document that provides a suitable inspection protocol procedure for welded steel moment frame buildings. FEMA 352 provides recommended numbers of connections to inspect that is based on the total number of connections of a specific type in the structure. It also provides a means by which analysis can be used to identify elements that should be inspected in addition to randomly selecting connections. The procedures in FEMA 352 can be extrapolated to members and connections for systems other than moment frames as one means of determining a statistically significant number of locations to inspect.

4. Component Properties

Structural elements of the seismic force-resisting system are composed of primary and secondary components, which collectively define element strength and resistance to deformation. Behavior of the components—including shear walls, beams, diaphragms, columns, braces, and their connections—is dictated by physical properties, such as cross-sectional area; material grade; thickness, depth, and slenderness ratios; lateral-torsional buckling resistance; and connection details.

The actual physical dimensions of components should be measured. Modifications to components should be noted, including holes. The presence of corrosion, other forms of deterioration, and distortion or deformation should be noted.

These primary component properties are needed to properly characterize building performance in the seismic analysis. The starting point for establishing component properties should be the available construction documents. Preliminary review of these documents should be performed to identify vertical-load (gravity-load) and seismic force-resisting elements and systems and their critical components and connections. Site inspections should be conducted to verify as-built conditions and to ensure that remodeling has not changed the original design concept. In the absence of a complete set of construction documents, the design professional is to thoroughly inspect the building to identify these elements, systems, and components and prepare field-survey drawings to document inspection findings, as indicated in Section A4.

A5. MATERIAL PROPERTIES

In addition to structural steel, the materials covered by these Provisions include historical cast iron and historical wrought iron. Over time, cast iron was gradually replaced by wrought iron, and then wrought iron was replaced by steel. Cast iron was often used for columns in construction from the 1850s to the 1890s, with limited use continuing through the 1920s. Wrought iron was used in structural applications from the 1870s to the 1890s, with limited use as tension rods continuing into the 1930s. The Commentary to Chapter I provides additional historical perspective for cast iron and wrought iron.

The material used in building construction from the later 1890s through approximately 2000 is likely to be carbon steel with a specified minimum yield stress between 28 ksi (195 MPa) and 36 ksi (250 MPa). For wide-flange shapes, structural steel with a specified minimum yield stress of 50 ksi (345 MPa) was not commercially dominant until after 2000. Nonetheless, after 2000, structural steel with a specified minimum yield stress of 36 ksi (250 MPa) continued to be commercially dominant for plates, bars, and rolled shapes other than wide-flange. In any event, when assessing a structure constructed during the 1990s and the first decade of the 2000s, due consideration should be given to the possibility that, for wide-flange shapes in particular, the specified minimum yield stress of the structural steel could be either 36 ksi (250 MPa) or 50 ksi (345 MPa), because the commercial transition between the two grades took place over a period of many years.

The connectors in construction prior to about 1950 were usually carbon steel rivets and lower strength bolts, although the earliest rivets and bolts, predating the mid-

1980s, were made from wrought iron. The use of these connectors was later replaced by high-strength bolts and by welds. Manufacturing specifications for easily welded structural steel were not developed until the 1950s, and easily welded structural steel was not commercially dominant until the 1960s. The Commentary to Chapter B provides additional perspective on historical structural steel, particularly with respect to welding considerations.

Further historical perspectives on ferrous structural metals in general are given in FEMA 274 (FEMA, 1997b) Section C5.2, in AISC Design Guide 15, *Rehabilitation and Retrofit* (Brockenbrough and Schuster, 2018), and in Paulson (2013). AISC Design Guide 21, *Welded Connections—A Primer for Engineers* (Miller, 2017), discusses weldability of historical steels.

1. General

The predecessor documents to these Provisions, FEMA 273 (FEMA, 1997a), FEMA 356 (FEMA, 2000f), and ASCE/SEI 41 (ASCE, 2017), provided variable requirements for determination of lower-bound yield stress and tensile strength of structural steel, in some instances using specified minimum values and at other instances using values greater than specified minimum. The values greater than specified minimum were based on the rule-of-thumb "mean minus one standard deviation," which lacks statistical rigor because of the lack of consideration of an appropriate level of statistical confidence and the quantity of samples tested. Instead, these Provisions always establish lower-bound values for yield stress and tensile strength as specified minimum values based on information provided by the available construction documents, such as the specified minimum values listed in the standard specification used to manufacture the in-place steel as cited by the available construction documents. On occasion, available construction documents may numerically state the minimum values themselves; these minimum values may be taken as lower-bound values. The approach of using specified minimum material strengths as lower-bound material strengths provides an appropriate lower-bound component strength, particularly where component strength is highly correlated with material strength.

Depending upon the available information, or lack thereof, sampling and testing of the in-place structural steel to determine yield stress and tensile strength could be necessary. Where testing is used to establish yield stress and tensile strength, such as by tensile testing of samples extracted from the in-place structural steel, these Provisions provide for calculation of equivalent specified minimum values using reliability-based statistical analysis of the materials test data. Further discussion is provided later in this Commentary. Expected yield stress and tensile strength remain as mean values of test results when determined by tensile testing of samples.

In accordance with the requirements of ASCE/SEI 41, expected material properties should be used to determine component strengths associated with deformation-controlled actions, and lower-bound material properties should be used to determine component strengths associated with force-controlled actions.

Mechanical properties of component materials, including connector and connecting element materials, dictate the structural behavior of the component under load. Mechanical properties of greatest interest include the expected yield stress, F_{ye}, the lower-bound yield stress, F_{yL}, the expected tensile strength, F_{ue}, and the lower-bound tensile strength, F_{uL}, along with modulus of elasticity, ductility, and toughness. The chemical composition of the steel is important in determining weldability. Examination of the existing steel for unsound features, as described in the Chapter B Commentary, can also be important when welding to older, existing steel.

The seismic performance of existing components depends heavily on the condition of the in-place material. Consequently, these Provisions require an examination for deterioration of components as a necessary part of the condition assessment.

Component strengths may also be determined by testing of subassemblies in accordance with Section A6. Analysis of the results of tests on subassemblies of components is to be performed in accordance with Section A6, which in turn relies upon the requirements of ASCE/SEI 41, Section 7.6.

2. Default Material Properties

Section A5.2 includes default material properties that may be used without the need for testing only where permitted by these Provisions or by ASCE/SEI 41. Otherwise, material properties are to be determined by sampling and testing of in-place materials, and subsequent analysis of the test results, in accordance with Sections A5.3 and A5.4 and ASCE/SEI 41.

2a. Structural Steel Materials from 1901 and After

For structural steel materials from 1901 and after, default lower-bound yield stress and tensile strength as provided by Table A5.1 are to be taken as the specified minimum values from the applicable ASTM standard specification, or by specified minimum values as listed on the available construction documents. The edition year of the applicable standard specification relied upon for establishing the default values should be consistent with the date of construction of the building because the specified minimum values listed in a standard specification of a given designation may evolve over time. Where construction documents are not available, or where the available construction documents do not provide sufficient information to establish default values, default values are not available. In those instances, material strengths are to be determined by testing of samples removed from in-place materials in accordance with Section A5.3.

The applicable standard specification as determined from information provided in the available construction documents may be a withdrawn standard specification that is no longer maintained by ASTM. Withdrawn standard specifications may be obtained from ASTM, although their availability may be limited. AISC Design Guide 15, *Rehabilitation and Retrofit* (Brockenbrough and Schuster, 2018), provides summaries of specified minimum yield stress and tensile strength values from withdrawn ASTM standard specifications for structural steel. Table C-A5.1 provides an abridged summary derived from AISC Design Guide 15 and from review of historical standard specifications published by ASTM and its predecessor organizations. For rivet steel and bolts, either Appendix 5 of AISC *Specification for Structural Steel Buildings*, ANSI/AISC 360 (AISC, 2022b), hereafter referred to as the *Specification*, or AISC Design Guide 15 should be consulted.

TABLE C-A5.1
Specified Strength Properties from Withdrawn Editions of Selected ASTM Standard Specifications[a]

Issue Date[b]	Yield Stress, Minimum, psi (MPa)	Tensile Strength	
		Minimum, psi (MPa)	Maximum, psi (MPa)
ASTM A9, August 10, 1901	30,000[c] (210)	60,000 (410)	70,000 (480)
ASTM A9, August 16, 1909, 1913, 1914, 1916, 1921	27,500[c] (190)	55,000 (380)	65,000 (450)
ASTM A9, 1924–1932	30,000 (210)	55,000 (380)	65,000 (450)
ASTM A9, 1933–1938	33,000 (230)	60,000 (410)	72,000 (500)
ASTM A7, 1939–1960	33,000 (230)	60,000 (410)	72,000 (500)
ASTM A36, 1960–1999	36,000 (250)	58,000 (400)	80,000 (550)

[a]Values listed are for the grade “medium steel” or “structural steel” for rolled shapes.
[b]ASTM A9 was titled “Standard Specification for Structural Steel for Buildings.” Beginning in 1939, ASTM A9 for buildings was consolidated with ASTM A7 for bridges and issued under the single designation ASTM A7, “Standard Specification for Steel for Bridges and Buildings.” ASTM A36 is titled “Standard Specification for Structural Steel.”
[c]The specification requirement between 1901 and 1923 was that the actual yield point is to be at least one-half of the actual tensile strength from the mill test; the numeric value stated here for specified minimum yield strength is therefore one-half of the specified minimum tensile strength.

The translation factors for default expected strengths used by these Provisions are harmonized with the factors R_y and R_t, as found in past and current editions of the *Seismic Provisions* (AISC, 2022a), where R_y is the ratio of the expected yield stress to the specified minimum yield stress F_y, and R_t is the ratio of the expected tensile strength to the specified minimum tensile strength F_u. This approach is generally consistent with the translation factor approach used by ASCE/SEI 41. For structural steels produced according to standard specifications issued beginning in 1994, these Provisions rely upon tabulated values found in current and past versions of the *Seismic Provisions*. For standard specifications issued prior to 1994, explicit values for R_y and R_t are provided in Table A5.2. The values provided in Table A5.2 are the same as those provided in ASCE/SEI 41 for the specification years indicated, with some adjustments that are based upon reanalysis of the historical databases of mill certificate data upon which these values are based. These databases typically include several thousand data points for the type and grade of steel indicated.

Where a standard specification is not listed in Table A5.2, or for a listed standard specification having a date that is outside of the date range indicated in Table A5.2, an appropriate database was not available for analysis, and as a result, values for R_y and R_t are not provided. In this analysis of databases, the default values listed in Table A5.2 presume that the test samples for wide-flange shapes have been extracted from the flanges, whereas prior to 1997, mill practice was to extract samples from the web. Older yield strength test results from tests on samples removed from the web have been adjusted using the correction factor, R, given in *Specification* Commentary Appendix 5, Section 5.2.2.

2b. Structural Steel Materials from Before 1901, Wrought Iron Materials, and Cast Iron Materials

Structural steel materials from before 1901, wrought iron materials, and cast iron materials were not always manufactured to industry-based consensus manufacturing standards. Consequently, the default values given in Table A5.3 are conservative by intent in recognition of the possible large variation in tensile properties that may be observed in these particular historical structural ferrous metals. Additional discussion regarding historical wrought iron and historical cast iron can be found in the Chapter I Commentary.

3. Testing to Determine Properties of In-Place Materials

The extent of testing of in-place materials required by these Provisions depends on the availability and accuracy of construction and as-built records, the quality of materials used, the construction performed, and the physical condition of the in-place materials. Data such as the properties and grades of material used for components and connectors as obtained from construction documents may be effectively used to reduce the amount of required testing of in-place materials; under certain circumstances, testing of in-place materials may be completely eliminated and default material properties may instead be assumed as in-place material properties. Consequently, the design professional is encouraged to seek out, acquire, and review all available records from the original construction.

FEMA 274 (FEMA, 1997b) provides information and references for several test methods.

To obtain the desired mechanical properties of in-place materials, it is often necessary to use destructive testing methods. Sampling of materials should take place in regions of a component where the required strength is less than the available strength determined with consideration of the lost section caused by sampling. Potential sampling locations include flange tips at the ends of simply supported beams and external edges of plates.

Test samples extracted from an existing structure may not always be taken from the sampling location and orientation as prescribed by ASTM A6/A6M (ASTM, 2019a) for the mill test. For example, a longitudinal sample extracted from the flange is most commonly prescribed for wide-flange shapes by ASTM A6/A6M. However, field samples might be more readily extracted from the web of the existing, in-place steel member. The Commentary to *Specification* Appendix 5 provides information for adjusting yield strength test results of web samples to be comparable to results from tests on web samples. Additionally, the test sample may not have been extracted from the in-place steel in the orientation as specified by ASTM A6/A6M, which is along the rolling direction of the shape. It is possible that field samples may have been instead extracted with an orientation that is transverse to the rolling of the structural shape. Orientation of the test sample is important because the same piece of steel when tested in the transverse direction (perpendicular to the rolling direction) will have slightly different tensile properties than when tested in the longitudinal direction (along the rolling direction, as required by ASTM A6/A6M).

It may be necessary to repair components after removal of samples, as stated in Section A5.3a. It is important that the area where the samples are removed from the component be free from characteristics that could create stress concentrations that adversely affect the strength or ductility in the region of the component affected by sample removal. To prevent adverse effects, sharp corners should be avoided and the area where cutting occurred to remove a sample should be ground smooth. The component affected by material removal should be repaired such that it has at least the same strength and ductility as it did prior to the removal of the sample.

It is inadvisable to remove samples from locations on components where inelastic deformations (plastic hinges) are anticipated to occur as a result of seismic events. This is because it is very challenging to design and implement a repair to the sampled component that will provide strength, ductility, and toughness that are comparable to that provided by the original component prior to the alteration that resulted from sampling.

To mitigate the difficulties associated with sample removal and subsequent repair of the sample location, it may be possible to use default values for material properties in lieu of material sampling and testing, where so permitted by these Provisions and ASCE/SEI 41. The use of default properties in lieu of sample removal and subsequent testing may reduce costs and risks associated with sampling, testing, and repair of the sampling location.

Of greatest interest to seismic system performance are the expected yield stress and expected tensile strength of the installed materials. Notch toughness of structural steel and weld material is also important for connections that undergo cyclic loadings and deformations during earthquakes. Compositional analysis and metallographic examination of the steel can, respectively, provide information on weldability of the steel and potential for lamellar tearing of the steel caused by through-thickness stresses. Virtually all elastic and inelastic limit states for a component are related to yield stress, tensile strength, and modulus of elasticity. Past research and accumulation of data by industry groups have resulted in published material mechanical properties for most primary metals and their dates of fabrication. Section A5.2 provides default properties. This default information may be used, together with tests from recovered samples, to establish expected strength properties for use in component strength and deformation analyses.

Review of other properties derived from laboratory tests, such as impact, fracture, and fatigue, is generally not needed for structural steel component capacity determination, but such tests may be required for historical materials and for connection evaluation. These properties may not be needed in the analysis phase if significant retrofit measures are already known to be required.

To quantify material properties and analyze the performance of welded moment connections, more extensive sampling and testing of welds may be necessary (FEMA, 2000b). This testing may include base and weld material chemical and metallurgical evaluation, expected strength determination, hardness, and Charpy V-notch testing of the heat-affected zone and neighboring base metal, and other tests depending on connection configuration. Weld samples should consist of both neighboring base and weld metal to allow for a determination of the composite strength of the welded connection.

One approach frequently used for determination of the tensile properties of existing steel components is by mechanical testing on samples of the existing steel extracted from the structure, as described earlier in this Commentary. At times, however, material removal may be problematic because the strength of the structure may be temporarily reduced due to the removal of samples, and replacement material may subsequently need to be provided, as stated in Section A5.3a. Where material removal is problematic, such as is usually the case for weld metal removal, hardness testing may be a desirable alternative. Hardness values are a reasonable predictor of tensile strength, but hardness testing cannot determine yield stress or ductility (elongation). Precise conversions of hardness values to tensile strengths are not possible, but hardness testing can be used to obtain a general understanding of the strength of the existing steel. Hardness testing can be performed in-situ on existing steel. If hardness testing is instead to take place in the laboratory, samples of the existing steel smaller than the samples usually extracted for tensile testing can be utilized.

Another consideration is that hardness testing results are available immediately after the testing is performed. Because of the low cost and essentially nondestructive nature of in-situ hardness testing, multiple steel members can be quickly and economically tested: beams and columns; flanges and webs; members of different foot-weights; angles and wide-flange shapes; connectors, such as rivets and bolts; and so forth.

Many hardness scales have been developed over the years, and there are many methods of testing. Brinell hardness testing is well suited for in-situ testing that is performed to determine the tensile strength of the steel. Brinell hardness testing measures the hardness over an area that results in an averaging of the localized hardness. Other hardness testing methods, like Rockwell, are better for identification of localized conditions, such as weld heat-affected zone properties; Brinell is better suited for identification of the tensile strength of the base metal.

Standard Test Method for Brinell Hardness of Metallic Materials, ASTM E10 (ASTM, 2018), prescribes testing procedures. In-situ hardness testing can be performed in accordance with *Standard Test Methods for Rockwell and Brinell Hardness of Metallic Materials by Portable Hardness Testers*, ASTM E110 (ASTM, 2014). *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*, ASTM A370 (ASTM, 2019b), can be used to convert hardness values from one scale to another. ASTM A370 and SAE J417 (SAE, 2018) provide conversions of hardness values to estimated steel tensile strengths.

3b. Interpretation of Test Results

Where yield stress and tensile strength of the in-place structural steel are determined by tensile testing of samples of steel extracted from the existing structural steel, these Provisions require that the lower-bound yield stress and lower-bound tensile strength be taken as an equivalent specified minimum value as determined from statistical analysis of test values, such that it is 90% confident that 95% of the test values fall above the equivalent specified minimum value. This statistical approach is called a lower tolerance-limit analysis, which, when using symbols that are typically applied to structural steel, may be stated in equation form as follows:

$$F_{min} = F_{avg} - k\sigma_{test} \tag{C-A5-1}$$

where

F_{avg} = average of test values, ksi (MPa)
F_{min} = equivalent specified minimum strength, ksi (MPa)
k = lower tolerance limit factor, a function of n, p, and γ
n = number of samples (statistical sample size)
p = proportion of test data falling above the lower limit
γ = confidence interval
σ_{test} = standard deviation of the sample of test values

This general approach, which assumes that the test results are normally distributed but can be readily adapted to a log-normal distribution, is used elsewhere for various kinds of structural materials and is consistent with the reliability-based approach for design strength of structural steel as utilized by the *Specification* (Paulson, 2013). The combination of confidence level ($\gamma = 0.90$) and proportion of test data ($p = 0.95$) falling above the lower limit, as used in these Provisions, provides reasonable values for the statistical analysis of results from tensile tests of samples of historical structural steel for statistical sample size, n, of six to eight and larger (Paulson, 2013).

Table C-A5.2 lists values for k at $\gamma = 0.90$ and $p = 0.95$ for number of samples, n, from 3 to 30. Values for the one-sided lower tolerance limit factor, k, may also be obtained from statistical handbooks (Odeh and Owen, 1980). Examination of the factors listed in Table C-A5.2 finds that the value of the factor increases significantly at relatively low numbers of samples. Consequently, to achieve practical values for equivalent specified minimum values, the number of samples in the data set to be analyzed should be at least six, with a number of samples of eight or larger being preferable. The standard deviation should be determined using the formula for the standard deviation of a sample of a population, not the standard deviation of a population. This is because the individual tests are obtained from only a limited number of representative components, not from each component in the entire population of all components.

Expected values of yield stress and tensile strength as determined from tensile test results are to be taken as the mean of test values. Even if the purpose of the tensile testing is to develop an expected value, the number of samples in the data set to be analyzed should again be at least six, with a number of samples of eight or larger again being preferable.

4. Extent of Testing of In-Place Materials

The extent of testing of the installed materials depends on the level of knowledge that is required (see Table C-A4.1) when assessing the building under consideration. For example, where construction documents are available for a building, the design drawings may identify the standard specification used for production of the materials

TABLE C-A5.2
One-Sided Tolerance Limit Factors, *k*, for Proportion of Data, *p*, 95% and Confidence Level, γ, 90%

Number of Samples, *n*	Factor, *k*	Number of Samples, *n*	Factor, *k*
3	5.311	17	2.272
4	3.957	18	2.249
5	3.400	19	2.227
6	3.092	20	2.208
7	2.894	21	2.190
8	2.754	22	2.174
9	2.650	23	2.159
10	2.568	24	2.145
11	2.503	25	2.132
12	2.448	26	2.120
13	2.402	27	2.109
14	2.363	28	2.099
15	2.329	29	2.089
16	2.299	30	2.080

installed in the building or may indicate specified minimum properties for the materials. As a result of this knowledge, for example, testing is not required for the particular case of usual testing. If specific materials information is not listed or if construction documents are not available, but the date of construction is known, some knowledge regarding materials likely used in the building can be obtained from published references that provide chronological listings of historical materials specifications, such as AISC Design Guide 15, *Rehabilitation and Retrofit* (Brockenbrough and Schuster, 2018). Absent any knowledge whatsoever, testing of the installed materials is required.

To quantify expected strength and other properties accurately, a minimum number of tests may be required to be conducted on representative components. As discussed previously in Commentary Section A5.3b, to provide for meaningful statistical analysis of test results, the minimum number of samples to be collected from a population of similar representative components should be at least six, with a number of samples of eight or larger being preferable.

The engineer should exercise judgment to determine how much variability of component sizes constitutes a significant change in structural material properties. It is likely that most of the sections of the same size within a building have similar material properties, because section shapes of the same nominal size designation likely were obtained from the same production heat of steel. Variation of material properties within a production heat is much less than variation of properties across multiple production heats. Differences in material properties are more likely to occur because of differences in size groups, differences in specified material properties [F_y of 36 ksi (250 MPa) versus 50 ksi (345 MPa)], and differences in section shapes. Where sampling is required, at a minimum, one tensile test sample should be removed from each nominal size designation of each wide-flange shape, angle, channel, hollow structural section, and other structural shape used as part of the components that resist significant seismic forces. Unless otherwise intended to assess variation of properties within a production heat of steel, replicate sampling from multiple members of the same nominal size designation should be avoided, so that the resulting statistical analysis of test results will represent variation of material properties across the multiple heats of steel used in the building. Additional sampling should be done where large variations in member sizes occur within the building and where the building was constructed in phases or over extended time periods where members may have come from different mills or from different batches of steel. Removal of coverings, including surface finishes, fireproofing, and other nonstructural materials, is generally required to facilitate test sample extraction and visual observations.

Material properties of structural steel vary much less than those of other construction materials. In fact, the expected yield stress and tensile strength are usually considerably higher than the specified minimum values. As a result, testing for material properties of structural steel may not be required. The properties of wrought iron are more variable than those of steel. The strength of cast iron components cannot be determined from small sample tests because component behavior is usually governed by inclusions in the cast iron and other manufacturing-related imperfections in the component. Nondestructive testing (NDT) of wrought iron and cast iron is complicated by their metallurgical structures; NDT techniques that are more commonly used with structural steel may be unsuccessful when applied to cast iron and wrought iron.

If ductility and toughness are required at or near an existing weld, the design professional may conservatively assume that no ductility is available, in lieu of testing. In this case, the welded joint would have to be modified if inelastic demands are anticipated and the possibility of fractures cannot be tolerated. Special requirements for welded moment frames are given in FEMA 351 (FEMA, 2000b).

The notch toughness of deposited weld metal depends on the filler metal as well as the welding parameters used when the weld was made, such as preheat and interpass temperatures, heat input levels, and other factors. Accurate measurement of the notch toughness of in-place welds is difficult. The typical method of evaluating weld metal toughness is with a Charpy V-notch (CVN) specimen. AWS A5 filler metal specifications and the AWS *Structural Welding Code—Steel* (AWS D1.1/D1.1M) (AWS, 2020a) have specific requirements associated with CVN testing, including the specimen orientation (perpendicular to the weld axis), specimen location with respect to the thickness of the plate, and speciman location with respect to the centerline of the weld. Weld metal extracted from an existing building will not likely permit testing in strict accordance with the filler metal specifications or AWS D1.1/D1.1M. Deviations may include CVN specimen orientation (longitudinal to the weld axis versus perpendicular), lateral location within the weld joint (i.e., the notch may not be in the weld centerline), or height within the weld joint (i.e., not at the mid thickness location). All such deviations will cause differences in the test results.

For testing in accordance with the AWS filler metal specifications or in accordance with AWS D1.1/D1.1M, it is typical to use full-sized CVN specimens [i.e., 0.394 in. (10 mm) × 0.394 in. (10 mm) bars that are 2.165 in. (55 mm) long]. Weld metal extracted from an existing building may not permit the use of full-scale CVN specimens, necessitating subsized specimens. Subsized CVN specimens are tested at lower temperatures to compensate for the reduced restraint associated with the smaller specimens (see AWS D1.1/D1.1M, clause 6.27.6).

These Provisions do not provide a standardized method for locating CVN specimens from weld metal extracted from existing buildings. Such details should be agreed upon by the engineer, contactor, and mechanical testing lab, with due consideration given to the conditions associated with the specimen removal site and the nature of any required material replacement.

To statistically quantify expected strength and other properties of in-place materials, these Provisions, along with ASCE/SEI 41, require that a minimum number of tests be conducted on materials from representative components. The minimum number of tests is established by considering available data from original construction, the type of structural system used, desired accuracy, and quality or condition of in-place materials. Visual access to the structural system also influences testing program definition. If a higher degree of confidence in results is desired, either the sample size should be determined using *Standard Practice for Calculating Sample Size to Estimate, With Specified Precision, the Average for a Characteristic of a Lot or Process*, ASTM E122 (ASTM, 2017), or the prior knowledge of material grades from Section A5.2 should be used in conjunction with approved statistical procedures. Design professionals may consider using Bayesian statistics and other statistical procedures contained in FEMA 274 (FEMA, 1997b) to gain greater confidence in the test results obtained from the sample sizes specified in this section.

CHAPTER B

GENERAL REQUIREMENTS OF COMPONENTS

These Provisions address components subject to seismically induced forces or deformations. Existing buildings tend to use braced frames, moment frames, shear walls, and other elements in various combinations that do not readily fit into a particular seismic force-resisting system (SFRS), as defined in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7 (ASCE, 2022), or the *Seismic Provisions* (AISC, 2022a). The frame assemblies identified in Chapters D and E do not require that the assembly be equivalent to a modern SFRS.

B1. GENERAL

1. Basis of the Analytical Model

Mathematical modeling of existing components depends on the design professional's knowledge of the condition of the structural system and material properties, as determined in accordance with Section A4 and Section A5, respectively. Certain damage—such as water staining, evidence of prior leakage, limited corrosion, and limited buckling—may not require consideration in the mathematical model. The design professional establishes the acceptability of such damage case-by-case based on capacity loss and deformation constraints. It may be necessary to modify both strength criteria and deformation permissible performance parameters to account for the damaged conditions of components. Degradation at connection points, in particular, should be carefully examined; significant capacity reductions may be involved, as well as a loss of ductility. Thickness variations due to manufacturing tolerances should not be interpreted as section loss.

B2. COMPONENT STIFFNESS, STRENGTH, AND PERMISSIBLE PERFORMANCE PARAMETERS

3. Strength Criteria

3a. Deformation-Controlled Actions

The relative magnitude of the component capacity modification factors, m, alone should not be interpreted as a direct indicator of performance. The stiffness of a component and the expected component strength, Q_{CE}, are also considered where evaluating expected performance.

3b. Force-Controlled Actions

When determining the lower-bound strength of a component whose limit state is governed by elastic buckling, a reduction factor of 0.85 is applied to the nominal strength in order to account for the uncertainties in strength of such behavior. This treatment stands in contrast to that of inelastic limit states, where the lower-bound strength is directly a function of the lower-bound yield stress, F_{yL}. The reduction factor applies to any elastic buckling limit state that is not directly proportional to F_{yL}. Without the reduction, there would be no difference in the component strength for a force-controlled and deformation-controlled action. The common limit states of lateral-torsional buckling for flexure or elastic flexural buckling for compression are two examples of this.

B3. RETROFIT MEASURES

2. Welds—General

When new welds are required to be made to existing steel components as part of a retrofit, the engineer is required by these Provisions to assess the weldability of the existing steel. Weldability, as defined in *Standard Welding Terms and Definitions; Including Terms for Adhesive Bonding, Brazing, Soldering, Thermal Cutting, and Thermal Spraying* (AWS A3.0M/A3.0) (AWS, 2020b), is "The relative ease with which a material may be welded to meet an applicable standard." Weldability is a qualitative term; steel with good weldability can be easily welded whereas steel with poor weldability may require specialized techniques, such as higher levels of preheat, post heat, and other measures. Additional information on weldability can be found in AISC Design Guide 21, *Welded Connections—A Primer for Engineers* (Miller, 2017). An assessment of the existing steel for soundness of the metal itself, as described later in this Commentary, should also be considered in many cases.

Once the weldability and soundness of the existing steel are established, the retrofit construction documents should specify requirements for weld procedure specifications (WPS) for new welds to the existing steel that are in conformance with these Provisions. The assessment could be as simple as a review of the available construction documents that describe the existing structural steel building, without any need for sampling or testing of the existing structural steel. The available information, or lack thereof, may indicate that sampling and testing of the structural steel to be welded is necessary.

For existing structures that were previously constructed by welding, the weldability of the existing steel can be established by observations, as described later in this Commentary, that it was successfully welded in the past. Modern welding processes and filler metals are better than those of the past, particularly in comparison with the bare electrodes that were used in the 1920s and 1930s. Modern buildings using modern steels that are prequalified in AWS D1.1/D1.1M (AWS, 2020a) need no investigation into weldability because these steels are permitted for use in AWS D1.1/D1.1M without any weldability investigations.

The greatest weldability challenge associated with projects governed by these Provisions involves existing structures that were riveted, not welded. For these situations, good weldability and soundness of the steel to be welded cannot be assumed, but neither are poor weldability and the presence of unsound features a certainty. Under such circumstances, weldability and soundness need to be determined on a case-by-case basis.

Table B3.1 provides several acceptable approaches for developing WPS for new welds to existing steel. For the majority of structural steel buildings constructed since

1950, it is anticipated that the approaches specified in Table B3.1 of these Provisions will result in the use of a prequalified WPS for making a new weld to an existing structural steel component. Nonetheless, there will be instances where the engineer will need to assess by testing the weldability of the existing steel to be welded as part of the requirements for developing a WPS.

For existing steels that were produced to standard specifications that are listed in *Structural Welding Code—Steel* (AWS D1.1/D1.1M) (AWS, 2020a) for use with prequalified WPS, no special requirements are specified by Table B3.1. Compliance with applicable provisions in AWS D1.1/D1.1M satisfies these Provisions. Because AWS D1.1/D1.1M is focused primarily on welding of new steels, structural steels manufactured according to now-obsolete standard specifications are not listed in the current edition of AWS D1.1/D1.1M for use with prequalified WPS; these steels are commonly referred to as "unlisted" steels. Consequently, Table B3.1 extends to certain selected unlisted steels the latitude to be welded using prequalified WPS, provided that preheat levels are increased. The selected steels are mostly limited to structural steels that were manufactured under some of the first standard specifications that were developed for structural steels with good weldability. Additionally, ASTM A7 structural steel in thickness not exceeding 1.5 in. (38 mm) that was produced after 1950 is also permitted to be welded under a prequalified WPS because ASTM A7 steel of this era is commonly held to have good weldability, as evidenced in part by the listing of ASTM A7 steel for use with prequalified welds in AWS D1.063 (AWS, 1963). AWS D1.0-63 is one of the predecessor standards to AWS D1.1/D1.1M. The thickness limit of 1.5 in. (38 mm) is based upon the limitations on welding to ASTM A7 steels as specified in AWS D1.063.

For unlisted steels that are produced to standard specifications that are not permitted for use with prequalified WPS by Table B3.1, or for steels of an unknown classification, these Provisions require that the engineer determine the welding requirements. This category necessarily includes steels with known good weldability that are prequalified by AWS D1.1/D1.1M but cannot be so designated by the engineer because the classification is not known due to the lack of documentation. For example, the existing steel may be ASTM A36/A36M, but the actual identity of the steel has been lost over time because original construction documents are not available to the engineer. Also included in the category of unlisted steels are those that have poor weldability due to limited control on the compositional characteristics; some steel in this category may be uneconomical to weld when executed under appropriate procedures.

Because of the wide range of possibilities with unlisted and unknown steels, the engineer is obligated by these Provisions to determine welding requirements. However, these Provisions do not provide requirements for how this task is to be completed because of the large number of variables involved and also because of the considerable engineering judgment to be exercised while completing this task. Instead, Tables C-B3.1, C-B3.2, C-B3.3a, and C-B3.3b provide general guidance to assist the engineer in fulfilling the obligation to determine welding requirements for steels that are not permitted to use prequalified WPS in accordance with these Provisions. The engineer may need to retain the services of a welding engineer or metallurgical specialist to determine appropriate welding requirements.

Table C-B3.1
Guidance for Metallographic Examination and Soundness Concerns

Existing Steel Classification	Considerations/Investigations	Welding Suggestions
The existing steel was produced to a known specification that is dated after 1950.	By this era, as-manufactured structural steels were generally sound, even though the steel was not necessarily produced according to a standard specification for structural steels that are intended to be weldable. One exception is structural steel manufactured using the acid-Bessemer process.	No compelling reason for metallographic examination for unsound features, although such an examination should be considered if compositional testing of the steel to be welded yields unusual results, or if the steel may have been manufactured using the acid-Bessemer process, either of which is potentially indicative of soundness concerns.
The existing steel was produced between 1930 and 1950 inclusive; steel manufacturing specification is known or unknown.	While many structural steels produced during this era were sound, there is nonetheless a chance that some structural steels from this era might contain significant inclusions, such as stringers and other features that may lead to soundness concerns.	If the existing structural steel in the completed connection could become stressed in the through-thickness direction, whether due to weld shrinkage or structural loading, or a new multipass weld is to be made to the existing steel, a metallographic examination of the existing steel for the presence of inclusions, such as stringers, and other soundness concerns is recommended. If inclusions or other soundness concerns are present, mechanical testing of samples extracted from the structure of the existing steel to be welded may demonstrate acceptable performance of the welded joint that is proposed to be used; otherwise, mechanical connections should be considered.
The existing steel was produced prior to 1930.	There is a significant probability that a structural steel from this era may contain significant inclusions, such as stringers, that could lead to lamellar tearing at welded joints. In structural steels produced during this era, such inclusions may occur frequently and may be relatively large. Other soundness concerns may also be present.	Regardless of the welded joint configuration, a metallographic examination of the existing steel should be undertaken to examine for the presence of inclusions, such as stringers, and other soundness concerns. If inclusions or other soundness concerns are present to an extent and severity such that susceptibility to lamellar tearing is heightened, mechanical connections should be considered.

Fusion welding began to be used for building construction in the 1920s. Before that, riveting was the primary joining method used for building construction. Weldability is governed by the chemical composition of the steel being joined, and the steel specifications of the 1920s and 1930s were not sufficiently restrictive in the allowable compositional ranges to ensure good weldability. Still, some steels produced during this era have compositions that may be easily welded, either by happenstance or by deliberate practices of the producing mills.

Table C-B3.2
Guidance for Determining Welding Requirements for New Welds to Existing Steel: for Existing Steel That Was Previously Welded

Existing Steel Classification	Considerations/Investigations	Welding Suggestions
The existing steel was produced in accordance with a known standard specification, and the steel is permitted to be used with prequalified WPS by Table B3.1 of these Provisions.	None.	Use prequalified WPS in accordance with clause 5 of AWS D1.1/D1.1M.
The existing steel was produced in accordance with a known standard specification, but the steel is not permitted to be used with prequalified WPS by Table B3.1 of these Provisions. Evidence is available showing that the existing steel was previously welded.	Weldability has been partially established by previous structural welding. It is recommended that at least 10% of the connections, or three (3) connections minimum, be inspected for lamellar tearing of the existing steel and for weld metal cracking. Existing complete-joint-penetration (CJP) groove welds should be inspected with Ultrasonic Testing (UT). Partial-joint-penetration (PJP) groove welds and fillet welds should be inspected with Magnetic Particle Testing (MT) or Penetrant Testing (PT). The strength level of the existing steel to be welded can be based on the tensile properties requirements listed in the standard specification.	If in the recommended inspection at least 80% of the inspected connections meet AWS D1.1/D1.1M acceptance criteria for statically loaded structures, then prequalified WPS in accordance with clause 5 of AWS D1.1/D1.1M may be used, but with preheat levels increased by 50°F (28°C) above the prequalified preheat level for steels with equivalent strength levels. If 80% passage rate is not achieved in the recommended inspection, preheat should be in accordance with AWS D1.1/D1.1M Annex B, based on the maximum compositional limits specified in the manufacturing specification. WPS should be qualified by test, using a sample of the existing steel to be welded that is extracted from the existing structure.
The standard specification used to produce the existing steel is unknown. Evidence is available showing that the existing steel was previously welded.	Weldability has been partially established by previous welding. It is recommended that at least 10% of the connections, or three (3) connections minimum, be inspected for lamellar tearing and weld metal cracking. Existing CJP groove welds should be inspected with UT. PJP groove welds and fillet welds should be inspected with MT or PT. Compositional analysis of the existing steel to be welded is recommended. Additionally, given unknown manufacturing specification, it is recommended that the strength level of the existing steel be estimated from hardness testing of the steel. Alternatively, samples of the existing steel to be welded could be extracted from the existing structure and subsequently tested for tensile properties.	If in the recommended inspection at least 80% of the inspected connections meet AWS D1.1/D1.1M acceptance criteria for statically loaded structures, then prequalified WPS in accordance with clause 5 of AWS D1.1/D1.1M may be used, but with preheat levels increased by 50°F (28°C) above the prequalified preheat level for steels with equivalent strength levels. If 80% passage rate is not achieved in the recommended inspection, preheat should be in accordance with AWS D1.1/D1.1M Annex B, based on the composition determined by testing of the steel to be welded. WPS should be qualified by test, using a sample of the existing steel to be welded that is extracted from the existing structure.

Table C-B3.3a
Guidance for Determining Welding Requirements for New Welds to Existing Steel That Was Not Previously Welded (Produced Under a Known Standard Specification)

Existing Steel Classification	Considerations/Investigations	Welding Suggestions
The existing steel was produced in accordance with a known standard specification, and the steel is permitted to be used with prequalified WPS by Table B3.1 of these Provisions.	None.	Use prequalified WPS in accordance with clause 5 of AWS D1.1/D1.1M.
The existing steel was produced in accordance with a known standard specification, but the steel is not permitted to be used with prequalified WPS by Table B3.1 of these Provisions. The existing steel was not previously welded.	The existing steel standard specification requirements meet all the mechanical and compositional limits of a steel that is permitted to be used with prequalified WPS in clause 5 of AWS D1.1/D1.1M.	The existing steel should be welded in accordance with the requirements of a steel with equivalent mechanical and compositional requirements that is permitted to be used with prequalified WPS in clause 5 of AWS D1.1/D1.1M.
	The existing steel standard specification requirements do not meet all the mechanical and compositional limits of a steel that is permitted to be used with prequalified WPS in clause 5 of AWS D1.1/D1.1M. The specified limits on phosphorus (P) and sulfur (S) are under 0.050%.	Preheat should be determined in accordance with AWS D1.1/D1.1M Annex B, based on the maximum compositional limits stated in the standard specification for the existing steel. WPS should be qualified by testing, using a sample of the existing steel to be welded that is extracted from the existing structure.
	The existing steel standard specification requirements do not meet all the mechanical and compositional limits of a steel permitted to be used with prequalified WPS in clause 5 of AWS D1.1/D1.1M. The standard specification permits levels of P or S or both to exceed 0.050%. This includes instances where the standard specification for the existing steel does not provide limits on P or S or both. In the case of unregulated levels of P or S or both, the composition of the existing steel to be welded should be determined by testing of samples of the existing steel to be welded that are extracted from the structure.	Preheat should be in accordance with AWS D1.1/D1.1M Annex B, based on the maximum compositional limits stated in the standard specification for the existing steel, or as determined from compositional testing. WPS should be qualified by test, using a sample of the existing steel to be welded that is extracted from the existing structure.

Table C-B3.3b
Guidance for Determining Welding Requirements for New Welds to Existing Steel That Was Not Previously Welded (Produced Under an Unknown Specification)

Existing Steel Classification	Considerations/Investigations	Welding Suggestions
The standard specification used to produce the existing steel is unknown. The existing steel was not previously welded.	The composition of the existing steel to be welded should be determined by testing. The mechanical properties of the existing steel should be determined by testing.	If the composition and mechanical properties are consistent with a steel permitted to be used with prequalified WPS in clause 5 of AWS D1.1/D1.1M, the steel may be welded in accordance with the prequalified WPS of a steel having equivalent mechanical and compositional requirements that is also permitted to be used with prequalified welds in clause 5 of AWS D1.1/D1.1M.
		If the composition and mechanical properties are not consistent with a steel that is permitted to be used with prequalified WPS in clause 5 of AWS D1.1/D1.1M, preheat should be in accordance with AWS D1.1/D1.1M Annex B, based on the results of compositional testing. WPS should be qualified by test, using a sample of the existing steel to be welded that is extracted from the existing structure.

The shipbuilding efforts of World War II brought about an increase in the popularity of welding as a joining method. While the governing steel specifications did not ensure good weldability, steel producers adjusted the compositional limits to meet the demands of shipbuilding that had transitioned from riveted construction to welded construction. Structural steels were not commercially produced under standard specifications written specifically for manufacture of easily weldable structural steels until after circa 1950. A general rule of thumb resulted from these trends: structural steels produced after World War II generally have good weldability. For steels manufactured before this time, weldability is variable; in some cases, the steel has good weldability, but in other cases, weldability is poor.

Additionally, steels manufactured before 1950 may exhibit physical features that adversely affect the soundness of the steel, such as nonmetallic inclusions, stringers, voids of various shapes, tears, and segregation. Steels exhibiting these soundness concerns are commonly referred to as "dirty" steels, particularly when compared to the soundness provided by modern steels manufactured with an intent to be welded. Certain manufacturing processes that were conducive to the production of steel with soundness concerns, such as the acid-Bessemer process, were permitted by standard specifications well into the 1960s; these types of processes would not ordinarily be used for manufacturing of steel with an intent for welding. Rimmed and capped steel may also have soundness concerns. In particular, structural steels manufactured prior to circa 1930 have an even greater likelihood of exhibiting soundness concerns because control of either chemical composition or manufacturing process with strict regard for weldability was not a commercial concern during that era; instead, riveted structural connections were used. Metallographic examination, as described in the paragraphs that follow and in Table C-B3.1, may be used to assess the soundness of existing structural steel to be welded.

A representative photomicrograph showing unsound features identified as elongated inclusions, or “stringers,” in a sample of structural steel obtained from a building constructed circa 1905 is given in Figure C-B3.1. Five different samples of structural steel from beam webs, beam flanges, angles, and column splice plates were obtained from this building and examined metallographically, and inclusions similar to that shown in Figure C-B3.1 were observed in all five samples. Inclusions of this nature need to be considered when the retrofit measures involve welding, because their presence can readily lead to lamellar tearing where the steel is stressed in the through-thickness direction, whether due to weld shrinkage restraint or due to applied loads. Weld shrinkage may cause the inclusions to join together and locally tear; this concern is most probable when the weld axis is parallel to the direction of rolling. Welded joints that are perpendicular to the direction of rolling may fail in tearing under applied loads.

Inclusions of the nature shown in Figure C-B3.1, along with other soundness concerns that may lead to poor weld performance, are detected visually through the use of metallographic examination, not by compositional analysis. Guidance for use of metallographic examination is provided in Table C-B3.1.

If significant inclusions or other significant soundness concerns are present in an existing structural steel component, connections may be accomplished by mechanical means such as bolting instead of by welding. Alternately, the suitability for welding could be established by testing on the actual steel to be welded. In this case, attention should be given to weld joint design and control of weld shrinkage strains.

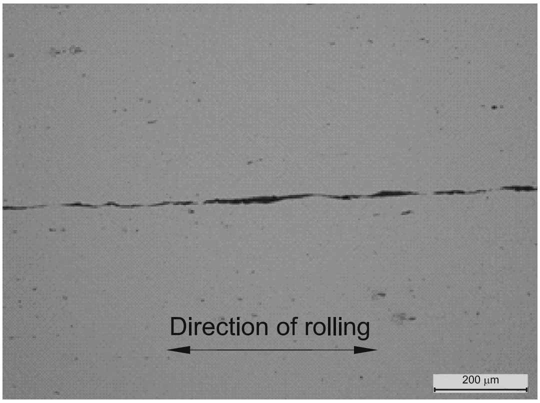


Fig. C-B3.1. Representative photomicrograph showing elongated inclusions (“stringers”) in structural steel that was manufactured circa 1905.

The historical structural wrought iron and historical gray cast iron that fall within the scope of these Provisions are considered to be not suitable for welded, load-bearing structural construction due to the metallurgical nature and compositional characteristics of these particular historical structural metals. The slag that is inherently present in historical structural wrought iron creates weakness in the through-thickness direction of the iron, and as a result, lamellar tearing is likely to develop if the wrought iron is welded (AWS, 2010). The coarse-grained metallurgy and compositional characteristics of historical gray cast iron are not conducive to fusion welding, although brazing is possible. However, a welded joint in cast iron is not as strong nor as ductile as the original cast iron itself (AWS, 1985). Structural welding to historical wrought iron and historical cast iron should be avoided (Miller, 2017), although welding might be used for nonstructural architectural and aesthetic purposes where stresses in the weld due to applied forces and weld shrinkage are very low. Structural connections to historical wrought iron and gray cast iron should instead be accomplished by mechanical means such as bolting. The guidance provided in Tables C-B3.1, C-B3.2, C-B3.3a, and C-B3.3b pertains to structural steel and is not intended to be directly applicable to historical structural wrought iron or historical gray cast iron.

CHAPTER C

COMPONENT PROPERTIES AND REQUIREMENTS

Prior to ASCE/SEI 41-13 (ASCE, 2013), permissible parameters for the nonlinear analysis procedures were given for both primary and secondary components. ASCE/SEI 41-13 removed the values for primary components and retained those for secondary components, which would be applicable to both component designations.

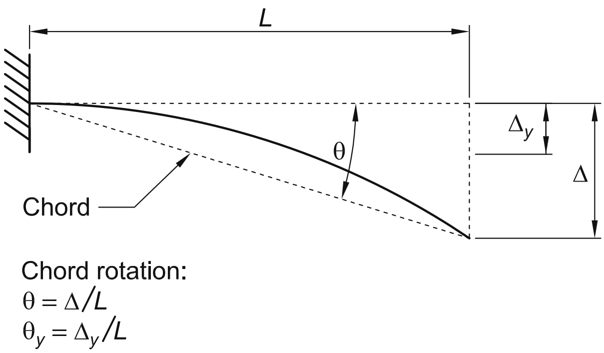
C1. GENERAL

The Commentary discussion for components and the Commentary for the various types of structural steel frame systems are inextricably linked together. Table C-C1.1 provides a cross-reference between general component types and frame-related Commentary sections where component-related discussion may be found.

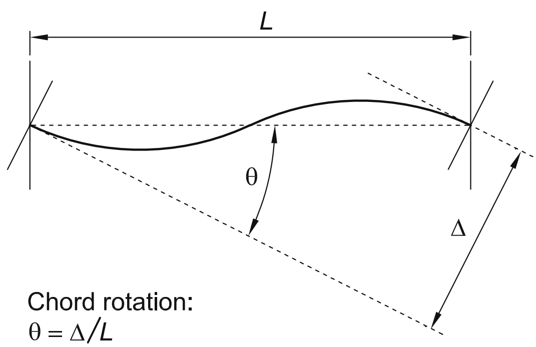
The parameters Q and Q_y in Figure C1.1 are the generalized component force and component yield strength, respectively. θ and Δ are the generalized parameters for component deformation. For beams and columns, θ is the total chord rotation (including elastic and plastic rotation), θ_y is the yield chord rotation corresponding to the flexural yield strength of the cross section, M_{pe}, Δ is total displacement (including elastic and plastic displacement), and Δ_y is yield displacement. The beam or column member may have other failure modes that govern the behavior of the member, e.g., lateral-torsional buckling resulting from lack of compression flange bracing. For panel zones, the generalized deformation component is γ, the angular shear deformation in radians, and γ_y is the angular shear yield deformation in radians. Figure C-C1.1 defines chord rotation for a cantilever and frame beam having length of beam L. The chord rotation is determined either by adding the yield chord rotation, θ_y, to the plastic rotation, or it can be taken as equal to the story drift. The yield rotation due to flexure is determined from Equations C2-2 and C3-15, where the point of contraflexure is anticipated to occur at the midlength of the beam or column.

TABLE C-C1.1
Commentary Cross-References for Components

Chapter C Component-Related Section	Related Commentary
C2. Beams	Commentary Sections D1 through D5
C3. Members Subjected to Axial or Combined Loading	Commentary Sections E1, E2, and E3
C4. Panel Zones	Commentary Sections D1 through D5
C5. Beam and Column Connections	Commentary Sections D1 through D5
C6. Steel Plates Used as Shear Walls	Commentary Sections E4



(a) Cantilever example



(b) Frame example

Fig. C-C1.1. Definition of chord rotation.

C2. BEAMS

1. General

Section C2 covers beams that, in many cases, resist an interaction of the flexural demand and shear demand. These Provisions include a strength ratio check, M_{CE}/V_{CE}, within a segment length of beam L_v, to determine the governing behavior for the segment, where M_{CE} is the expected flexural strength, V_{CE} is the expected shear strength, and L_v is the clear length between supports that resist translation in the direction of the shear force, as shown for example in Figure C-C2.1. In the traditional case of a shear-dominated link in an eccentrically braced frame (EBF), L_v is identical to the length of EBF link, e, as defined in the *Seismic Provisions* (AISC, 2022a). In the case of a shear-dominated beam in a moment frame, L_v is taken as the clear length between column faces.

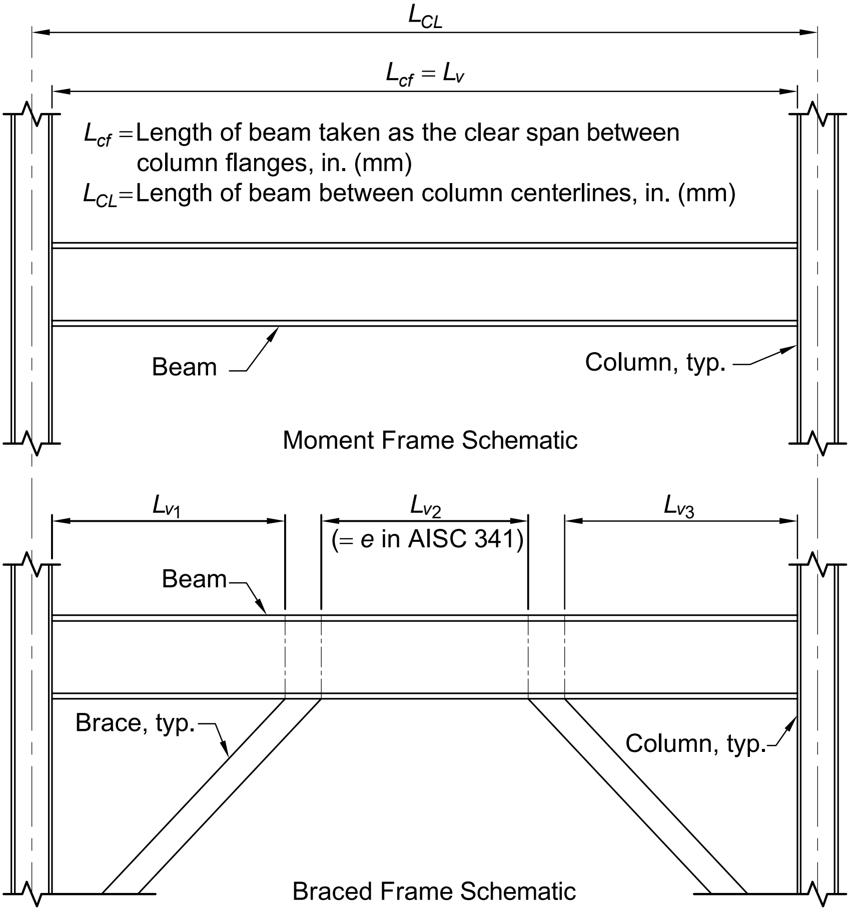


Fig. C-C2.1. Examples of the length L_v.

C3. MEMBERS SUBJECTED TO AXIAL OR COMBINED LOADING

1. General

This section addresses both diagonal braces (buckling-permitted and buckling-restrained) and vertical, or near vertical, columns, as well as horizontal beams with axial load ratio demands greater than 0.1 (referred to generically in these Provisions as columns). While it is true that braces and columns carry axial force, other aspects of their behavior are quite different. During an earthquake, buckling braces sustain significant inelastic axial deformation due to cyclic buckling, tensile yield, and inelastic post-buckling deformations to dissipate earthquake energy input into the structural system. Columns on the other hand are expected to sustain, at most, limited secondary axial yielding. In reality, a column or brace (or beam) can be oriented in any direction, and it is the action on the member and its desired behavior in response to that action that designates it as a brace or column as follows:

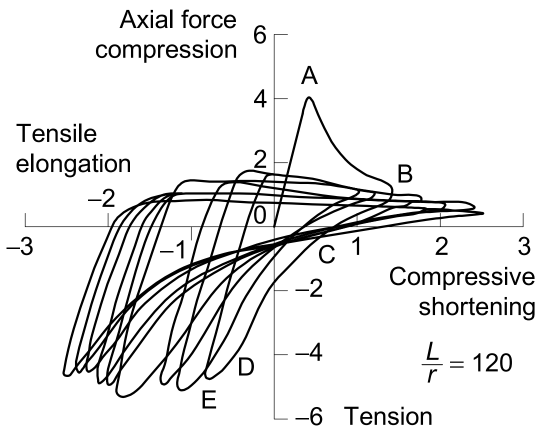
(1) Buckling brace: A member oriented in any direction that carries significant axial force with negligible flexure. A buckling brace sustains repeated buckling in compression and yielding in tension to dissipate earthquake energy.

(2) Buckling-restrained brace: A member oriented in any direction that carries significant axial force with negligible flexure. A buckling-restrained brace is restrained from buckling in compression and can sustain repeated yielding in compression and tension to dissipate earthquake energy.

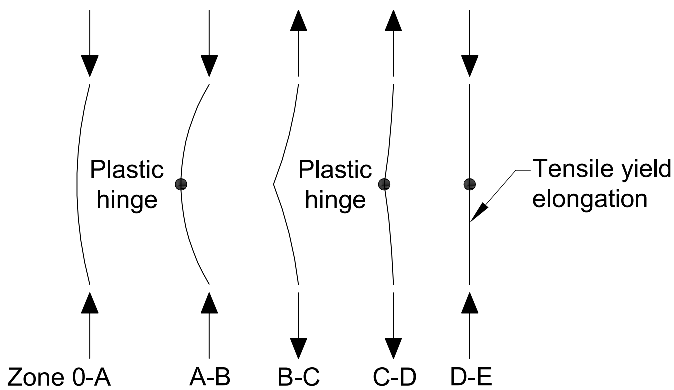
(3) Column: A member oriented in any direction that carries significant axial force with the possibility of significant flexure. A column is not permitted to buckle in compression, but is permitted to yield in tension, and can develop, when and where permitted, a plastic flexural hinge to dissipate earthquake energy. A column that resists significant demand from flexure concurrent with axial demand is commonly referred to as a beam-column but is generically referred to as a column in these Provisions.

These Provisions have been reorganized to reflect the differences in behavior between primary yield mechanisms sustained by the brace and secondary yield mechanisms sustained by the columns in braced frames. The provisions for braces have changed quite significantly in recognition of the hundreds of brace and braced frame tests for seismic behavior that have been completed during the past 20 years.

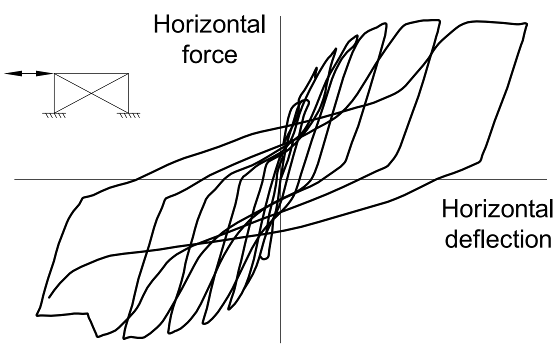
Cyclic inelastic behavior of buckling braces and braced frames is a pinched hysteretic behavior that is significantly different in tension and compression with deterioration in compressive resistance, as illustrated in Figure C-C3.1. The deterioration and resistance depend on many factors, including the brace and its connections, and as a result, a force-deformation envelope is introduced for brace members in Figure C3.1. This relationship is primarily dependent on the cross-sectional compactness ratio, where the denominator of the ratio is the seismic compactness limit provided by the *Seismic Provisions* (AISC, 2022a).



(a) Brace axial force-deflection hysteresis



(b) Zones of behavior



(c) Resulting frame lateral load-deflection hysteresis

Fig. C-C3.1. Response of concentrically braced frame (CBF) with buckling braces (Popov et al., 1976).

2. Stiffness

The stiffness of columns and buckling-restrained braces is largely unchanged from prior editions of ASCE/SEI 41 (ASCE, 2017). However, the stiffness of buckling braces varies widely during an earthquake due to the large inelastic deformations. A major deficiency in existing braced frames is braces that are much more cross-sectionally slender than current limits for highly ductile members in the *Seismic Provisions* (AISC, 2022a). Braces with this deficiency can be economically and efficiently retrofitted to provide performance comparable to a brace meeting the highly ductile slenderness limits by filling the brace with concrete so the concrete fills the tube but does not engage or contact the end connection of the brace (Sen et al., 2017). In this way, the stiffness, buckling resistance, and tensile resistance are largely unchanged by the concrete fill, but the inelastic performance of the brace is dramatically improved. Concrete fill that contacts and engages the end connection of the brace increases the stiffness and compressive resistance of the brace (Liu and Goel, 1988), and thus places greater demands on the frame and the connection and may adversely affect the inelastic performance of the concentrically braced frame (CBF) system.

3. Strength

The axial, flexural, and shear strengths of a brace or column are determined as the nominal strength determined for the controlling limit state as provided in the *Specification*. These Provisions require that the axial, flexure, or shear action be classified as either deformation-controlled or force-controlled. If a deformation-controlled column or brace is expected to dissipate earthquake energy in the structural system, it is required that the controlling limit state for that action be such that the member can sustain inelastic actions.

3a. Deformation-Controlled Actions

1. Expected Axial Strength

For the evaluation of buckling brace strength, the effective length, L_c, of the brace about both major axes should be considered.

For braces buckling out of the plane of the frame, the effective length factor, K, may be taken as follows:

(a) For braces with rotation-restrained end connections, as defined in Section C7, $K = 0.65$.

(b) For rotation-accommodating end connections, as defined in Section C7, $K = 1.0$.

(c) For intersecting braces in X-braced frames, $K = 0.7$.

Other values of K may be justified by analysis.

4. Permissible Performance Parameters

4a. Deformation-Controlled Actions

1. Axial Actions

This section contains significant updates, including the following:

(a) The tabulated values of the component capacity modification factors, *m*, modeling parameters, and deformation limits for columns and buckling-restrained braces are largely unchanged from prior editions of ASCE/SEI 41.

(b) Tables C3.2 and C3.4 contain values of *m*, modeling parameters, and deformation limits for buckling braces. These values have changed significantly. Prior values for buckling braces were essentially unchanged since the 1990s, and prior values had some significant inconsistencies due to the limited research on braced frames and the small-scale experimental results from that earlier era. The data in these tables on buckling braces benefit from the hundreds of brace and braced frame tests completed in the past two decades. The limits used to define brace performance consider the full range of parameters needed to predict brace behavior. While there have been extensive tests on braces and braced frames in recent years, primarily wide-flange and HSS braces, other brace types have little additional data and limited changes have been made. In some cases, these changes result in increased capacity for braces that had previously been considered deficient. These changes have reduced the number of footnotes and special conditions previously noted with these tables, but some footnotes were retained because there was no rational evidence to change them.

(c) Detailed attention has been given to rectangular HSS braces here to incorporate data collected from 69 different experiments (Liu and Goel, 1988; Lee, 1988; Shaback and Brown, 2003; Tremblay et al., 2003; Yang and Mahin, 2005; Han et al., 2007; Uriz and Mahin, 2008; Fell et al., 2009; Richard, 2009; Roeder et al., 2011a; Sen et al., 2016, 2017; Ibarra, 2018). For these experiments, the expressions in Tables C3.2 and C3.4 were derived from the total axial deformation range prior to tearing or fracture of the brace. The axial deformation capacity in tension and compression was assumed to be half the total axial deformation range (i.e., assuming a symmetric cyclic deformation history); this approach is recognized as simplistic but is intended to be suitable for use in commercial software. These data were fit to expressions using linear regression in logarithmic space.

(d) Less detailed attention has been given to other brace types. For these cross sections, expressions were fit as previously described but using the existing modeling parameters in ASCE/SEI 41. Therefore, the values of *m*, modeling parameters, and deformation limits for these cross sections are largely unchanged but presented as expressions to facilitate their use.

It should be emphasized the braced frames meeting the requirements of *Seismic Provisions* Section F2 are controlled by this section combined with requirements of Section E1. All other CBF are controlled by the lesser capacity provided by Sections C3 and C7 combined with the requirements of Section E1.

(e) The expressions in Tables C3.2 and C3.4 were fitted for braces with width-to-thickness ratios for the element λ of 8 or larger, where λ is as defined in the *Seismic Provisions*, and are not recommended for use with lower values of λ.

2. Flexural Actions Concurrent with Axial Actions

a. Linear Analysis Procedures

This section for columns is different from other components in that typically these Provisions would provide the permissible performance parameters to be used to verify the acceptance criteria given in ASCE/SEI 41, Section 7.5. For a column that is subject to flexure demand concurrent with axial demand, this section provides both the permissible performance parameters and the acceptance criteria, and was developed in accordance with ASCE/SEI 41, Section 7.5.

Column assessment for flexure concurrent with axial force is a two-step process: (1) verify the section flexural strength and (2) verify the member flexural strength (instability).

1. Section Strength

The first step is to check the acceptability of the column on the basis of the plastic rotation within a flexural plastic hinge. This is done by checking the section flexural strength of the column. When in compression, the elements of the column cross section can buckle or yield, with yielding having two degrees of compactness: (1) moderately ductile and (2) highly ductile. This is similar in nature to classification as slender, noncompact, or compact in accordance with the *Specification*. The ductility capacity of the flexural plastic hinge, which is based on the extent of local buckling of the elements within the hinge, is determined by the degree of compactness.

Cross-sectional elements should be classified as compact as a minimum (matches that provided for moderately ductile) for compression to develop a fully yielded section (i.e., capable of sustaining some inelastic strains beyond yield before local buckling occurs). When the section flexural strength is governed by full yielding of the cross section, this strength curve is commonly referred to as the yield surface. A yield surface is the plastic capacity of a cross section and is provided by Equation C3-4. This surface does not capture the effects of global member buckling on the capacity of the plastic hinge. The ductility, or plastic capacity of a cross section that contains elements that are classified as noncompact or slender by the *Specification* is further reduced by local buckling. While local buckling conceptually limits section strength, local buckling is addressed in member flexural strength (discussed in Commentary Section C3.4a.2.a.2).

Two anchor points are used to develop the uniaxial *P-M* interaction curve representing the yield surface. The first is P_y when $M = 0$, and the second is M_p when $P = 0$, where

M = bending moment, kip-in. (N-mm)
M_p = plastic bending moment, kip-in. (N-mm)
= $F_y Z$
F_y = specified minimum yield stress, ksi (MPa)
Z = plastic section modulus taken about the axis of bending, in.^{3} (mm^3)
P = axial force (compression or tension), kips (N)
P_y = axial yield strength, kips (N)
= $F_y A_g$
A_g = gross area of cross section, in.^{2} (mm^2)

The curve connecting these anchor points depends on the geometry of the cross section and characterizes the plastic flexural strength of the cross section in the presence of axial force (compression or tension), M_{pc}.

For a wide-flange shape with bending about its major principal axis (referred to here as the *x*-axis), the bilinear idealization of the *P-M* interaction curve is given by Equations C-C3-1 and C-C3-2. By substituting in the appropriate levels of strength and demand, Equations C-C3-1 and C-C3-2 can be rearranged to give Equations C3-5 and C3-6.

(a) When $\dfrac{P}{P_y} < 0.15$

$$\frac{P}{2P_y} + \frac{M_{pc}}{M_p} = 1 \tag{C-C3-1}$$

(b) When $\dfrac{P}{P_y} \geq 0.15$

$$\frac{P}{P_y} + \frac{8}{9}\frac{M_{pc}}{M_p} = 1 \tag{C-C3-2}$$

This bilinear representation has been widely accepted and can be found in *Specification* Commentary Chapter H (Equations C-H1-2a and C-H1-2b). Alternatively, the *P-M* interaction curve can be taken from classic plastic analysis as determined by Equations C-C3-3 and C-C3-4.

(a) When $\dfrac{P}{P_y} < 0.2$

$$\frac{M_{pc}}{M_p} = 1 \tag{C-C3-3}$$

(b) When $\dfrac{P}{P_y} \geq 0.2$

$$\frac{P}{P_y} + 0.85\left(\frac{M_{pc}}{M_p}\right) = 1 \tag{C-C3-4}$$

Equations C-C3-3 and C-C3-4 can also be written as

$$M_{pc} = 1.18\left(1 - \frac{P}{P_y}\right)M_p \leq M_p \tag{C-C3-5}$$

Earlier editions of ASCE/SEI 41 used Equation C-C3-5. ASCE/SEI 41-17 (ASCE, 2017) was updated to use Equations C-C3-1 and C-C3-2 for consistency with the *Specification*. These equations are representative of wide-flange shapes with bending about the major axis.

Expanding this idealization to account for biaxial moments gives Equations C-C3-6 and C-C3-7.

(a) When $\frac{P}{P_y} < 0.2$

$$\frac{P}{2P_y} + \left(\frac{M_{pcx}}{M_{px}} + \frac{M_{pcy}}{M_{py}}\right) \leq 1 \tag{C-C3-6}$$

(b) When $\frac{P}{P_y} \geq 0.2$

$$\frac{P}{P_y} + \frac{8}{9}\left(\frac{M_{pcx}}{M_{px}} + \frac{M_{pcy}}{M_{py}}\right) \leq 1 \tag{C-C3-7}$$

where

M_{pcx} = plastic flexural strength of the cross section about the major principal axis (*x*-axis) in the presence of axial force (compression or tension), kip-in. (N-mm)

M_{pcy} = plastic flexural strength of the cross section about the minor principal axis (*y*-axis) in the presence of axial force (compression or tension), kip-in. (N-mm)

M_{px} = plastic flexural strength of the cross section about the *x*-axis in the absence of axial force, kip-in. (N-mm)

M_{py} = plastic flexural strength of the cross section about the *y*-axis in the absence of axial force, kip-in. (N-mm)

This biaxial form assumes that the shape of the idealized *P-M* interaction curve is the same for both axes and interaction of M_x-M_y between M_{pcx} and M_{pcy} at a given value for P is linear, as shown in Figure C-C3.2. These Provisions further limit $P/P_y \leq 0.75$ (see discussion in Commentary Section C3.4a.2.a.2), which is also shown in Figure C-C3.2.

It is conservative to apply Equations C-C3-1 and C-C3-2 to a wide-flange shape bent about its minor axis. The curve provided by Equations C-C3-6 and C-C3-7 was adopted during the development of load and resistance factor design (LRFD) to represent all sections, which is beyond the scope of this discussion. The equations for minor-axis bending of wide-flange shapes are recommended here to be given by Equations C-C3-8 and C-C3-9. By substituting in the appropriate levels of strength and demand, Equations C-C3-8 and C-C3-9 can be rearranged to give Equations C3-7 and C3-8.

(a) When $\dfrac{P}{P_y} < 0.4$

$$\frac{P}{4P_y} + \frac{M_{pc}}{M_p} \leq 1 \tag{C-C3-8}$$

(b) When $\dfrac{P}{P_y} \geq 0.4$

$$\frac{P}{P_y} + \frac{2}{3}\frac{M_{pc}}{M_p} \leq 1 \tag{C-C3-9}$$

Classic plastic analysis provides the following equations for minor-axis bending for a wide-flange shape.

(a) When $\dfrac{P}{P_y} < 0.4$

$$\frac{M_{pc}}{M_p} = 1 \tag{C-C3-10}$$

(b) When $\dfrac{P}{P_y} \geq 0.4$

$$\left(\frac{P}{P_y}\right)^2 + 0.84\left(\frac{M_{pc}}{M_p}\right) = 1 \tag{C-C3-11}$$

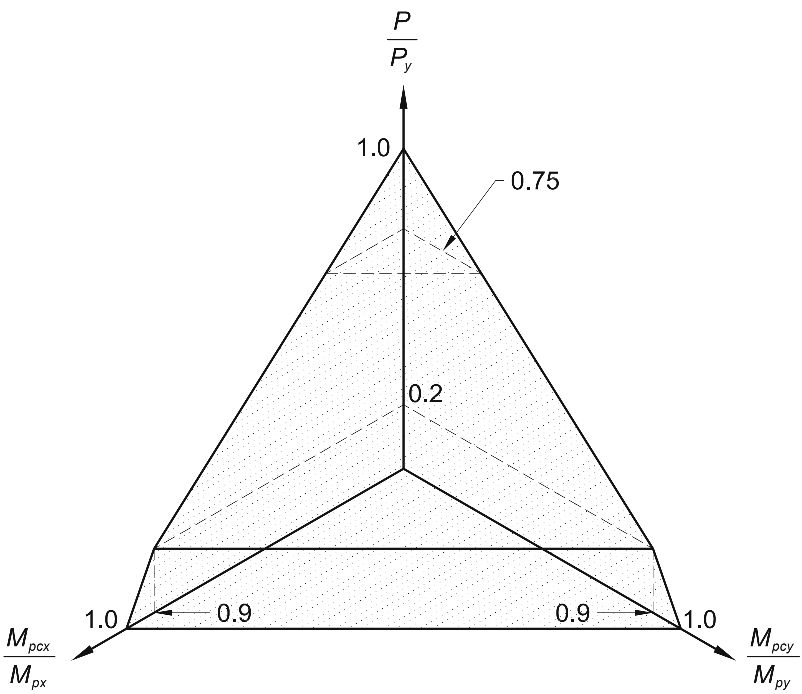


Fig. C-C3.2. Idealized bilinear yield surface for biaxial moments.

Equations C-C3-10 and C-C3-11 can also be written as

$$M_{pc} = 1.19\left[1-\left(\frac{P}{P_y}\right)^2\right]M_p \le M_p \tag{C-C3-12}$$

Figure C-C3.3 compares the curve given by Equations C-C3-8 and C-C3-9 to the one given by Equation C-C3-12 for minor-axis bending of a wide-flange shape.

The math becomes complicated when trying to develop equations similar to Equations C-C3-6 and C-C3-7 for biaxial bending that account for difference between the *P-M* curves for each axis. Figure C-C3.4 shows the interaction surface for biaxial bending that recognizes those differences.

To effectively capture the M_x-M_y interaction, a plane is cut through the yield surface at P and the M_x-M_y interaction is evaluated between the two anchor points, M_{pcx} and M_{pcy}, located where the cut plane intersects the *P-M* interaction curves for each axis, as shown in Figure C-C3.5.

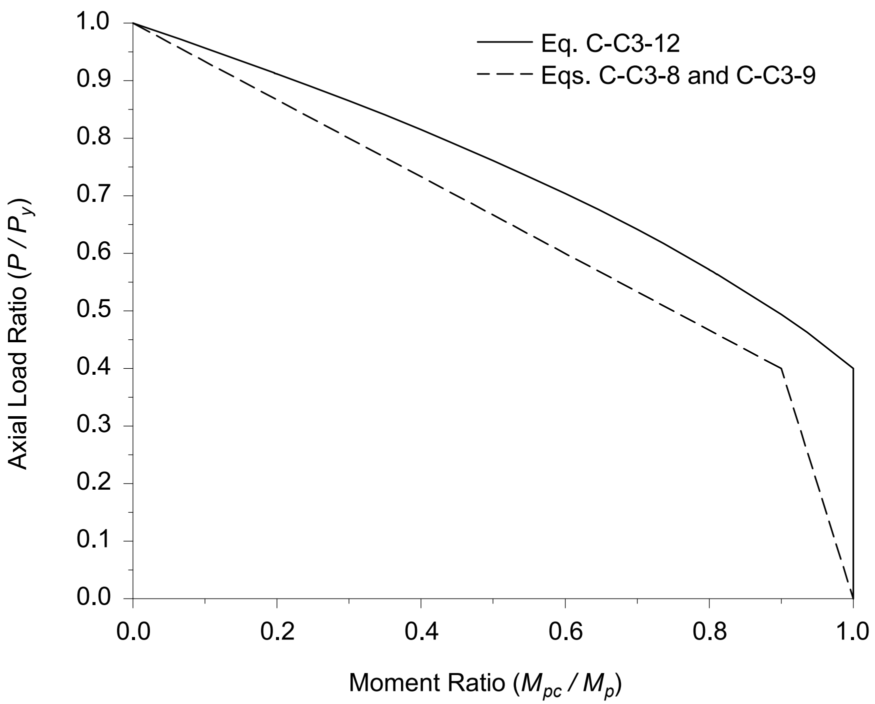


Fig. C-C3.3. Idealized bilinear yield surface for minor-axis bending of a wide-flange shape.

Cutting the plane gives

$$\left(\frac{M_x}{M_{pcx}}\right)^{\alpha} + \left(\frac{M_y}{M_{pcy}}\right)^{\beta} \leq 1 \tag{C-C3-13}$$

where

M_x = bending moment about the x-axis, kip-in. (N-mm)
M_y = bending moment about the y-axis, kip-in. (N-mm)
α = exponent for nonlinear yield surface
β = exponent for nonlinear yield surface

For a linear interaction between M_{pcx} and M_{pcy} at P, as shown in Figure C-C3.5, $\alpha = \beta = 1.0$. By substituting in the appropriate levels of strength and demand for a deformation-controlled action, and taking $\alpha = \beta = 1.0$, Equation C-C3-13 gives Equation C3-4.

It should be noted that the two interaction approaches described above, one written in the form of Equations C-C3-1 and C-C3-2 (or Equations C-C3-6 and C-C3-7) and one written in the form of Equation C-C3-13, do not produce the same result.

To illustrate this, consider the uniaxial case illustrated in Figure C-C3.6. Assume $P/P_y = 0.3$ (> 0.2) and $M/M_p = 0.35$.

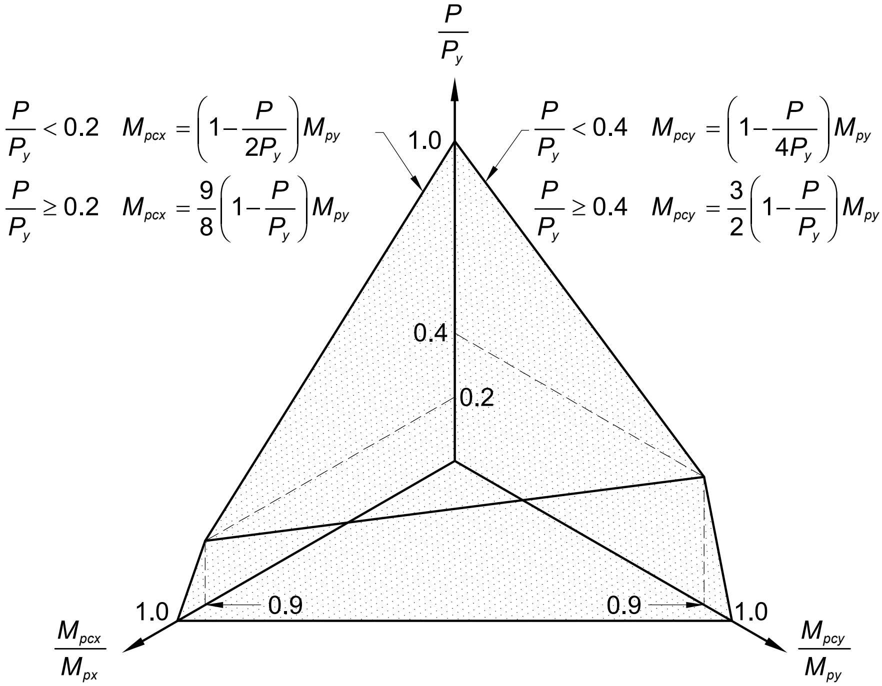


Fig. C-C3.4. Idealized bilinear yield surface for biaxial bending of wide-flange shapes.

Approach 1

$$\frac{P}{P_y} + \frac{8}{9}\frac{M}{M_p} \leq 1$$

$$0.3 + \frac{8}{9}(0.35) = 0.61 < 1$$

Approach 2

$$\frac{M}{M_{pc}} = \frac{M}{\frac{9}{8}\left(1 - \frac{P}{P_y}\right)M_p} \leq 1$$

$$\frac{0.35}{\frac{9}{8}(1 - 0.3)} = 0.44 < 1$$

The difference is that the interaction curve for Approach 1 is a measure of the reserve strength assuming that failure (hitting the yield surface) is by a proportional increase in both P and M. Whereas, Approach 2 is a measure of the reserve flexural strength at a constant level of P.

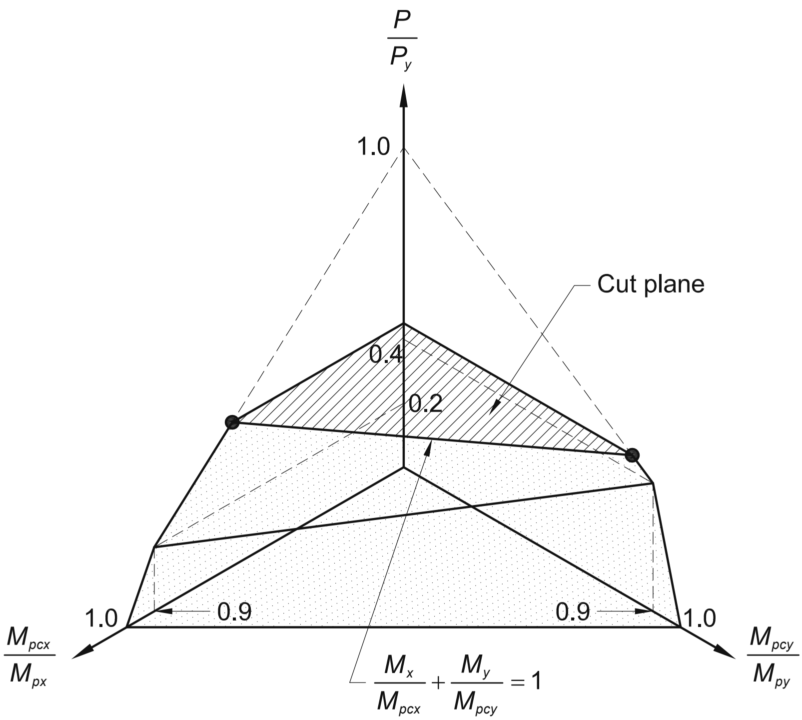


Fig. C-C3.5. Idealized bilinear yield surface for biaxial bending with plane cut that intersects the interaction curve on each axis.

Approach 1

$$\frac{0.461}{0.754} = \frac{0.35}{\underbrace{0.573}_{\text{moment}}} = \frac{0.3}{\underbrace{0.491}_{\text{axial}}} = 0.61 \text{ (increase in } P \text{ is } 1.64P)$$

Approach 2

$$\frac{0.35}{0.788} = 0.44$$

Using 1.64P in Approach 2,

$$\frac{M}{M_{pc}} = \frac{M}{\frac{9}{8}\left(1 - \frac{P}{P_y}\right)M_p} \le 1$$

$$\frac{0.35}{\frac{9}{8}\left[1-(1.64)(0.3)\right]} = 0.61 < 1\text{: a +37\% change.}$$

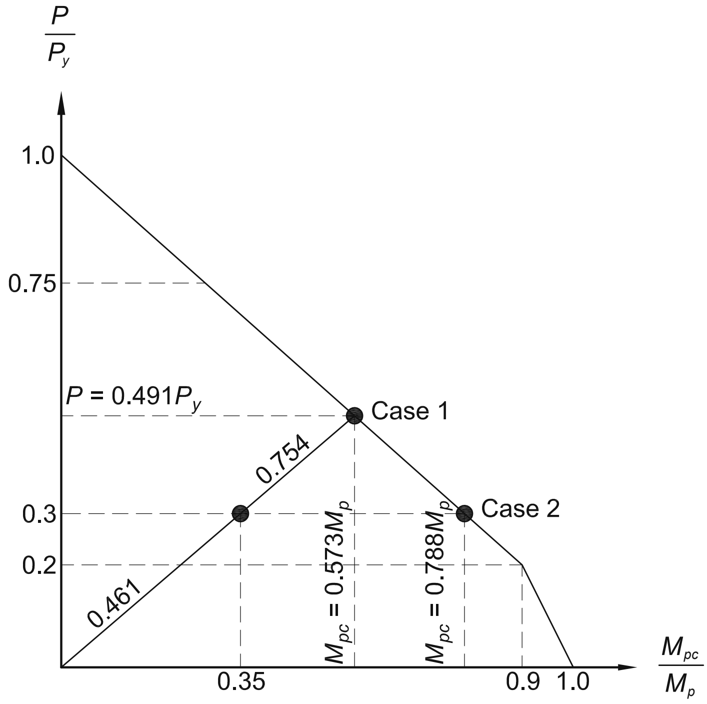


Fig. C-C3.6. Interaction curve showing Cases 1 and 2.

The approach corresponding to Equation C-C3-13, given in Equation C3-4, was adopted for two reasons:

(1) m and the plastic rotation capacity are a function of P. To maintain consistency, the P used to compute these is the same P used to check interaction. Otherwise, assessment would become an iterative process where the solution is updated each time it hits the yield surface and the new value for P is used in the next increment, eventually converging on the solution.

(2) In a seismic assessment for a column with plastic hinges, the axial load changes will generally be capped by hinging in the beams. The increase in axial force will not be proportional to increase in flexural straining (using m or a nonlinear hinge).

What is described in the preceding also holds for the biaxial case. Assume $P/P_y = 0.3$ (> 0.2) and $M_x/M_{px} = M_y/M_{py} = 0.35$.

Approach 1

$$\frac{P}{P_y}+\frac{8}{9}\left(\frac{M_x}{M_{px}}+\frac{M_y}{M_{py}}\right)\le 1$$

$$0.3+\frac{8}{9}\left(0.35+0.35\right)=0.92<1$$

Approach 2

$$\frac{M_x}{M_{pcx}}+\frac{M_y}{M_{pcy}}=\frac{M_x}{\frac{9}{8}\left(1-\frac{P}{P_y}\right)M_{px}}+\frac{M_y}{\frac{9}{8}\left(1-\frac{P}{P_y}\right)M_{py}}\le 1$$

$$\frac{0.35}{0.788}+\frac{0.35}{0.788}=0.88<1$$

or using Equation C-C3-8 for a wide-flange shape,

$$\frac{M_x}{M_{pcx}}+\frac{M_y}{M_{pcy}}=\frac{M_x}{\frac{9}{8}\left(1-\frac{P}{P_y}\right)M_{px}}+\frac{M_y}{\left(1-\frac{P}{4P_y}\right)M_{py}}\le 1$$

$$\frac{0.35}{0.788}+\frac{0.35}{0.925}=0.82<1$$

A column can be assessed for only in-plane flexural demands when the ratio of out-of-plane flexural demand to out-of-plane plastic flexural strength is less than 0.15. This is done to simplify the assessment of a column when out-of-plane demands do not significantly impact in-plane effects by more than 10%.

2. Member Strength

The second step checks the acceptance of the column based on its member strength (instability), which allows some flexural yielding depending on whether the column is in an inelastic buckling mode.

These Provisions take a similar approach for member strength as that described above for section strength. ASCE/SEI 41-17 (ASCE, 2017) used Equations C-C3-14 and C-C3-15 to assess the member strength of a column.

(a) When $\dfrac{|P_{UF}|}{P_{CL}} < 0.2\kappa$

$$\frac{|P_{UF}|}{2P_{CL}} + \left(\frac{M_{Ux}}{m_x M_{CxLTB}} + \frac{M_{Uy}}{m_y M_{Cy}}\right) \le \kappa \tag{C-C3-14}$$

(b) When $\dfrac{|P_{UF}|}{P_{CL}} \ge 0.2\kappa$

$$\frac{|P_{UF}|}{P_{CL}} + \frac{8}{9}\left(\frac{M_{Ux}}{m_x M_{CxLTB}} + \frac{M_{Uy}}{m_y M_{Cy}}\right) \le \kappa \tag{C-C3-15}$$

where

M_{CxLTB} = lateral-torsional buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm). If flexure is deformation-controlled, M_{CxLTB} = M_{CExLTB}; otherwise, flexure is force-controlled and M_{CxLTB} = M_{CLxLTB}.

M_{Cy} = flexural strength of the member about the minor principal axis (y-axis) at P_{UF}, kip-in. (N-mm). If flexure is deformation-controlled, $M_{Cy} = M_{CEy}$; otherwise flexure is force-controlled and $M_{Cy} = M_{CLy}$.

M_{CExLTB} = expected lateral-torsional buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)

M_{CEy} = expected flexural strength of the member about the y-axis, kip-in. (N-mm)

M_{CLxLTB} = lower-bound lateral-torsional buckling flexural strength of the member about the x-axis in the absence of axial force, kip-in. (N-mm)

M_{CLy} = lower-bound flexural strength of the member about the y-axis, kip-in. (N-mm)

M_{Ux} = bending moment about the x-axis, kip-in. (N-mm)

M_{Uy} = bending moment about the y-axis, kip-in. (N-mm)

P_{CL} = lower-bound compressive strength, kips (N)

P_{UF} = axial force (compression or tension) determined as a force-controlled action in accordance with ASCE/SEI 41, Section 7.5, kips (N)

m_x = component capacity modification factor, m, for column flexure about the x-axis at P_{UF} in accordance with Table C3.5

m_y = component capacity modification factor, m, for column flexure about the y-axis at P_{UF} in accordance with Table C3.5

κ = knowledge factor

For the following discussion, consider the two principal axes separately and take the values for m and κ as unity for simplicity.

For the major axis (x-axis), Equations C-C3-14 and C-C3-15 reduce to

(a) When $\dfrac{|P_{UF}|}{P_{CL}} < 0.2$

$$\frac{|P_{UF}|}{2P_{CL}} + \left(\frac{M_{Ux}}{M_{CxLTB}}\right) \leq 1 \tag{C-C3-16}$$

(b) When $\dfrac{|P_{UF}|}{P_{CL}} \geq 0.2$

$$\frac{|P_{UF}|}{P_{CL}} + \frac{8}{9}\left(\frac{M_{Ux}}{M_{CxLTB}}\right) \leq 1 \tag{C-C3-17}$$

Note that in the requirements of these Provisions, the additional subscripts E and L denote expected or lower-bound strength, respectively, and D and F denote deformation- or force-controlled action, respectively. Equations C-C3-16 and C-C3-17 can be rearranged to give Equations C3-12 and C3-13.

Equations C-C3-16 and C-C3-17 are specific to flexural strength for out-of-plane buckling and lateral-torsional buckling (LTB) and intend to follow a method similar to that of *Specification* Section H1.3(b). As identified in ASCE/SEI 41 and *Specification* Section H1.3, when computing the available LTB strength (i.e, M_{C_LTB}) with $C_b > 1.0$, where C_b is the lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the segment are braced, the user is not limited to M_p as written in *Specification* Chapter F.

To understand what occurs when C_b is neglected, consider a frame column with $P_{CLy} = 0.6P_{ye}$ and $M_{CxLTB} = 0.8M_{pe}$,

where

M_{pe} = expected plastic flexural strength, kip-in. (N-mm)

P_{CLy} = lower-bound compressive strength out of the plane of bending, kips (N)

P_{ye} = expected axial yield strength, kips (N)

Adopting C_b = 1.0, Figure C-C3.7 indicates that member strength will always govern assessment in this case; m_x will be reduced for LTB (as a beam) in addition to P_{UF}. Note that the curve provided by *Specification* Equation H1-3 is also shown in Figure C-C3.7, with C_b = 1.0, and with ASCE/SEI 41-level demands and strengths, for comparison. (These provisions use Equations C3-12 and C3-13, which correspond to *Specification* Equation H1-1, rather than an equation in the form of *Specification* Equation H1-3, for simplicity.)

Alternatively, take C_b = 1.75 (thus $M_{CxLTB} = 1.4M_{pe}$ at $P_{UF} = 0$), but consider what happens when the user caps the anchor point M_{CxLTB} at M_{pe}, as one would do using *Specification* Equations F2-2 and F2-3, at $P_{UF} = 0$ (see Figure C-C3.8). Again, member strength will govern assessment, and m_x would not be reduced for LTB in this case, just P_{UF}. The issue is that *Specification* Equations F2-2 and F2-3 for LTB do not address axial force, nor does C_b, which both can be shown to be a function of the axial force. Thus, it is assumed that the adopted shape of the *P-M* interaction curve is representative of a column failing in LTB at various levels of P_{UF}.

However, this is not what happens, because tests have shown that columns bent in double curvature can develop plastic hinges at low axial loads without instability. Taking C_b = 1.75 ($M_{CxLTB} = 1.4M_{pe}$ at $P_{UF} = 0$) and not capping to M_{pe} provides Figure C-C3.9. At low levels

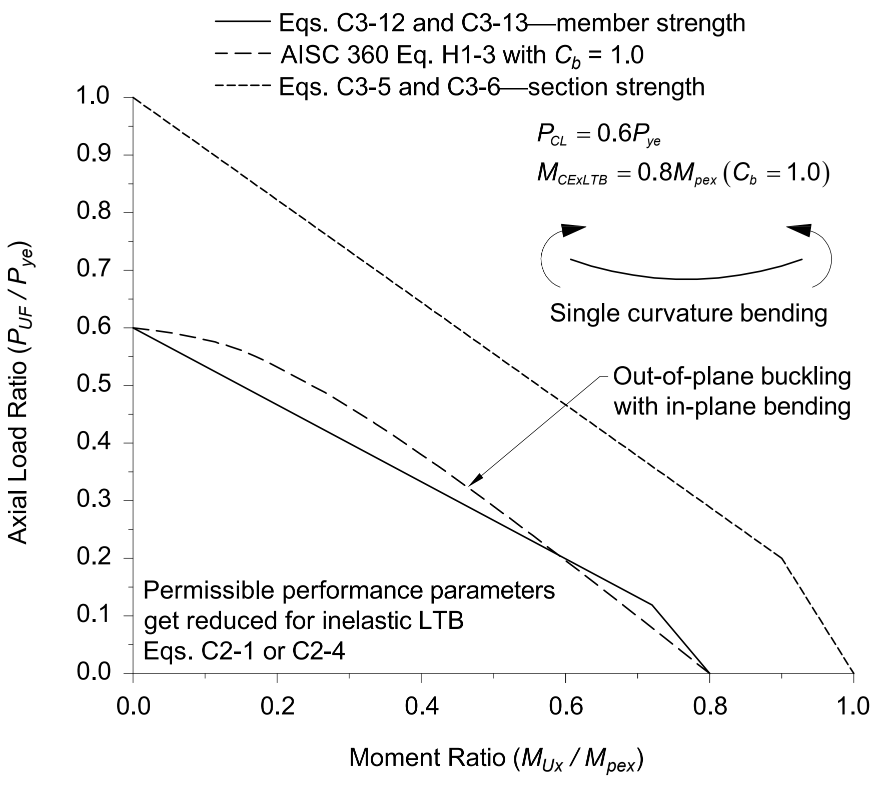


Fig. C-C3.7. Uniaxial P-M$_x$ *interaction curves with* $P_{CL} = 0.6P_{ye}$ *and* $M_{CxLTB} = 0.8M_{pe}$.

of P_{UF}, section strength governs the assessment with member strength governing at high levels of P_{UF}. Similarly, m_x would not be reduced for LTB. This approach best represents the strategy of these Provisions, which is to shift the anchor point when $P_{UF} = 0$ out beyond the plastic moment when $C_b > 1.0$ and let the yield surface govern when applicable.

If a member does not fail by LTB but fails by local buckling (LB), then replace M_{CxLTB} with M_{CxLB}, where M_{CxLB} is the local buckling flexural strength of the member about the x-axis in the absence of axial force. Like LTB, LB should be avoided if a column is required to maintain stability.

In most cases, out-of-plane buckling with LTB will govern the assessment of a column as shown in Figure C-C3.10, which shows the interaction curves for a case where $P_{CLy} = 0.8P_{ye}$, $P_{CLx} = 0.7P_{ye}$, and M_{CxLTB} is limited to be no greater than M_{pe}, where P_{CLx} is the lower-bound compressive strength in the plane of bending. To assess the potential for in-plane instability, with LTB restrained or unrestrained, these Provisions limit the axial load ratio to $P_{UF}/P_{CL} < 1.0$, which is deemed sufficient because there is a limited range at low axial load ratios that in-plane stability governs and the curve is reasonably close to the yield

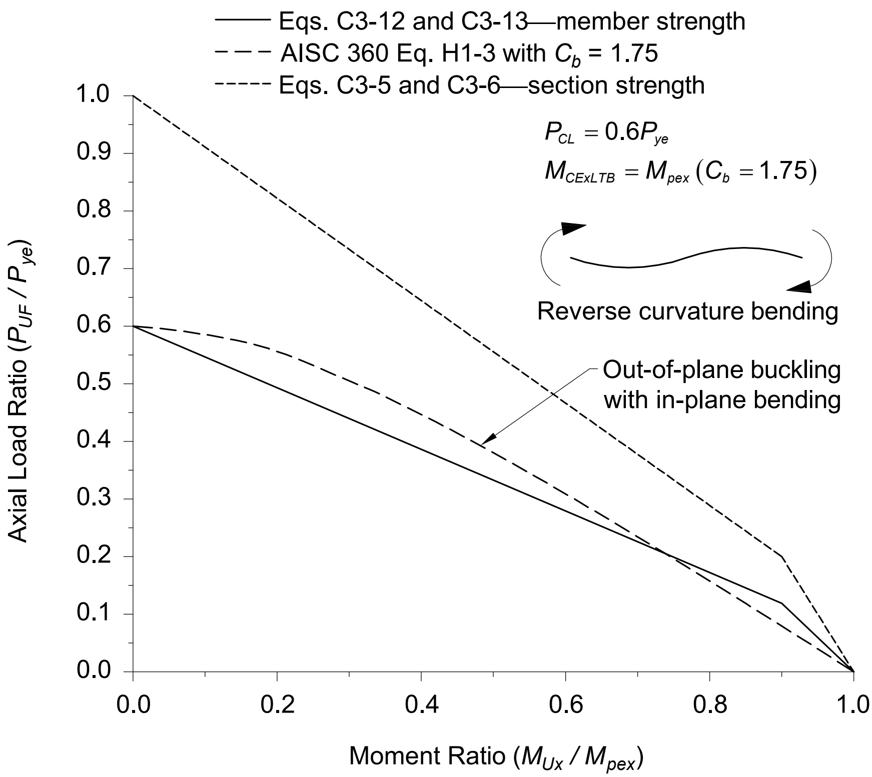


Fig. C-C3.8. Uniaxial P-M_x *interaction curves with* C_b *= 1.75, but limiting* M_{CxLTB} *to a maximum of* M_{pe}.

surface. Following these Provisions, the curve for in-plane stability would not be limited to the plastic moment, as is shown in Figure C-C3.11. In-plane instability will only govern when out-of-plane instability is prevented so that $P_{CLy} > P_{CLx}$.

For the minor-axis bending only, Equations C-C3-14 and C-C3-15 reduce to the following:

(a) When $\dfrac{|P_{UF}|}{P_{CL}} < 0.2$

$$\frac{|P_{UF}|}{2P_{CL}} + \left(\frac{M_{Uy}}{M_{Cy}}\right) \leq 1 \tag{C-C3-18}$$

(b) When $\dfrac{|P_{UF}|}{P_{CL}} \geq 0.2$

$$\frac{|P_{UF}|}{P_{CL}} + \frac{8}{9}\left(\frac{M_{Uy}}{M_{Cy}}\right) \leq 1 \tag{C-C3-19}$$

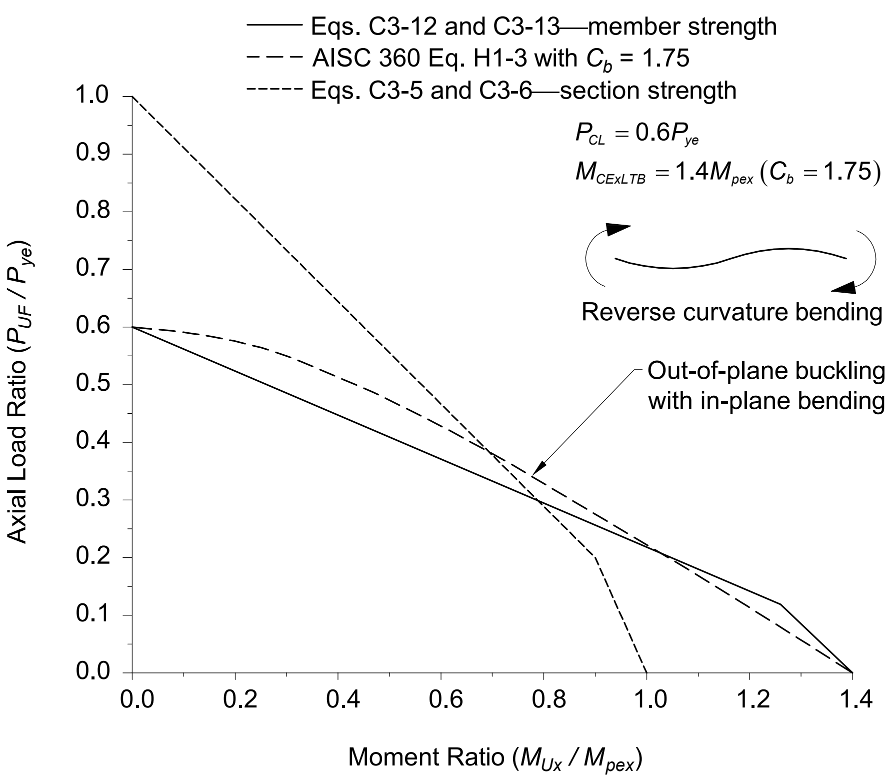


Fig. C-C3.9. Uniaxial P-M_x *interaction curves with* C_b *= 1.75 without limiting* M_{CxLTB} *to a maximum of* M_{pe}.

Lateral-torsional buckling is not applicable for minor-axis bending and the member strength, M_{Cy}, is the section strength M_{py} (assuming there is no local bucking). Consequently, when using Equations C-C3-18 and C-C3-19 for minor-axis bending, member strength will always govern assessment. This is even more true if the yield surface is already conservative for a given cross-section as shown in Figure C-C3.12, which compares the section strength interaction curves to the member strength interaction curve.

Figure C-C3.13 shows the biaxial interaction surface for member strength if the same interaction curve is used for both axes, and C_b is taken as 1.0. This approach would be overly conservative.

The yield surface for minor axis should be used to represent the P-M_y interaction and a secondary limit should be established such that $P_{UF}/P_{CLy} \le 1.0$, as shown in Figure C-C3.14.

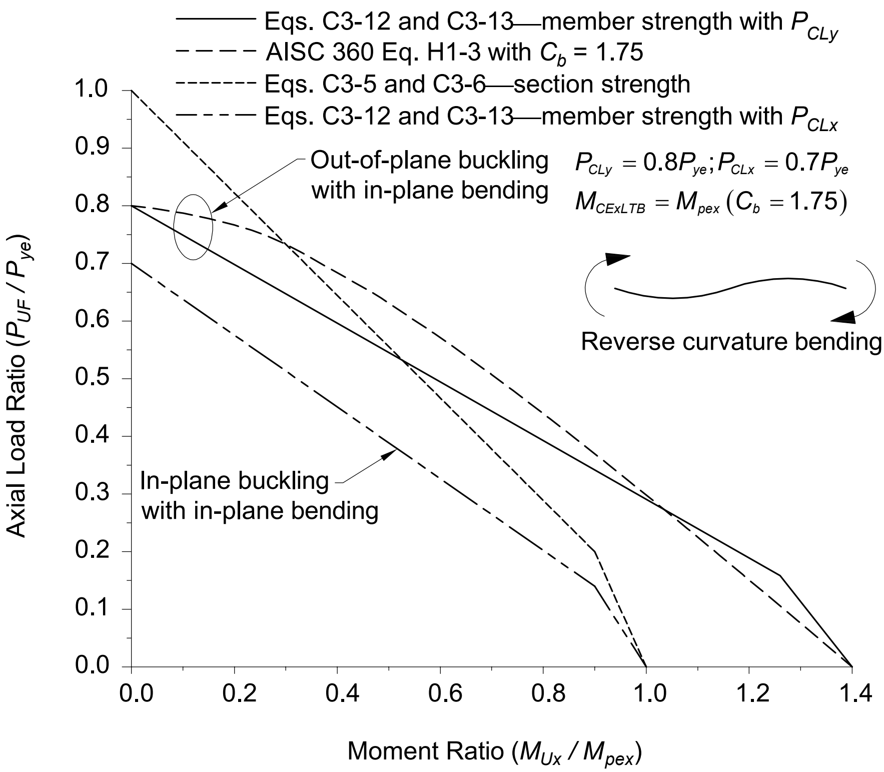


Fig. C-C3.10. Uniaxial P-M_x *interaction curves with* $P_{CLy} = 0.8P_{ye}$, $P_{CLx} = 0.7P_{ye}$, *and limiting* M_{CxLTB} *to be no greater than* M_{pe}.

M_{CxLTB} should also incorporate $C_b > 1.0$, when applicable, as shown in Figure C-C3.15.

To effectively capture the M_x-M_y interaction in this case, a plane is cut at P and the M_x-M_y interaction evaluated between the anchor points, M_{Cx} and M_{Cy}, created where the cut plane intersects the uniaxial P-M interaction curves for each axis, as shown in Figure C-C3.16. This gives

$$\left(\frac{M_{Ux}}{M_{Cx}}\right)^{\alpha} + \left(\frac{M_{Uy}}{M_{Cy}}\right)^{\beta} \le 1 \qquad \text{(C-C3-20)}$$

where

M_{Cx} = flexural strength of the member about the major principal axis (x-axis) at P_{UF}, kip-in. (N-mm). If flexure is deformation-controlled, $M_{Cx} = M_{CEx}$; otherwise, flexure is force-controlled and $M_{Cx} = M_{CLx}$.

M_{CEx} = expected flexural strength of the member about the x-axis, kip-in. (N-mm)

M_{CLx} = lower-bound flexural strength of the member about the x-axis, kip-in. (N-mm)

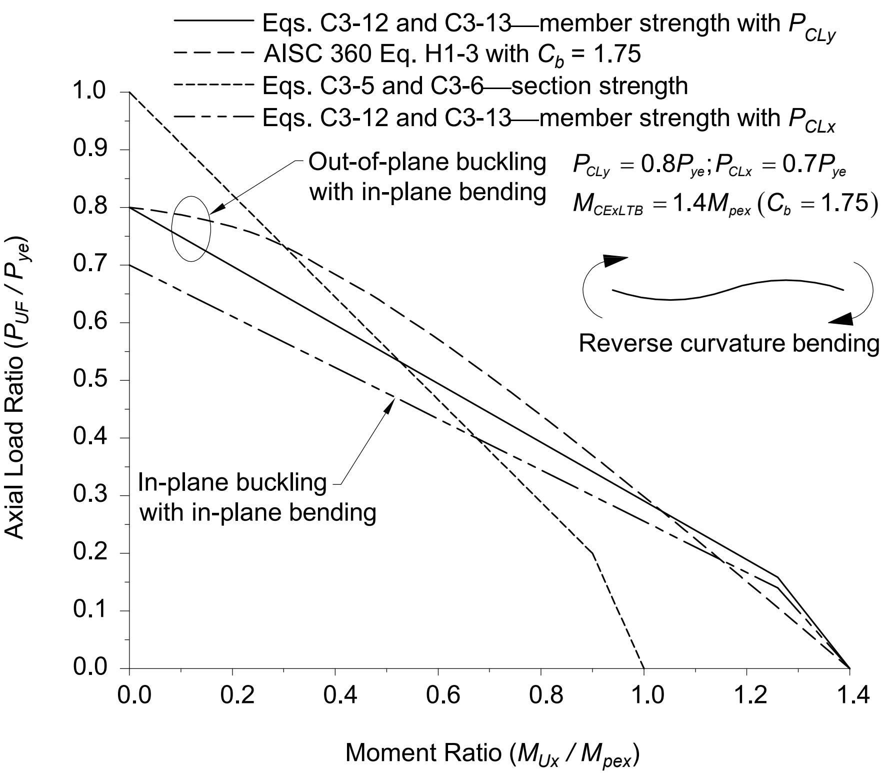


Fig. C-C3.11. Uniaxial P-M$_x$ *interaction curves with the in-plane instability curve not limited to the plastic moment.*

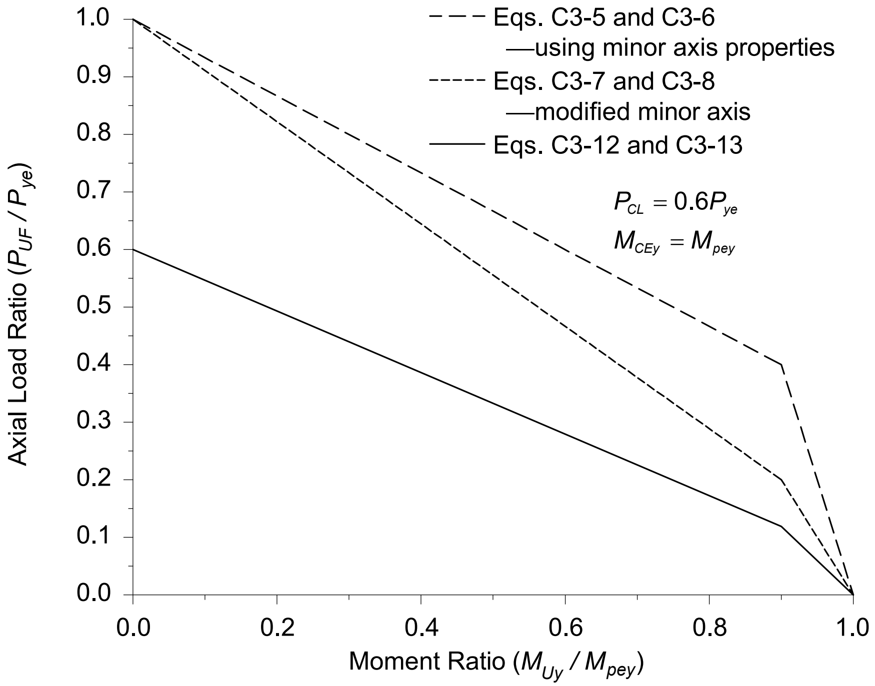


Fig. C-C3.12. Uniaxial P-M_y *interaction curves.*

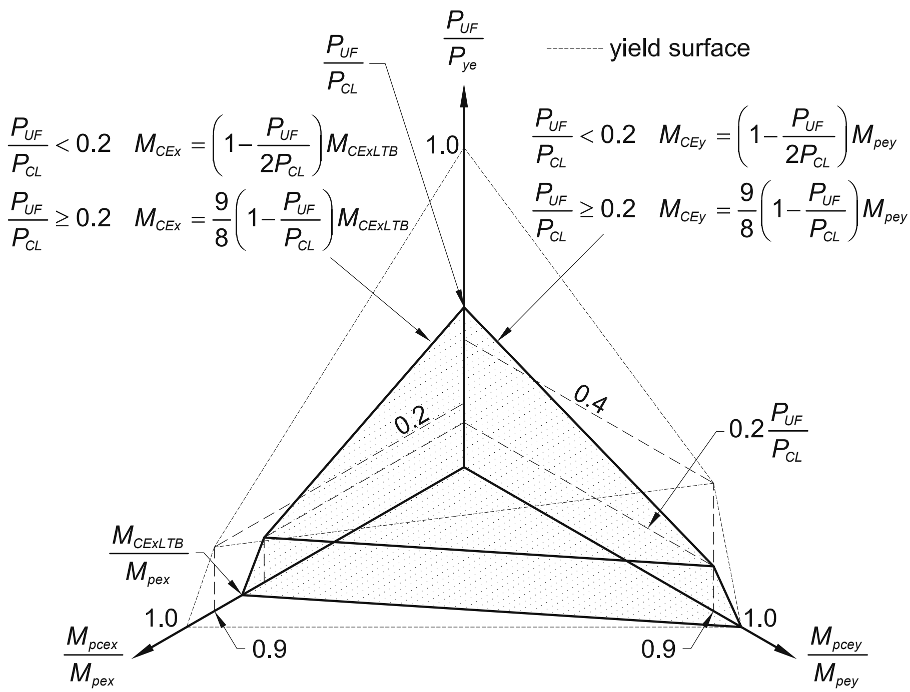


Fig. C-C3.13. Biaxial P-M_x-M_y *interaction surface* (C_b = *1.0).*

For a linear interaction between M_{Cx} and M_{Cy} at P_{UF}, as shown in Figure C-C3.16, $\alpha = \beta = 1.0$. By substituting in the appropriate levels of strength and demand, and taking $\alpha = \beta = 1.0$, Equation C-C3-20 gives Equation C3-9.

A column can be assessed for only in-plane flexural demands when the ratio of out-of-plane flexural demand to out-of-plane plastic flexural strength is less than 0.15. This is done to simplify the assessment of a column when out-of-plane demands do not significantly impact in-plane effects by more than 10%.

b. Nonlinear Analysis Procedures

In determining the modeling parameters and permissible deformations provided in Table C3.6, detailed attention was given to steel columns to incorporate data collected from 80 different experiments on wide-flange steel columns (Popov et al., 1975; MacRae et al., 1990; Newell and Uang, 2008; Ozkula et al., 2017; Elkady and Lignos, 2018a; Cravero et al., 2019; Suzuki and Lignos, 2021) and 329 experiments on HSS steel columns (Yamada et al., 1993, 2012; Ishida et al., 2012; Fadden and McCormick, 2012; Bai and Lin, 2015; Mukaide et al., 2016; Suzuki and Lignos, 2021).

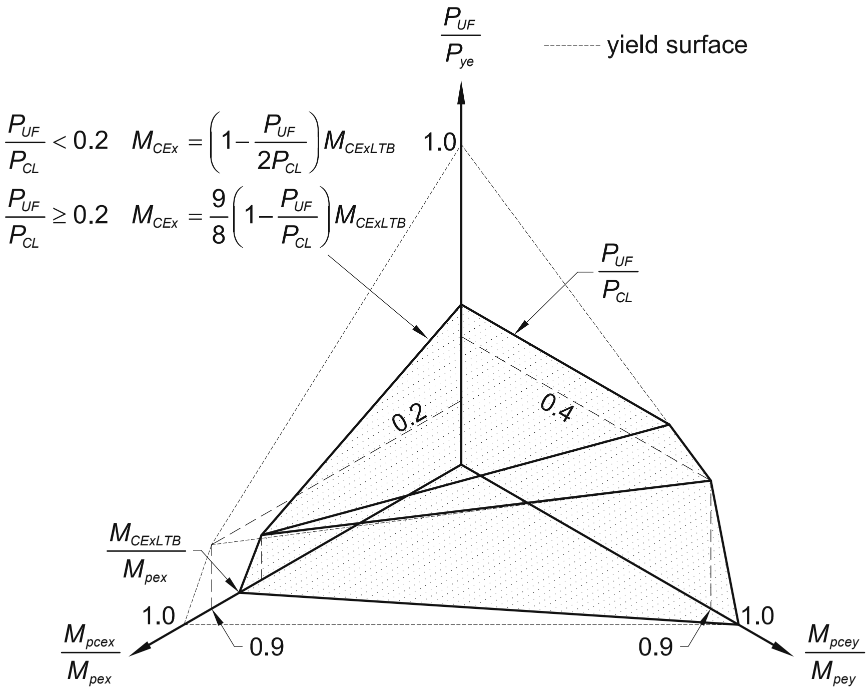


Fig. C-C3.14. Biaxial P-M$_x$-M$_y$ *interaction curves (*C$_b$ *= 1.0)—using the yield surface and upper limit for axial load for the minor axis.*

Wide-flange column experiments were completed by data from computational simulations (Elkady and Lignos, 2018b). For these data, the expressions for modeling parameters *a* and *b* in Table C3.6 were derived from moment chord rotation relationships prior to reaching loss of lateral load-carrying capacity of the wide-flange steel column. These data were fit to expressions computational simulations (Lignos et al., 2019) using multivariate regression. The data were shifted by at least one logarithmic standard deviation to the mean value given by the expressions to represent a safe estimate of deformation capacities of wide-flange steel columns.

Experimental evidence (Newell and Uang, 2008) suggests that complete-joint-penetration (CJP) groove-welded connections of wide-flange steel columns featuring stocky cross sections ($h/t_w < 20$, where h is defined in Table C3.6 and t_w is the thickness of web) could be susceptible to fracture at inelastic cyclic drift demands on the order of 7% to 8%. Therefore, the expressions in Table C3.6 for highly ductile members were capped at 0.07 rad. This value acknowledges the inherent flexibility from the steel column base connection (i.e., embedded or exposed) (Kanvinde et al., 2012; Grilli et al., 2017) to total story deflection.

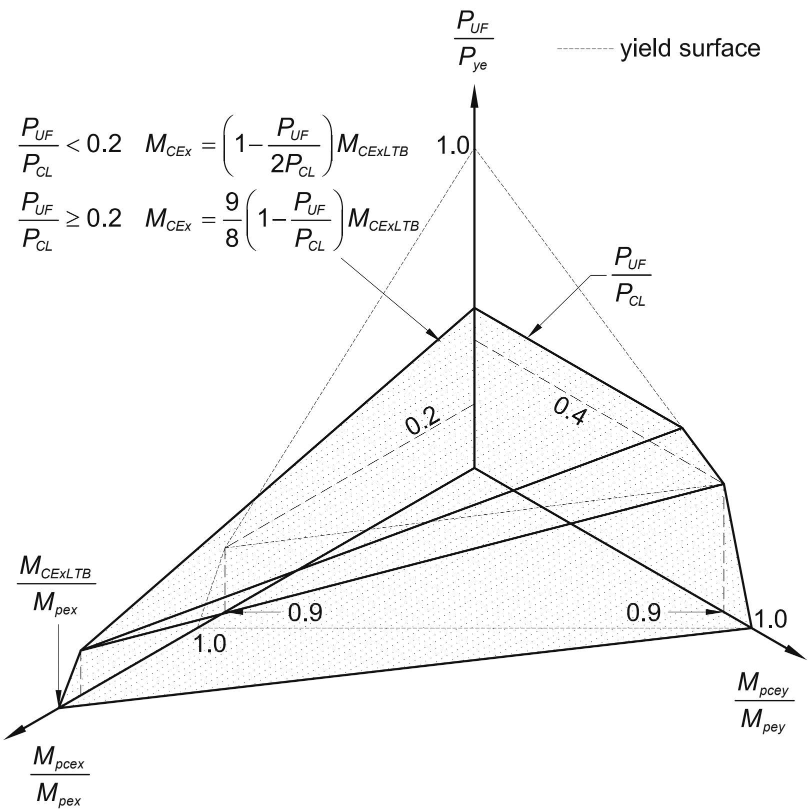


Fig. C-C3.15. Biaxial P-M_x-M_y *interaction curves (*C_b *> 1.0).*

Complementary but limited test data on steel columns featuring built-up box sections suggests that their deformation capacity is about 20% smaller than that of hot-rolled or cold-formed hollow structural steel sections.

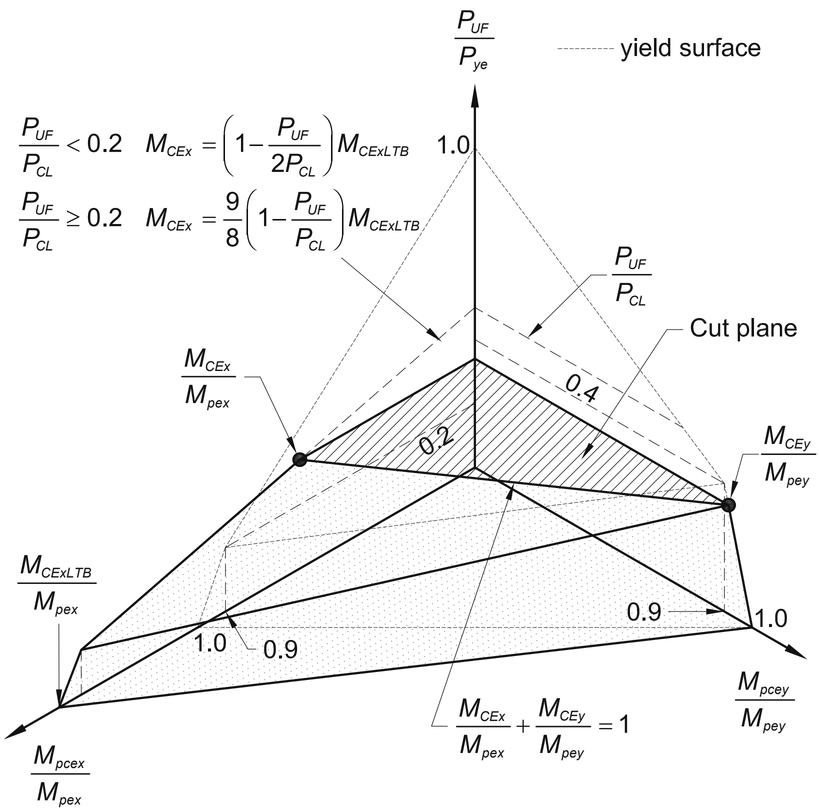


Fig. C-C3.16. Biaxial P-M_x-M_y *interaction curves* (C_b *> 1.0) with cut plane that intersects the interaction curve on each axis.*

C4. PANEL ZONES

Some moment frames designed before the 1994 Northridge, Calif., earthquake (1985 to 1994 era) tended to result in weak panel zones; weak in the sense that the panel zone can yield before the beam. It was shown during the SAC project that weak panel zones are seen to trend towards low levels of total plastic rotation [see FEMA 355D (FEMA, 2000g)]. In particular, test results show that above V_{PZ}/V_{ye} of about 1.10, where V_{PZ} is the panel-zone shear and V_{ye} is the expected shear yield strength of the panel zone, tested subassemblies do not achieve very much ductility—about one-half of the permissible performance parameters for panel zones assumed for joints where the beam-to-column connections are made with conforming weld metal. Large panel-zone shear deformations associated with weak panel zones can instigate fracture of the complete-joint-penetration (CJP) groove weld at the beam-flange-to-column-flange connection. This phenomenon is caused by a flexural plastic hinge developing in the column flanges resulting in a strain concentration at the corners of the panel zone, see Figure C-C4.1. Research (Kim et al., 2015) has shown that the primary cause that can increase the risk of CJP groove weld fracture is excessive column flange bending (or kinking) at the corners of the panel zone, which was suggested to be a function of d_b/t_{cf}, where d_b is the smallest depth of the connecting beams at a panel zone and t_{cf} is the thickness of the column flange.

The permissible performance parameters reflect this risk of weld fracture for weak panel zones. A 50% reduction from factors found in the 2017 edition of ASCE/SEI 41 (ASCE, 2017), and earlier editions, for panel zones was applied to provide a lower-bound limit for pre-Northridge connections made with CJP groove welds. These values reflect approximately the average of the total plastic rotation values shown in FEMA 355D for weak panel zones. These provisions permit an increase in

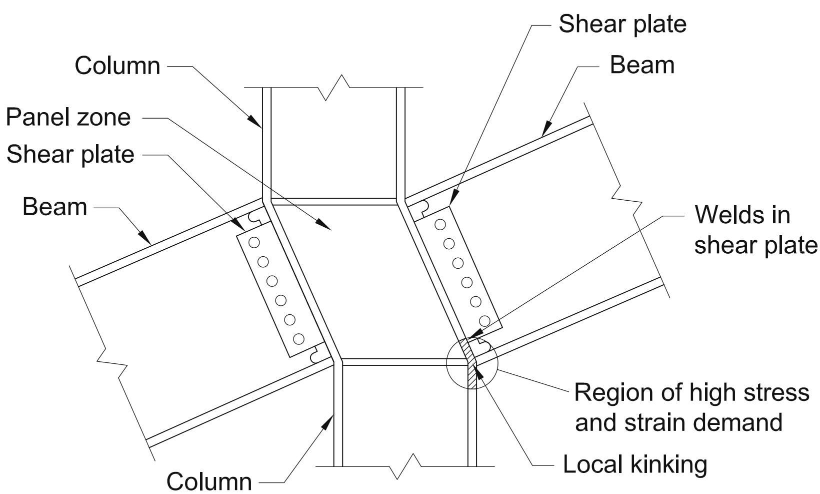


Fig. C-C4.1. Weak panel-zone behavior (Jin and El-Tawil, 2005).

the permissible values when certain beam-to-column connection concerns are satisfied. The permissible performance parameters for panel zones when $V_{PZ}/V_{ye} < 1.10$, with welds conforming to the requirements of the *Seismic Provisions* (AISC, 2022a), Section A3.4, are twice that of those without conforming weld metal. The beam-flange-to-column-flange weld is also not to be located in a region of high strain demand. Equation C4-3 is slightly different from that proposed by Kim et al. (2015) because of the chosen shear yield strain.

Fracture of the beam-flange-to-column-flange weld is not a damage state of the panel zone itself, which is covered by the upper-bound limits. These provisions use panel-zone performance as an indicator of the increased risk of weld fracture and, thus, the potential consequences associated with damage of the beam-to-column connection. Once the weld has fractured, the behavior of the beam-column joint is based on the performance of the beam-column connection following weld fracture. These provisions set the ultimate panel-zone deformation based on the more conservative of the two ultimate rotations of the beam-column connections on each side of the panel zone. This allows the mathematical model to continue to track the nonlinear deformation in the panel-zone hinge, while using that hinge as a surrogate for the deformations in the beam-column connections framing into the panel zone.

C5. BEAM AND COLUMN CONNECTIONS

1. General

Beam and column connections are to be classified as fully restrained (FR) or partially restrained (PR), based on the strength and stiffness of the connection assembly. The beam-to-column connections described in Tables C5.1 and C5.2, and the permissible performance parameters for these connections have been adopted from the referenced SAC documents, FEMA 350 (FEMA, 2000a), FEMA 351 (FEMA, 2000b), FEMA 355D (FEMA, 2000g), and FEMA 355F (FEMA, 2000h). The number of connections identified is based on research that has shown behavior to be highly dependent on connection detailing. The design professional should refer to those guidelines for more detailed descriptions of these connections and a methodology for determining permissible performance parameters for other connections not included in ASCE/SEI 41 or in these Provisions.

FEMA 351 (FEMA, 2000b) identifies two connections, Type 1 (ductile) and Type 2 (brittle). These definitions are not used in ASCE/SEI 41 or these Provisions because the distinction is reflected in the permissible performance parameters for the connections.

Table C5.1 provides a list of common FR beam-to-column connections. The most common FR beam-to-column connection used in steel moment frames since the late 1950s required the beam flange to be welded to the column flange using complete-joint-penetration (CJP) groove welds. Many of these connections have fractured during recent earthquakes. The design professional is referred to FEMA 274 (FEMA, 1997b) and FEMA 351 (FEMA, 2000b).

The evaluation process for beam-to-column connections in the seismic force-resisting system by the design professional should include a review of all welding inspection reports in order to verify compliance with the benchmark codes and standards listed in ASCE/SEI 41, Table 3-2. In jurisdictions where the adopted building code identified in ASCE/SEI 41, Table 3-2, may not have addressed the enhanced welding requirements as identified, at the earliest, in the 1994 UBC Emergency Provisions as issued by International Conference of Building Officials (ICBO) in September/October 1994, the design professional should use other verification techniques as evidence that CJP groove welds are in compliance with the *Seismic Provisions* (AISC, 2022a) welding requirements. CJP groove welds satisfying the welding requirements in the *Seismic Provisions* are notch-tough welds, otherwise the welds should be considered to have limited capacity.

Table C5.2 includes simple shear or pinned connections classified as PR connections. Although the gravity load-carrying beams and columns are typically neglected in the seismic analysis of steel moment-frame structures, SAC research contained in FEMA 355D (FEMA, 2000g) indicates that these connections are capable of contributing some stiffness through very large drift demands. Including gravity load-carrying elements (subject to the modeling procedures and permissible performance parameters in this section) in the mathematical model could be used by the design engineer to reduce the demands on the moment-frame elements.

FEMA 351 (FEMA, 2000b) provides an alternative methodology for determining column demands that has not been adopted into ASCE/SEI 41 because that method ignores axial and flexure interaction. Recent research (Uang et al., 2015) has shown the steel columns can behave differently when subjected to inelastic flexure deformation while under axial force. Various factors account for the difference, such as compactness, axial load ratio, aspect ratio, and slenderness. Columns that are not compact for high ductility, have high axial force, or have high aspect ratios can experience torsional buckling without much inelastic flexural deformation. It is important to account for this behavior, as column buckling can lead to global building collapse.

Welded beam and column splices are treated as force-controlled actions. The strength of CJP groove-welded splices is taken as the strength of the base metal. For partial-joint-penetration (PJP) groove-welded splices (Section C5.3b.3.b), the strength of the splice is taken as the lesser of the fracture of the weld, the fracture of the base metal over the welded portion, or yielding of the section. Column splices made with PJP groove welds have been shown in laboratory testing (Bruneau and Mahin, 1991) and in the 1995 Kobe Earthquake (AIJ, 1995) to fracture, in some cases in a manner that completely severs the column at the splice. The method to determine the fracture stress is based on provisions found in NIST GCR 17-917-46v2 (NIST, 2017a). The fracture toughness parameters, K_{IC}, of Table C5.3 are taken as the mean value correlated against the results of a Charpy V-notch impact test for the weld. Detailed discussion of the correlation between K_{IC} and Charpy V-notch toughness can be found in NIST (2017a). Typical pre-Northridge weld tests show a mean absorbed energy value of approximately 10 ft-lb (14 J). A value of 5 ft-lb (6.8 J) is used as the default if no material testing is performed in order to establish a lower-bound capacity.

2. Stiffness

2a. Beam-to-Column Connections

1. Fully Restrained (FR) Connections

FEMA 355D (FEMA, 2000g) is a useful reference for information concerning nonlinear behavior of various tested connection configurations.

2. Partially Restrained (PR) Connections

In the absence of more rational analysis, the equivalent rotational spring stiffness, K_θ, may be estimated by Equation C-C5-1:

$$K_\theta = \frac{M_{CE}}{0.005} \tag{C-C5-1}$$

where

M_{CE} = expected flexural strength of connection, kip-in. (N-mm), for the following PR connections:

(a) PR connections encased in concrete, and where the connection resistance, M_{CE}, includes the composite action provided by the concrete encasement;

(b) PR connections encased in masonry, where composite action cannot be developed in the connection resistance, M_{CE}; and

(c) Bare steel PR connections (welded or with standard-sized holes).

Where PR connections are encased in concrete but M_{CE} is determined neglecting composite action, and for all other PR connections not addressed by Equation C-C5-1, the equivalent rotational spring stiffness may be estimated by Equation C-C5-2:

$$K_\theta = \frac{M_{CE}}{0.003} \tag{C-C5-2}$$

2b. Column-to-Base Connections

In the absence of more rational analysis, the column base rotational spring can be approximated by Equation C-C5-1.

3. Strength

FEMA 351 (FEMA, 2000b) provides guidance on determining the strength of various FR beam-to-column connection configurations.

4. Permissible Performance Parameters

4a. Deformation-Controlled Actions

1. Beam-to-Column Connections

FEMA 355D (FEMA, 2000g) provides information concerning nonlinear behavior of various tested connection configurations and is the basis for most of the values in Table C5.5. For the ANSI/AISC 358 (AISC, 2022c) conforming welded unreinforced flange-welded web (WUF-W) and reduced beam section (RBS) connection, the parameters in Table C5.5 are based on NIST GCR 17-917-45 (NIST, 2017b). The pre-Northridge welded unreinforced flange-bolted web (WUF-B) values are based on modifications to recommendations found in NIST GCR 17-917-45, specifically the plastic rotation. In NIST GCR 17-917-45, the plastic rotation parameter was set as the rotation at peak moment, as opposed to the rotation at 80% of the post-peak strength. The FEMA 355D plastic rotation parameters were all derived from 80% of post-peak strength, so the WUF-B was modified to remain consistent. In most cases, the values found in the table are based on equations predicting the median response minus a value of 0.01 to account for the beam yield.

The continuity plate modifier provided in Section C5.4a.1.a.1(a) is based on recommendations in FEMA 355F (FEMA, 2000h) for continuity plate detailing in relationship to column flange thickness.

The panel zone modifier provided in Section C5.4a.1.a.1(b) is based on research in FEMA 355F indicating that connection performance is less ductile where the strength of the panel zone is either too great or too small compared with the flexural strength of the beam. The panel-zone strength range between 60% and 90% of the beam strength is considered to provide balanced yielding between the beam and panel zone, which results in more desirable performance.

The beam flange and web slenderness modifiers provided in Section C5.4a.1.a.1(c) are based on the same modifications to beam permissible performance parameters contained in Tables C2.1, C2.3, C3.1, C3.5, C4.1, C5.4, C5.6, and C6.1. Though not an aspect of the connection itself, beam flange and web slenderness affect the behavior of the connection assembly.

The clear span-to-depth ratio modifier provided in Section C5.4a.1.a.1(d) for linear permissible performance parameters reflects the decreased apparent ductility that arises because of increased elastic rotations for longer beams. The decreased plastic rotation capacity of beams with very small L_{cf}/d_b ratios, where L_{cf} is the length of beam taken as the clear span between column flanges and d_b is the depth of beam, is not reflected directly. However, the modifier for linear criteria was developed so that it would be appropriate for the predominant case of L_{cf}/d_b ratios greater than about 5.

FR connections designed to promote yielding of the beam in the span, remote from the column face, are discussed in FEMA 350 (FEMA, 2000a).

C7. BRACED FRAME CONNECTIONS

1. General

A wide variety of connection configurations can be used to join braces to framing members in concentrically, eccentrically, and buckling-restrained braced frames. These connections may be formed using an assemblage of components, such as gusset plates, shear plates, end plates, angles, bolts, and welds to develop the required forces from a linear analysis, demands from a nonlinear analysis, or based on a capacity-based evaluation approach. For concentrically braced frames (CBF) and buckling-restrained braced frames (BRBF), the capacity-based evaluation forces are calculated from the plastic capacity of the braces.

For CBF and BRBF, the tensile and compressive yielding (or compressive buckling for CBF) braces are expected to be the primary yield mechanism. For eccentrically braced frames (EBF), the primary yield mechanism is shear or flexural yielding of the link [either in flexure or shear, depending on the geometry; the reader is referred to the *Seismic Provisions* (AISC, 2022a) for additional information]. Though braces primarily resist axial forces, their end connections may have axial, shear, and flexural demands induced by buckling and yielding of braces and/or frame action. Previous provisions for seismic evaluation have classified all braced frame connection actions as force-controlled, but experimental and computational research conducted in the past decade has shown that connections in CBF can develop secondary yielding mechanisms that enhance system behavior and drift capacity. Specifically, yielding of the gusset plate or bolt-hole elongation are two secondary yield mechanisms that are beneficial to both new and older CBF. In addition, the effects of connections on system stiffness and strength are significant and therefore important for seismic performance evaluation; in particular, that gusset-plate connections provide significant rotational restraint and therefore should be modeled as fully restrained. The new provisions for braced frame connections reflect these advancements in the understanding of braced frame behavior and primarily address the evaluation methods for brace end connections in concentrically braced frames.

Braced frame connections are classified as either rotation-restrained or rotation-accommodating connections based upon the concept for accommodation of brace buckling in the *Seismic Provisions* Section F2.6c.3. Rotation-restrained connections prevent end rotation of the brace, as shown in Figure C-C7.1(a); in CBF with buckling braces, a brace with rotation-restrained connections develops three plastic hinges in the brace after buckling and, thus, such a connection also resists the axial and flexural demands resulting from this action. Rotation-accommodating connections are usually gusset-plate or knife-plate connections where flexural yielding can occur, as shown in Figure C-C7.1(b) and (c). Such a connection is designed to resist only axial demands resulting from brace buckling but has rotational capacity dependent upon detailing. In contrast, BRBF transfer only axial demands (if rotation of the brace end is sufficiently restrained) and therefore the gusset-plate connection is designed to be capable of transferring the full axial capacity of the buckling-restrained brace (BRB) with yielding of the gusset plate permitted after the yield strength of the BRB is achieved.

The evaluation of Whitmore width is required several times in this section. The Whitmore width is recommended to be defined by a 37.6° angle (a 3-4-5 triangle) in this section, because research has shown that this angle provides a more accurate estimate of the strength and resistance of the gusset plate connection than the 30° recommended in the AISC *Steel Construction Manual* (AISC, 2017). In addition, this angle permits use and retention of a larger number of existing connections without excessive retrofit costs.

2. Stiffness

The provisions for stiffness of braced frame connections include the connection components and their effects on adjacent members. Braced frame connection components are assumed to be relatively stiff due to their short lengths. A specific rotational stiffness expression is provided in Equation C7-1 for flexure (a deformation-controlled action) in rotation-accommodating connections, including gusset plates and knife plates. This equation is based on a 37° Whitmore section for determining the gross area, and the average length method proposed by Thornton (1984) for determining the length. The average unrestrained length of gusset plate L_{avg} should be taken as the average of the unrestrained gusset plate lengths to the nearest adjacent member at the Whitmore width ends (L_1 and L_3) and center (L_2). (See Figure C-C7.2.) If the Whitmore width end intersects the adjacent member, the corresponding length should be taken as a negative value. For knife-plate connections, the average unrestrained length should be the linear clearance in the knife plate as shown in Figure C-C7.3.

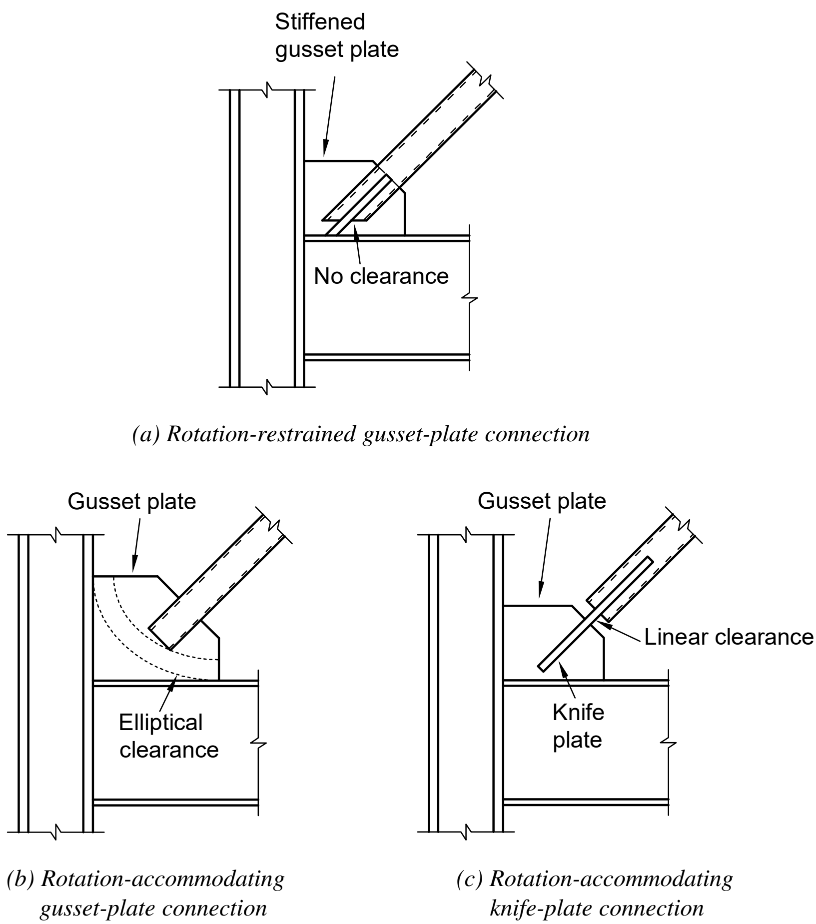


Fig. C-C7.1. Example rotation-restrained and rotation-accommodating connections.

This rotational stiffness is considered in the direction of buckling, and the rotational stiffness in other directions may be assumed to be stiff. When force-controlled actions in connections are modeled explicitly as permitted by ASCE/SEI 41 (ASCE, 2017), appropriate stiffness can be determined from mechanics (e.g., using the Whitmore section for gusset plates under axial load) or empirically derived relationships [e.g., Lesik and Kennedy (1990) for welded connections].

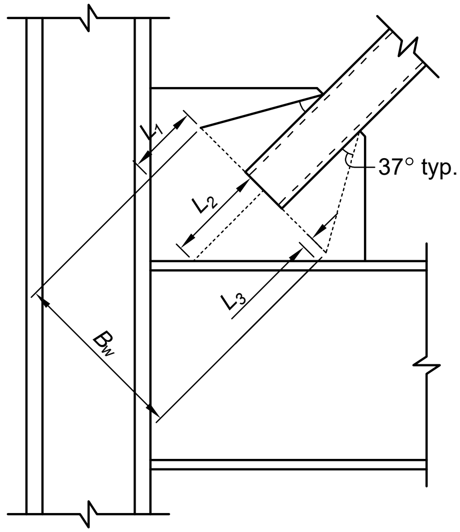
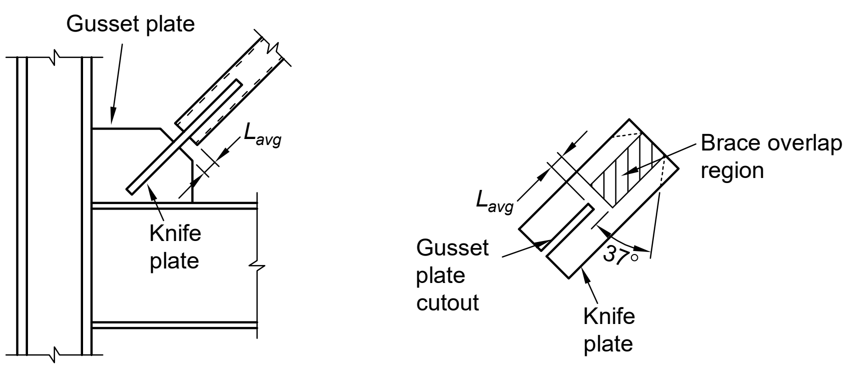


Fig. C-C7.2. Whitmore section, and unrestrained lengths of gusset plate, for evaluation of gusset plate axial and flexural actions.



(a) Unrestrained length of knife plate

(b) Effective width of knife plate limited by plate width

Fig. C-C7.3. Effective width and average unrestrained length for evaluation of axial and flexural actions of knife plates.

The provisions in Sections C7 and E1.2 recommend the use of a sophisticated model for prediction of the inelastic behavior of concentrically braced frames. Such a model leads to much more accurate predictions of braced frame forces and deformations as well as the deformation capacity of braces and gusset plate connections. However, it is recognized the simplified models are often required. For simplified models, pinned connections of the brace to the frame are required. Some documents suggest that fixed brace connections are recommended, but this is a misinterpretation of published results. Fixed brace connections result in significantly underestimating braced frame deformations and overestimating braced frame resistance. Pinned end brace connections tend to result in conservative predictions of braced frame behavior, and it is also recommended that they be used with phenomenological models of the inelastic behavior of the brace, based on Section C3, to improve estimated performance.

The contribution of braced frame connections on system stiffness is significant, especially when the connection restrains relative rotation between the beam and column. Notably, welded gusset plates, including those that are welded directly to the flange of a beam or column and those that are indirectly fastened through shear plates, end plates, or double angles, act like haunches to stiffen the beam-column interface. This finding has been justified through numerical simulation of a large quantity of tests of concentrically and buckling-restrained braced frames with welded gusset plates (Hsiao et al., 2012; Palmer et al., 2016).

The connection model diagrammed in Figure C7.1 corresponds to that used by Hsiao et al. (2012), which showed high fidelity compared to experimental results.

3. Strength

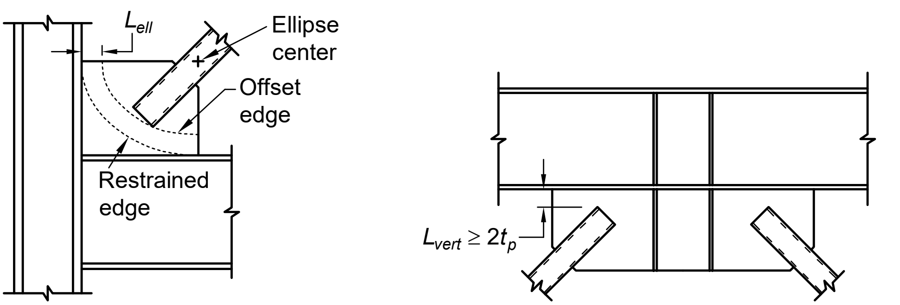
Explicit guidance for evaluating braced frame connection strength is provided for several actions. For use of the Whitmore section, a 37° projection angle may be used. For gusset-plate yielding in tension, the 30° projection angle has been shown to conservatively estimate available strength of gusset plates due to load redistribution upon yielding (Yam and Cheng, 2002). Different effective length factors are given for buckling of corner gusset plates [Figure C-C7.4(a)] and beam midspan gusset plates [Figure C-C7.4(b)]; the latter have larger effective length factors due to the reduced transverse restraint at this location.

Because braced frames are expected to sustain significant inelastic deformation demands in large earthquakes, bolt-hole deformation is a permitted yielding mechanism, provided that the bolt group under evaluation is not the sole load transfer mechanism on the brace-to-frame load path. In Figure C-C7.5, the brace-to-gusset plate bolt group does not meet this requirement, whereas the gusset-plate-to-shear-plate bolt group does.

The strength of welded joints formed with filler metal that does not meet demand critical requirements is reduced to 75% of the nominal strength, which matches the 0.75 resistance factor for welded joints in the Specification, because this failure mode is especially critical. This reduction does not apply to weld strength computed to evaluate the deformation-controlled action of welded gusset-plate rotation. The expression for gusset-plate rotation capacity was calibrated using the full nominal strength (not 75% of the nominal strength).

4. Permissible Performance Parameters

These provisions introduce moment-rotation behavior and permissible performance parameters for rotation-accommodating welded gusset-plate connections that do not meet the requirements of the *Seismic Provisions* (AISC, 2022a). The deformation (rotation) capacity of such connections has been determined from experimental testing of braced frame subassemblages designed to simulate pre-1988 (where 1988 corresponds to the implementation of modern steel seismic provisions) concentrically braced frame construction. Therefore, the gusset plates in these tests had relatively low rotational clearance and welds formed with electrodes that did not meet demand-critical requirements (Sen et al., 2017). Rotational capacity of these connections can be improved by providing greater rotational clearance and overlaying notch-tough weld metal to develop the tensile strength of the plate. The rotation



(a) Corner gusset plate at brace-beam-column intersection

(b) Beam midspan gusset plate with stiffeners at brace-beam intersection

Fig. C-C7.4. Example gusset-plate connections.

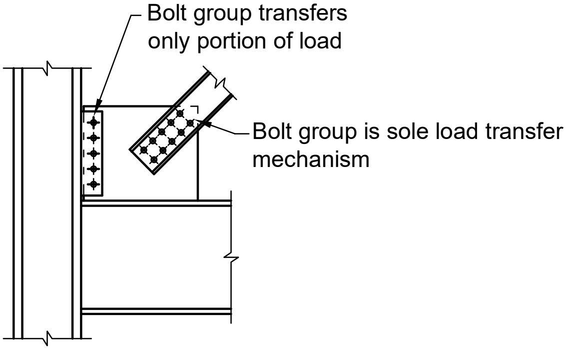


Fig. C-C7.5. Example bolt groups in braced frame connections.

parameters for these connections are converted to brace axial deformations for linear procedures and nonlinear procedures in which brace buckling is not explicitly modeled assuming the brace deflected shape is triangular and small-angle approximations are permissible.

For the limit states of gusset-plate yielding in tension, bolt fracture in shear, and bearing and tearout of bolt holes in shear, demand-capacity ratios exceeding unity are permitted. The larger demand-capacity ratios are justified from experimental observations of numerous tests simulating pre-1988 concentrically braced frames (Sen et al., 2017). For bolt fracture in shear, the larger demand-capacity ratio is permitted only if the bolt fracture resistance is not significantly lower than that for bearing or tearout of the corresponding connected material. This provision is intended to promote bolt-hole elongation, which precludes bolt fracture.

CHAPTER D

STRUCTURAL STEEL MOMENT FRAMES

D1. GENERAL

Steel moment frames are those frames that develop their seismic resistance through bending of steel beams and columns and moment-resisting beam-to-column connections. A moment-resisting beam-to-column connection is one that is designed to develop moment resistance at the joint between the beam and the column and also designed to develop the shear resistance at the panel zone of the column. Beams and columns consist of either hot-rolled steel sections or built-up members from hot-rolled plates and sections. Built-up members are assembled by riveting, bolting, or welding. The components are either bare steel or steel with a nonstructural coating for protection from fire or corrosion, or both, or steel with either concrete or masonry encasement. The behavior of steel moment-resisting frames generally depends on the connection configuration and detailing.

FEMA 351 (FEMA, 2000b) identifies connections as Type 1 (ductile) and Type 2 (brittle). These definitions are not used in ASCE/SEI 41 (ASCE, 2017) or these Provisions because the distinction is reflected in the permissible performance parameters for the connections.

The most common fully restrained (FR) beam-to-column connection used in steel moment frames since the late 1950s required the beam flange to be welded to the column flange using complete-joint-penetration (CJP) groove welds. Many of these connections have fractured during recent earthquakes. The design professional is referred to FEMA 274 (FEMA, 1997b) and FEMA 351 (FEMA, 2000b).

The evaluation process for beam-to-column connections in the seismic force-resisting system by the design professional should include a review of all welding inspection reports in order to verify compliance with the benchmark codes and standards listed in ASCE/SEI 41, Table 3-2. In jurisdictions where the adopted building code identified in ASCE/SEI 41, Table 3-2, may not have addressed the enhanced welding requirements as identified, at the earliest, in the 1994 UBC Emergency Provisions as issued by the International Conference of Building Officials (ICBO) in September/October 1994, the design professional should use other verification techniques as evidence that complete-joint-penetration (CJP) groove welds are in compliance with the *Seismic Provisions* (AISC, 2022a) welding requirements. CJP groove welds satisfying the welding requirements in the *Seismic Provisions* are notch-tough welds; otherwise the welds should be considered to have limited capacity.

Table C5.2 includes simple shear or pinned connections classified as partially restrained (PR) connections. Although the gravity load-carrying beams and columns are typically neglected in the seismic analysis of steel moment-frame structures, SAC research contained in FEMA 355D (FEMA, 2000g) indicates that these connections are capable of contributing some stiffness through very large drift demands. Including gravity load-carrying elements (subject to the modeling procedures and permissible performance parameters in this section) in the mathematical model could be used by the design engineer to reduce the demands on the moment-frame elements.

D2. STIFFNESS

1. Linear Analysis Procedures

FEMA 274 (FEMA, 1997b) is a useful reference for information concerning stiffness properties and modeling guidelines for PR connections.

2. Nonlinear Static Procedure

Equations C2-2 and C3-15 for computing the yield chord rotation, θ_y, of a beam and a column, respectively, assume that the rotations at the two ends of the beam or column are equal (i.e., double-curvature bending with an inflection point at midspan). Consequently, plastic chord rotation in Table C2.2 and Table C3.6 assume that the plastic rotation at the ends of the beam or column are equal. It is common practice to assume that chord rotation and rotation in a plastic hinge are equivalent. However, this assumption can be violated when boundary conditions restrain the ability to essentially have equal end rotations.

Strain hardening should be considered for all components. Permissible values for the post-elastic slope are provided in Chapter C.

Research performed by Newell and Uang (2006, 2008) indicated that elastic shear deformation can contribute significantly (10 to 50%) to the total rotation in stocky columns. The term $(1 + \eta)$ in Equations C2-2 and C3-15 adjusts the yield chord rotation resulting from flexure to account for the effect of shear deformation on the elastic curve. This adjustment to the flexural stiffness can be found in textbooks covering advanced structural analysis. The criterion for the 5% variation on stiffness to address component-specific phenomena in Section D2.2(c), including shear deformations, was considered a reasonably low percentage based on engineering judgment. Shear deformations are typically included by default in commercial structural analysis software and the analyst has to manually turn this feature off.

Equation C3-15 accounts for the change in rotation resulting from shear deformation but does not include the change in flexural stiffness from the axial load. Using the geometric stiffness matrix, the yield chord rotation can be determined as

$$\theta_y = \underbrace{M_{pe}\left(1-\frac{|P|}{P_{ye}}\right)}_{\text{strength}} \underbrace{\frac{L(1+\eta)}{6(\tau_b E)I\left(1+\dfrac{\pi^2}{60(1+\eta)}\dfrac{P}{P_E}\right)}}_{\text{stiffness}} \tag{C-D2-1}$$

where

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

I = moment of inertia about the axis of bending, in.^{4} (mm^4)
L = length of span, in. (mm)
M_{pe} = expected plastic flexural strength, kip-in. (N-mm)
P = axial force (compressive or tension), kips (N)
P_{ye} = expected axial yield strength, kips (N)
= $F_{ye}A_g$
A_g = gross area of cross section, in.^{2} (mm^2)
F_{ye} = expected yield stress, ksi (MPa)
P_E = elastic critical buckling strength of a member in the plane of bending, kips (N)

$$= \frac{\pi^2 EI}{L^2}$$

$$\eta = \frac{12EI}{L_{CL}^2 GA_s}$$

A_s = effective shear area of the cross section, in.^{2} (mm^2)
G = shear modulus of elasticity of steel
= 11,200 ksi (77 200 MPa)
L_{CL} = length of beam taken between column centerlines, in. (mm)
τ_b = stiffness reduction parameter, as given in *Specification* Chapter C, except that P_{ye} is substituted for P_{ns} and P_{UF} is substituted for αP_r

This formula accounts for the local second-order effect (P-δ). Global second-order effects (P-Δ) do not influence the flexural stiffness of a column. P is taken as negative when in compression. This adjustment is not included in Equation C3-15 because the column length required to get an approximate 15% reduction in rotational stiffness and limit P_E to $0.5P_{ye}$ (elastic case when τ_b = 1.0) is much greater than conventional story heights and therefore can be ignored. Furthermore, local second-order effects are generally not explicitly included in structure analysis software packages. Typically, these software packages recommend subdividing columns to implicitly account for local second-order effects.

Equations C2-2 and C3-15 do not account for stiff end zones at the ends of the beam or column, nor do they address the condition when the anticipated plastic hinge locations are some distance away from the ends of the beam or column. For example, a beam with strong panel zones (rigid) and plastic hinges located at the face of the column, the yield chord rotation can be determined from

$$\theta_y = \frac{M_{pe}L_{cf}\left(1+\eta'\right)}{6EI}\left(\frac{L_{cf}}{L_{CL}}\right) \tag{C-D2-2}$$

where

L_{cf} = length of beam taken as the clear span between column flanges, in. (mm)

$$\eta' = \frac{12EI}{L_{cf}^2 GA_s}$$

In this equation, M_{pe} is measured at the face of the column and the lengths of the rigid end zones at the ends of the beam are assumed to be equal. Therefore, the rotation at the end of the beam (at joint) and at the face of the column (start of end zone) are equal.

A yield surface is the plastic capacity of a cross section (P_{ye}-M_{pe} interaction curve). The surface is based on full yielding of the cross section and does not capture the effects of global member buckling on the capacity of the plastic hinge. As a result, the cross-section elements have to be classified as compact for compression (i.e., capable of sustaining some inelastic strains beyond yield before local buckling occurs) in order to develop a fully yielded section. Information concerning the yield surface given by Equations C3-5 and C3-6 can be found in the *Specification* Commentary Chapter H. This interaction curve was selected to be applicable to many column shapes. It can be conservative for specific actions (e.g., plastic hinging for bending about the weak axis of a wide-flange shape).

Equations C3-5 and C3-6 are a linear approximation of the nonlinear yield surface. A nonlinear formulation for the yield surface is

$$M_{CE} = M_{pce} = M_{pe}\sqrt[\beta]{1-\left(\frac{|P|}{P_{ye}}\right)^{\alpha}} \tag{C-D2-3}$$

where

M_{CE} = expected flexural strength, kip-in. (N-mm)

M_{pce} = expected plastic flexural strength of the cross section in the presence of axial force, P, kip-in. (N-mm)

M_{pe} = expected flexural strength of the cross section in the absence of axial force, kip-in. (N-mm)

α = exponent for nonlinear yield surface

β = exponent for nonlinear yield surface

The exponents (α and β) can be determined to provide the best fit to test results for plastic hinges developed in beam-columns. This type of formulation is useful because it aligns with column hinge models provided in commercially available structural analysis software packages.

Panel-zone strength is determined according to *Specification* Section J10.6(a) and is targeted at full yielding of the web, and doubler plates if any, and does not include post-yield strength contributions from thick column flanges as can be found in *Specification* Section J10.6(b). This is because the derivation of these equations assumed a shear strain ductility of 4, which is beyond the deformation associated with full yielding of the web.

3. Nonlinear Dynamic Procedure

See Commentary Sections C3.4b and D2.2 for information regarding the yield surface for plastic hinges located in columns. FEMA 355D (FEMA, 2000g) is a useful reference for information concerning nonlinear behavior of various tested connection configurations.

The plastic rotation angles in Table C3.6 for plastic hinges in a column are provided for columns subjected to axial compression force and for columns subjected to axial tension force. In most framing configurations, the column in compression will control the assessment; however, there may be a rare case where columns are subjected to sustained tension forces. It is conservative to apply the values provided for a column in compression to define the flexural backbone curve and permissible plastic rotations for the column in tension. For stocky columns having low width-to-thickness ratios with the nominal axial strength, P_n, approximately equal to the axial yield strength, P_y, there will not be much difference between the responses (i.e., a symmetric hysteresis curve). Analytical research (Newell and Uang, 2008) has suggested that deep, slender columns can have different responses.

Some tests were done with constant axial loads maintained throughout the tests, while others had some initial axial load applied with a small amount of axial load cycled throughout the test. In the latter cases, the constant applied portion of the total axial load was used in the statistical analysis of the test results. As a result, it is permitted to use the constant axial load in the column as the basis for the modeling parameters and permissible performance parameters. This constant axial load is typically taken as the axial force component of the gravity load as determined by ASCE/SEI 41, Equation 7-3, P_G, in the column. This is a significant change from past versions of ASCE/SEI 41, Chapter 9, which used the total axial force, P, as the basis for plastic rotation angles in columns—requiring the rotations to be updated at every time step throughout the analysis.

Testing has shown that plastic hinges in compact, stocky columns with constant axial load ratios not exceeding $0.6P_{ye}$ can have plastic deformation capacity. This capacity is a function of the member and section slenderness parameters described in Table C3.6. Alternatively, plastic hinges in stocky columns with constant axial load ratios exceeding $0.6 \times P_{ye}$ have reduced plastic deformation capacity and are therefore not permitted to yield in these Provisions. Columns with gravity loads equal to or exceeding $0.6 \times P_{ye}$ are likely insufficient to support design gravity load combinations. Furthermore, some tests (Ozkula and Uang, 2015) have illustrated that member slenderness, L/r_y, where L is the laterally unbraced length of the member and r_y is the radius of gyration about the y-axis, can influence the plastic deformation capacity at various axial load ratios in deep slender columns. Therefore, the column is not permitted to yield when any modeling parameter goes to zero.

The modeling parameters and evaluation criteria for plastic hinges in structural steel columns in tension are the same as those provided in past versions of ASCE/SEI 41, Chapter 9, and therefore remain a function of θ_y given by Equation C3-15.

The 3% strain hardening recommendation given in Section C3 is generally conservative for plastic hinges that develop in structural steel columns. Research (Elkady and Lignos, 2015) has shown that larger strain hardening values are possible.

In past versions of ASCE/SEI 41, Chapter 9, columns in compression were classified as force-controlled for flexure when $P/P_{CL} > 0.5$, where P_{CL} is the lower-bound compressive strength, and lower-bound material properties were used to compute component strengths, Q_{CL}. The change in column properties could not be implemented in the nonlinear procedures efficiently. New criteria in Table C3.6 for columns are based on column hinges being deformation-controlled for flexure (using expected material properties). At a specific axial force ratio, P_G/P_{ye} (compression or tension), the column hinges are not permitted to yield, in lieu of switching to a force-controlled mechanism. Column member stability verifications are included that use lower-bound material properties when required.

D3. STRENGTH

FEMA 351 (FEMA, 2000b) provides guidance on determining the strength of various fully restrained (FR) beam-to-column connection configurations.

FEMA 355D (FEMA, 2000g) provides information concerning nonlinear behavior of various tested connection configurations.

D4. PERMISSIBLE PERFORMANCE PARAMETERS

1. General

The strength and behavior of steel moment-resisting frames is typically governed by the connections. It is recommended that the controlling limit state of the system be determined when selecting the corresponding acceptance criterion.

2. Linear Analysis Procedures

Columns. The component capacity modification factors, m, for the linear procedures have been taken from the 2017 edition of ASCE/SEI 41, which are principally unchanged from those provided in the 2013 edition of ASCE/SEI 41 (ASCE, 2013). How m is applied in a structural assessment has been technically revised beginning with the 2017 edition of ASCE/SEI 41, and with these Provisions, to be consistent with the intended use of the assessment procedures. So doing resulted in revising the axial load ratio to match that initially recommended in FEMA 273 (FEMA, 1997a) (which used P/P_{ye})—also see Commentary Section D2 for additional information. Using the original equations to capture P-M interaction effects on m results in $m = 0$ at $P/P_{ye} = 0.6$ (taken from the P-M interaction for the nonlinear procedures, which results in plastic chord rotation, $\theta_p = 0$, when $P/P_{ye} = 0.6$). The equations for P-M interaction are revised in these Provisions so that they result in $m = 1$ at $P/P_{ye} = 0.6$. P is kept as P_{UF} as an estimate of the total expected axial force in the column since the effective values of m were not significantly changed from those prescribed in FEMA 273 (FEMA, 1997a), where P_{UF} is the axial compressive force determined in accordance with ASCE/SEI 41, Section 7.5.2.1.2 for linear analysis procedures; with ASCE/SEI 41, Section 7.4.3.3, for nonlinear static procedure; and with ASCE/SEI 41, Section 7.4.4.3, for nonlinear dynamic procedure. Furthermore, the values of m have not been calibrated to the permissible performance parameters for columns using the nonlinear procedures, which explicitly use P_G/P_{ye} to match that used in the regression analyses. Future efforts should evaluate calibrating the permissible performance parameters for the two assessment philosophies, which may result in the linear procedures similarly using P_G/P_{ye}.

The width-to-thickness ratios of the cross-section elements in compression at axial load ratios of zero and 0.2 were changed in ASCE/SEI 41 to match the compactness requirements in the *Seismic Provisions* (AISC, 2022a). Though the terms are not used in ASCE/SEI 41 or these Provisions, the lower-bound curve matches that for highly ductile elements and the upper-bound curve matches those for moderately ductile elements (which matches the compactness requirements in the *Specification*). The axial load ratio of 0.2 was selected to align with other provisions (i.e., yield surface) and the permissible performance parameters in ASCE/SEI 41. This is slightly different from using the axial load ratio of 0.125 in the *Seismic Provisions* based on plastic design theory. A linear change is adopted between axial load ratios of zero and 0.2. In these Provisions, the compactness criteria are revised to match the *Seismic Provisions* λ_{hd} and λ_{md} to simplify the assessment and align the two standards, though the axial load used in the equations is required to match that specified in ASCE/SEI 41 (i.e., P_{UF} or P_G). This change does remove the connection at an axial load ratio of 0.2 that is prevalent in ASCE/SEI 41 and these Provisions. Future editions of these Provisions, the *Seismic Provisions*, and the *Specification* will aim to align the compactness requirements and permissible performance parameters. The compactness requirements are not applicable to cross-section elements in tension.

The values of m are chosen to be equal to a beam at an axial load ratio of 0.2. Uniaxial *P-M* interaction reduces these values to $m = 1$ at $P_{UF}/P_{ye} = 0.6$. This is slightly different from past versions of ASCE/SEI 41, Chapter 9, which treated the component as force-controlled, but is consistent in that a column hinge does not yield when $m = 1$.

The axial load basis of P/P_{ye} is maintained to be consistent among all parameters.

Many older frames may have steel columns with reinforced concrete encasement for fire protection. The composite stiffness and resistance of these members may be significant, but the composite resistance may be lost at larger deformations if the concrete encasement does not have adequate confinement. It may frequently be advantageous to use this increased resistance, but this can be done only where the increased resistance is justified by analysis of the composite section, including full consideration of the ductility and inelastic deformation capacity of the member.

FR connections designed to promote yielding of the beam in the span, remote from the column face, are discussed in FEMA 350 (FEMA, 2000a).

3. Nonlinear Analysis Procedures

Columns. As in the linear procedures, flexural hinges in columns are checked for yielding (section strength) and the column members are checked for stability (member strength). Section strength is verified by evaluating the permissible performance parameters so that the rotation demand does not exceed the permissible plastic rotation for a given performance level. Member strength is verified using the same *P-M* interaction equations applicable for the linear procedures. An elastic column can generally be checked neglecting the moment contribution so that $P/P_{CL} \le 1.0$ is verified. However, testing (Ozkula et al., 2017) has shown that deep, slender wide-flange columns are susceptible to out-of-plane buckling modes during cyclic motions after plastic hinges have developed at both ends. If the column has developed flexural plastic hinges, the maximum moment demand will commonly be at the hinge and follow the yield surface (adjusted for stain hardening) where M_{pce} changes as P changes. Depending on the denominator in the moment term, this case may result in P/P_{CL} being compared to some number less than unity. The lateral-torsional buckling strength in the denominator should also include modification by the lateral-torsional buckling modification factor, C_b, for nonuniform moment diagrams when both ends of the segment are braced, as defined in *Specification* Chapter F. When computing P_{CL}, it is generally acceptable to use an effective length factor of unity unless a smaller value is justified by analysis.

Many older frames may have steel columns with reinforced concrete encasement for fire protection. The composite stiffness and resistance of these members may be significant, but the composite resistance may be lost at larger deformations if the concrete encasement does not have adequate confinement. It may frequently be advantageous to use this increased resistance, but this can be done only where the increased resistance is justified by analysis of the composite section, including full consideration of the ductility and inelastic deformation capacity of the member.

D5. RETROFIT MEASURES

The following measures, which are presented in greater detail in FEMA 351 (FEMA, 2000b), may be effective in retrofitting moment frames with FR connections:

(a) Add steel braces to one or more bays of each story to form concentrically or eccentrically braced frames to increase the stiffness of the frames. The attributes and design criteria for braced frames are specified in Chapter E. The location of added braces should be selected so as to not substantially increase horizontal torsion in the system.

(b) Add concrete or masonry shear walls or infill walls to one or more bays of each story to increase the stiffness and strength of the structure. The attributes and design requirements of concrete and masonry shear walls are specified in ASCE/SEI 41, Sections 10.7 and 11.3, respectively. The attributes and design requirements of concrete and masonry infills are specified in ASCE/SEI 41, Sections 10.6 and 11.4, respectively. The location of added walls should be selected so as not to substantially increase horizontal torsion in the system.

(c) Attach new steel frames to the exterior of the building. The retrofitted structure should be checked for the effects of the change in the distribution of stiffness, the seismic load path, and the connections between the new and existing frames. The retrofit scheme of attaching new steel frames to the exterior of the building has been used in the past and has been shown to be effective under certain conditions. This retrofit approach may be structurally efficient, but it changes the architectural appearance of the building. The advantage is that the retrofit may take place without disrupting the use of the building.

(d) Reinforce moment-resisting connections to force plastic hinge locations in the beam material away from the joint region to reduce the stresses in the welded connection, thereby reducing the possibility of brittle fractures. This scheme should not be used if the welded connections in the existing structure did not use weld material of sufficient toughness to avoid fracture at stresses lower than yield or where strain-hardening at the new hinge location would produce larger stresses than the existing ones at the weld. The retrofit measures to reinforce selected moment-resisting connections should consist of providing horizontal cover plates, vertical stiffeners, or haunches. Removal of beam material to force the plastic hinge into the beam and away from the joint region can also be used subject to the foregoing restrictions. Guidance on the design of these modifications of FR moment connections is discussed in FEMA 351.

(e) Add energy dissipation devices as specified in ASCE/SEI 41, Chapter 15.

(f) Increase the strength and stiffness of existing frames by welding steel plates or shapes to selected members.

The retrofit measures for moment frames with FR connections may be effective for moment frames with PR connections as well. Moment frames with PR connections are often too flexible to provide adequate seismic performance. Adding concentric or eccentric bracing or reinforced concrete or masonry infills may be a cost-effective retrofit measure.

PR connections are usually components that are weak, flexible, or both. Connections may be retrofitted by replacing rivets with high-strength bolts, adding weldment to supplement rivets or bolts, or welding stiffeners to connection pieces or combinations of these measures. Refer to FEMA 351 for additional information concerning the retrofit of moment frames with PR connections.

CHAPTER E

STRUCTURAL STEEL BRACED FRAME AND STEEL PLATE SHEAR WALL REQUIREMENTS

Steel braced frames act as vertical trusses where the columns are the chords and the beams and braces are the web members. In standard braced frame configurations, connections between braces and beams and columns are typically made with gusset plates. Gusset plates at brace-to-beam or brace-to-column intersections can have a significant effect on the rigidity of beam-to-column connections, even for simple framing connections, when the size of the gusset plate is reasonably large. Column bases connected to braces at grade level are mainly subjected to large axial and shear loads, with small secondary moments in the elastic state.

Components can be bare steel, steel with a nonstructural coating for fire protection, or steel with concrete or masonry encasement.

The use of concentrically braced frames (CBF) as seismic force-resisting systems has a long history as compared with the more recent use of buckling-restrained braced frames (BRBF). Seismic design and detailing of CBF have evolved over time, and code requirements have been continually updated. Modern seismic design codes for structural steel such as the *Seismic Provisions* (AISC, 2022a) place great attention on section compactness, component slenderness, and seismic detailing of connections to ensure ductile behavior and acceptable performance. Thus, when modeling inelastic deformation capacities for nonductile connections and components of older existing frames, the modeling parameters that are applicable to ductile detailing and compact sections as presented in this section should be used with caution. In lieu of experiments, engineering judgment and application of approved methods using engineering mechanics are permitted with proper prediction of inelastic deformation or consideration of expected nonductile behavior.

E1. CONCENTRICALLY BRACED FRAMES (CBF)

1. General

The connection response and beam and column behavior have a strong influence on the seismic performance of CBF. In contrast to the intended performance of BRBF described in Section E3, the braces in CBF are likely to buckle both globally and locally in compression under large seismic demands, resulting in strength reduction and stiffness degradation of framing members and increasing inelastic demands for their connections after buckling.

Provisions of ASCE/SEI 41 (ASCE, 2017) for CBF have largely remained unchanged from the original documentation provided in FEMA 273 (FEMA, 1997a). Over the last two decades, there has been significant research investigating the seismic performance of braced frame systems (as opposed to individual components). This work has yielded significant advances in nonlinear structural modeling, design for performance objectives and ductility, and evaluation and retrofit of existing CBF (pre-1988, where 1988 corresponds to the implementation of modern steel seismic provisions). To reflect this work, these Provisions include a significant reorganization of the ASCE/SEI 41 provisions.

The special concentrically braced frame (SCBF) in *Seismic Provisions* Section F2 uses the principles of capacity-based design to ensure that the inelastic capacity of the brace controls the seismic capacity of the frame. As a result, Section C7 does not apply to SCBF meeting all requirements of *Seismic Provisions* Section F2; however, Sections E1 and C3 are required for those systems.

Different configurations of braced frames are used, and Figures C-E1.1 and C-E1.2 define components and configurations discussed here.

2. Stiffness

2b. Nonlinear Analysis Procedures

Much research has taken place in recent years to better understand and quantify the nonlinear behavior of CBF. Useful references for information regarding nonlinear load-deformation behavior of braces and related connections include PEER/ATC 72-1 (PEER, 2010), Aviram et al. (2010), Davaran and Far (2009), Fahnestock and Stoakes (2009), Fell et al. (2009, 2010), FEMA 274 (FEMA, 1997b), FEMA

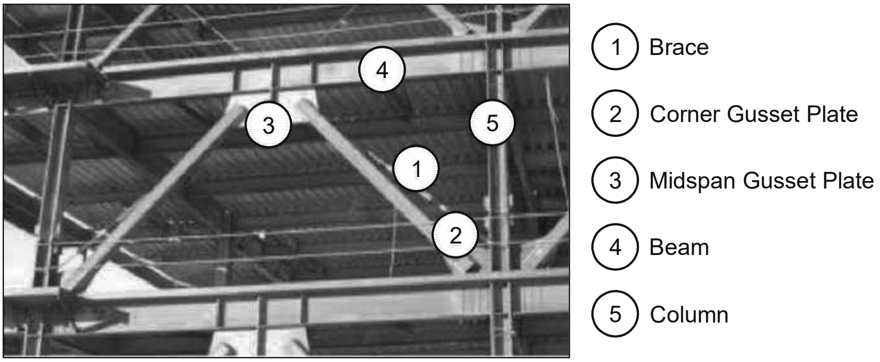


Fig. C-E1.1. Typical CBF components.

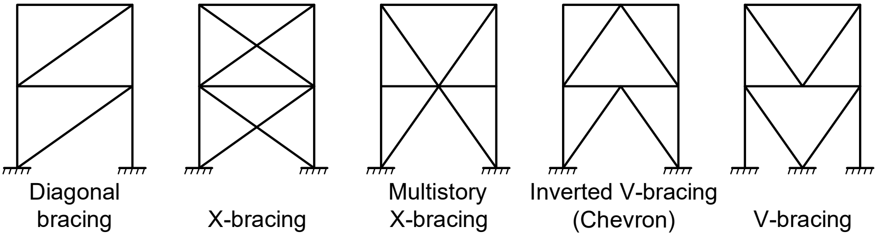


Fig. C-E1.2. Typical CBF configurations.

P-440A (FEMA, 2009), Jordan (2010), Liu and Astaneh-Asl (2000), NIST (2010a, 2010b), Richards (2009), Roeder et al. (2004, 2009a, 2009b, 2011b), Uriz and Mahin (2008), Stoakes and Fahnestock (2010), Yang (2006), Yang et al. (2008), Yoo (2006), Yoo et al. (2008, 2009), and Zhang et al. (2011).

Recent research has shown that allowing desirable controlled yielding to occur at multiple locations (i.e., in gussets and beams, in addition to braces) increases the inelastic deformation capacity of SCBF (Roeder et al., 2011b). Also, providing flexural strength and rigidity at the beam-to-column connections for nonductile CBF can increase the redundancy and improve resistance against collapse after the buckled brace fractures.

Nonlinear analysis is increasingly used in seismic design, evaluation, and retrofit. Considerable research on the inelastic dynamic analysis has been completed, and this analytical research has been compared to experimentally measured behavior to establish the accuracy and reliability of the generalized force-displacement and fiber-based line element nonlinear procedures. This necessitates the use of computer analysis programs that have the capability to simulate nonlinearity due to material behavior and geometric effects of the system (i.e., P-Δ) and individual members (i.e., buckling and P-δ). The deformation demands and forces of all elements, connections, and components are required by these Provisions to be evaluated using the appropriate limits provided in Sections C3 and C7. The following guidance has been verified to provide reasonable accuracy in past research studies.

Analysis with Generalized Force-Deformation Relation. Brace and brace end-connection behavior may be represented by the generalized force-deformation relation given in Figure C3.1. Different force-deformation relations are used to represent the tensile and compressive brace behaviors consistent with Section C3.4a, Table C3.4, and the requirements of Section C3.2. The brace response envelope is to have deteriorated compressive resistance after brace buckling and no capacity after brace fracture. The resistance of beams, columns, and beam-column connections should be directly modeled in accordance with Sections C3, C4, C5, and C7 and consider their nonlinear behavior, as appropriate. Beam-column connections where a gusset plate is attached to both the beam and column should be considered fully or partially restrained consistent with Section C5. When the generalized force-deformation relations are used to represent the brace axial behavior, the brace flexural stiffness should be neglected. This method has been shown to provide acceptable comparison between computed behavior and measured experimental results.

Analysis with Fiber-Based Line Elements. Nonlinear, fiber-based beam-column elements (also referred to as line elements) can be used to configure the CBF system, including members and connections, with the following constraints.

(a) Each brace should have an initial displacement in the direction of buckling in the shape of a sine curve with amplitude $L_{ee}/500$, where L_{ee} is the end-to-end brace length, to provide an accurate representation of the buckling force.

(b) Braces should be simulated with 10 or more nonlinear elements along the brace length. Beams and columns should be simulated with at least four nonlinear elements along the member length for displacement-based element formulations and at least one nonlinear element along the member length for force-based element formulations. Each element should have at least four integration points along the length.

(c) The cross sections of all members should be segmented with at least four fiber layers through the cross-sectional dimensions with stress variation due to flexure and at least two fiber layers through the cross-sectional dimensions with no variation in normal stresses resulting from flexure, as shown in Figure C-E1.3.

(d) The constitutive models of all steel elements are to represent bilinear inelastic behavior with kinematic hardening and include nonlinear geometric effects. The use of advanced constitutive relationships capable of simulating the Bauschinger effect is permitted. Expected or measured yield stress of the steel should be used, and 1% strain hardening should be employed unless additional information is available to justify a different value.

(e) For the beam-column-gusset connections, rigid offsets for braces, beams, and columns at gusset-plate connections should be employed in accordance with Section C7.2c and as shown in Figure C-E1.4 to simulate the enhanced connection stiffness. Nonlinear rotational springs with constitutive behavior described previously in part (d) and stiffness and strength based upon the gusset plate and end clearance of the brace should be employed as described in Section C7 and as shown in Figure C-E1.4.

(f) All other connections (e.g., beam-to-column and column base) should be modeled as fully restrained (FR) or partially restrained (PR) connections as appropriate for the structure.

This method has been shown to provide a very accurate comparison between computed behavior and measured experimental results. Models have been developed to simulate brace or connection fracture and to analyze system response after these initial fractures. While this is clearly the most accurate and economically efficient method currently available for predicting nonlinear braced frame response, it will not predict local buckling and other similar local deformations. Higher-resolution finite element models are required to capture these local response effects.

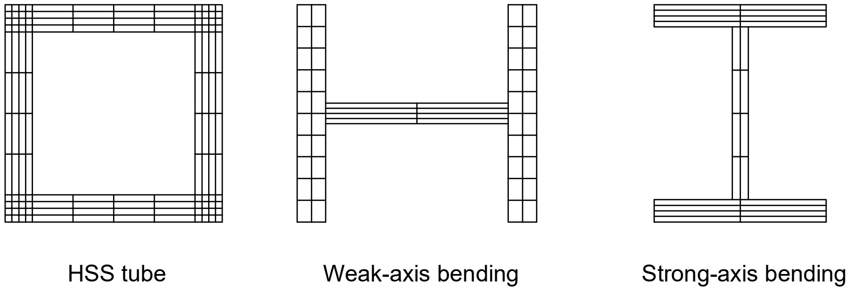


Fig. C-E1.3. Schematic layout of fibers for HSS and hot-rolled cross sections.

Analysis with Lumped Plasticity Line or Concentrated Spring Elements. Lumped plasticity elements can include nonlinear beam-column elements with lumped plasticity and/or nonlinear axial elements with force-displacement envelopes. All analyses should include nonlinear geometric effects. Force-deformation relations should be based upon engineering mechanics and should be used to represent the tensile and compressive brace behaviors, including deterioration of resistance after brace buckling. The brace response envelope should have no capacity after brace fracture. The resistance of beams, columns, and beam-column connections should be directly modeled in accordance with Sections C3, C4, C5, and C7 and with nonlinear behavior considered where appropriate. The accuracy of this method has not been documented and the user will need to verify the accuracy and reliability of this method before using it in practice.

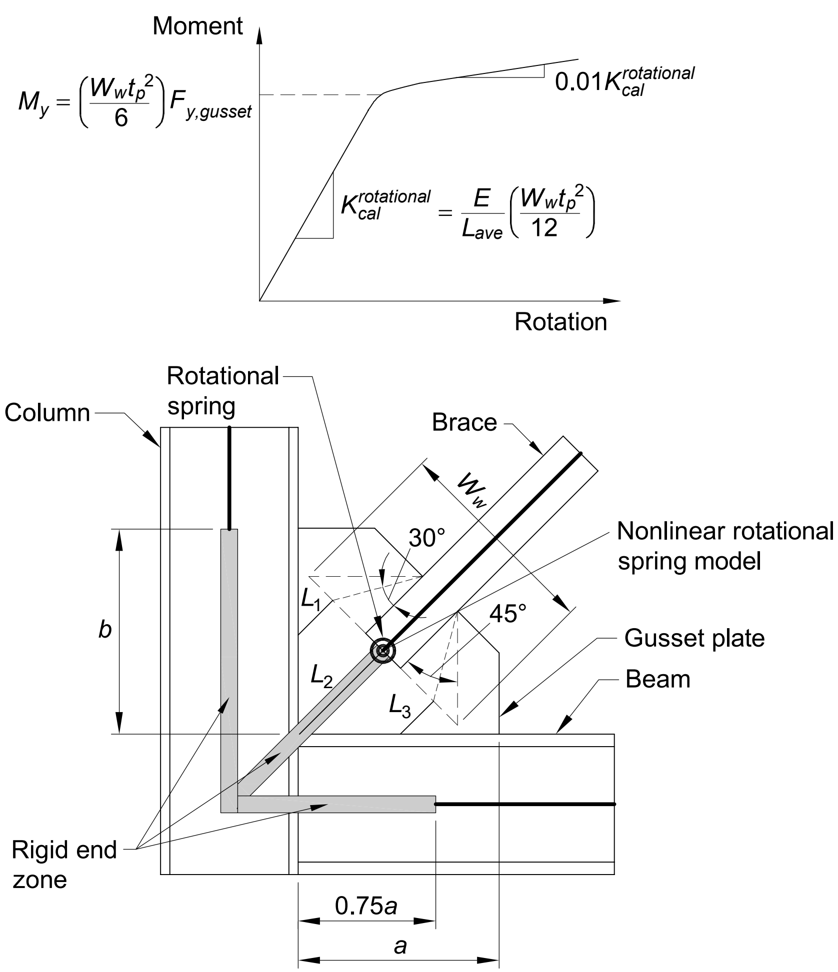


Fig. C-E1.4. Rotational connection model of corner connections.

Inelastic behavior of the column base subjected to net tension should be considered in the modeling for a potential rocking mode of the entire braced frame.

Modeling inelastic behavior of column splice connections should be considered for flexural, axial, and shear deformations based on connection details properly judged as FR or PR connections unless complete-joint-penetration groove welds are used to join columns at the splice or the splice is strengthened to the full strength of the adjacent weaker column. When test data are not available, modeling parameters and permissible performance parameters for PR moment-frame connections in Table C5.7 or Table C5.7M may be used for modeling of the splice with proper consideration of axial load effects on the reduction of flexural deformation and strength.

Compared with the braces of BRBF, the braces in CBF buckle in compression, both globally and locally, under large seismic action. This buckling may result in significant cyclic stiffness and strength degradation and in-cycle strength degradation of axial load resistance. These cyclic degradation behaviors should be modeled for braces and other components having similar behavior using the nonlinear dynamic procedure.

Strength degradation of braces in compression results in unbalanced brace loads in V-, inverted V-, and multistory X-braced frames. Beams in SCBF utilizing these bracing configurations and designed in accordance with *Seismic Provisions* (AISC 2022a) Section F2 are sized to resist the axial, shear, and moment demands associated with these unbalanced loads. Other similarly configured CBF (including ordinary, nonseismic, and pre-1997 CBF) are/were not designed using these unbalanced brace loads and, consequently, their beams are susceptible to yielding after brace buckling. CBF with yielding beams may have reduced lateral resistance because beam yielding prevents the development of the full strength of the brace in tension, but this effect also prolongs brace fracture life and yielding beams in CBF can exhibit significant ductility (Fukuta et al., 1989; Sen et al., 2016; Bradley et al., 2017; Roeder et al., 2019; Roeder et al., 2020). Therefore, nonlinear response of beams and beam connections of CBF with these bracing configurations should be considered. Yielding may occur on beam segments on either side of the midspan brace connection. Modeling axial-flexural actions of beams in inverted V-braced frames using fiber-based line elements can provide reasonably accurate nonlinear system and component response when local buckling effects in beams are not severe (Sen et al., 2019; Asada et al., 2020). For lumped plasticity modeling approaches, hinges are recommended to be located on either side of the midspan gusset plate and at the beam ends.

3. Strength

To reflect the complexity of CBF response, these provisions have new sections that provide tables and expressions to quantify the connection resistance and deformability. Section C7 allows limited yielding in some connections, because experimental research has shown that permitting yield mechanisms in the connection does not adversely affect seismic performance and in some cases may improve it. SCBF designed to requirements of *Seismic Provisions* Section F2 have connections that are designed to fully develop brace behavior, and therefore these frames only need to meet Section C3.

It is recommended that the effect of axial force on flexural strength or axial force-moment interaction in either uniaxial or biaxial bending be modeled for columns, braces, and beams that are subjected to large axial forces.

It is recommended that the effect of cyclic strength degradation caused by the cyclic nature of loading on the force–deformation capacity boundary or backbone curve at the plastic hinges be considered as prescribed in Sections C3 and C7. In lieu of derivation from experiments, the percentage reduction for strength capacity may be modeled in accordance with PEER/ATC 72-1, *Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings* (PEER, 2010), a report resulting from the Tall Building Initiative.

A multitude of research studies have been performed to better understand the behavior of CBF connections: Aviram et al. (2010), Jordan (2010), Liu and Astaneh-Asl (2000), Roeder et al. (2004, 2011b), Stoakes and Fahnestock (2010), Wijesundara et al. (2010), and Zhang et al. (2011). Connection strength and behavior have a dramatic effect on the performance of CBF, particularly in frames that do not comply with the modern detailing requirements presented in the *Seismic Provisions*. Therefore, it is recommended that connections be explicitly modeled in a proper way to simulate realistic characteristics of their full range of strengths. In lieu of derivation from tests, approved methods using engineering mechanics are permitted to model the strengths. For models where connection strength has not been explicitly considered, refer to Table C3.2 footnotes for additional reduction factors on the component permissible performance parameters.

For nonlinear dynamic procedures, the hysteretic load and deformation paths should not cross beyond the force-displacement capacity boundary or backbone curve. The characteristics of the hysteretic loops should be realistically represented in the modeling if exact cyclic degradation slopes vary for different components and are hard to predict.

FEMA 274 (FEMA, 1997b) is a useful reference for information concerning hysteretic behavior of braced frame components. Additional useful references for information regarding nonlinear load-deformation behavior of braces include those in Commentary Section E2.2b; in particular, FEMA P-440A (FEMA, 2009).

HSS braces with local slenderness exceeding high ductility requirements can be economically retrofit by filling the tube with concrete, which will delay fracture of the brace and improve the seismic performance. If this retrofit approach is used, it is important that the concrete fill does not engage or make contact with connections at each end of the brace. Under these conditions, the forces that that brace can develop are limited by force transferred by the steel brace connection. If the concrete fill engages or makes contact with the end connection of the brace, the brace will develop significantly larger forces because the end connections will permit development of the larger force. This larger force reduces the benefit of the concrete fill and often resulted in fracture of the brace at lower deformation demands.

4. Permissible Performance Parameters

4a. General

The forces computed by linear methods are limited by the computed resistance (demand-capacity ratio of 1) and the component capacity modification factors, m. The permissible deformations computed using nonlinear analysis are limited by values in the tables provided in Sections C3 and C7. For SCBF designed to meet all requirements of the *Seismic Provisions* Section F2, the values of m and deformation limits are all provided in Section C3. For all other CBF, the values of m and deformation limits are the smallest value for each given element provided in Sections C3 and C7.

4b. Linear Analysis Procedures

Beams, columns, and connections in braced frames develop axial, shear, and moment demands after brace buckling that cannot be simulated in linear analysis. A separate limit-state analysis is required to evaluate the strength of these members. Two states are to be evaluated based on the requirements of the *Seismic Provisions* Section F2.3. Most V- and inverted V-braced frames built prior to approximately 1997 do not meet these requirements and are susceptible to beam yielding after brace buckling. Recent research suggests that braced frames in these configurations with beams that develop approximately 40% of the unbalanced brace load state assuming strength degradation in compression can develop a controlled beam yielding mechanism capable of achieving required lateral strength demands and developing significant inelastic deformation capacity (Roeder et al., 2019; Roeder et al., 2020; and Asada et al., 2020). Therefore, beams in V-, inverted V-, and multistory X-braced frames can be evaluated with m up to 2.5 for both the life safety and collapse prevention performance objectives for flexural actions in combination with axial load. m is not increased for the collapse prevention performance objective because this would permit the use of weaker beams that result in unacceptable deterioration of post-brace-buckling lateral strength. Weaker beams may have substantial ductility, but nonlinear analysis is required to evaluate demand driven by system-level interactions between the beams, braces, and columns. For the immediate occupancy performance objective, $m = 1.0$ is specified to avoid yielding and inelastic beam deflections.

5. Retrofit Measures

The retrofit measures for FR moment frames described in Commentary Section D5 may be effective for braced frames. Other modifications that may be effective include replacement or modification of connections that are insufficient in strength and/or ductility and encasement of columns in concrete to improve their performance. Research has shown that the following are effective retrofit measures for CBFs.

(a) Filling a locally slender brace with concrete. The concrete fill should be separated from the gusset plate to prevent contact and an increase in brace force. Normal weight concrete should be employed unless the use of lightweight concrete is experimentally evaluated for that application.

(b) To mitigate gusset-plate-interface weld fractures, the brace should be replaced to permit brace-end rotation and/or the gusset-plate-interface welds should be strengthened by overlaying demand-critical filler metal to develop the required resistance in Section C7.

(c) For welded continuous shear plate with inadequate strength, reinforcing the shear plate with bolts can substantially improve deformation capacity and resistance.

(d) In-plane buckling retrofits can be beneficial to retain a large gusset plate. In this retrofit approach, the design considers the impact of connection rotational stiffness on buckling direction or employs a brace cross section with radii of gyration that favor IP buckling.

(e) Another retrofit option is to replace the buckling brace with a BRB. These retrofits should have a beam and column web thickness that is at least 75% of the gusset plate thickness to mitigate yielding and local deformation in the beam and column adjacent to the connection.

As noted in Section E1.1, some brace configurations are not suitable for seismic response. In these cases (such as a K-brace or knee-brace systems), modification of bracing configurations is required. In other cases, it may be advisable to convert a chevron (V-type or inverted V-type) bracing system to a two-story X-brace configuration or zipper-braced frame configuration, in particular if the beam supporting the chevron (or inverted chevron) braces is significantly undersized for the unbalanced brace forces. In addition, new steel braced frames added for retrofit purposes (i.e., adding new CBFs into an existing system) should be modeled and evaluated per the requirements of this standard and should satisfy modern detailing requirements set forth in the *Seismic Provisions*. FEMA 547 (FEMA, 2007) contains useful information pertaining to the retrofit of existing buildings.

Modification of bracing configurations (i.e., converting V-type or inverted V-type bracing to two-story X-brace configuration) may be beneficial for improved seismic performance (Yoo et al., 2009; Yang et al., 2008), however, changes to these provisions relax many of the difficulties currently encountered with chevron of V-braced frames.

New steel braced frames added for retrofit purposes (i.e., adding new CBF into an existing system) should be modeled and evaluated per the requirements of ASCE/SEI 41 and these Provisions and should satisfy modern detailing requirements set forth in the *Seismic Provisions*. FEMA 547 (FEMA, 2007) contains useful information pertaining to the retrofit of existing buildings. Additional references discussing the retrofit of CBF include Rai and Goel (2003), Di Sarno et al. (2006), and Roeder et al. (2009b).

E2. ECCENTRICALLY BRACED FRAMES (EBF)

2. Stiffness

2c. Nonlinear Dynamic Procedure

FEMA 274 (FEMA, 1997b) is a useful reference for guidelines on modeling the link beams and information regarding the hysteretic behavior of eccentrically braced frame (EBF) components.

The elastic shear stiffness, K_e, of the link beam may be determined from Equation C-E2-1, unless justified otherwise by analysis.

$$K_e = \frac{12EI}{L_v^3(1+\eta)} \tag{C-E2-1}$$

where

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

I = moment of inertia about the axis of bending, in.^{4} (mm^4)

L_v = clear length between supports that resist translation in the direction of the shear force, in. (mm)

$$\eta = \frac{12EI}{L_v^2 GA_s} \tag{C-E2-2}$$

A_s = effective shear area of the cross section, in.^{2} (mm^2)
(for a wide-flange shape in strong-axis bending, $A_s = d_b t_w$)

d_b = depth of beam, in. (mm)

t_w = thickness of web, in. (mm)

G = shear modulus of elasticity of steel
= 11,200 ksi (77 200 MPa)

3. Strength

Equation C3-18 includes axial load effects. Where required for linear procedures, each action capacity, P_{ye} and $0.6F_{ye}A_s$, should be multiplied by the knowledge factor, κ, where P_{ye} is the expected axial yield strength and F_{ye} is the expected yield stress. The resulting value of the interaction equations κQ_{CE}, where Q_{CE} is the expected component strength. This value provides direct incorporation into ASCE/SEI 41 Equation 7-36.

4. Permissible Performance Parameters

The permissible performance parameters for complete-joint-penetration groove-welded beam-to-column connections are based on testing of typical moment-frame proportioning and span ratios.

5. Retrofit Measures

The retrofit measures described in Commentary Section D5 for moment frames with FR connections and in Commentary Section E1 for concentrically braced frames may be effective for many beams, columns and braces. Cover plates and/or stiffeners may be effective in retrofitting these components. The strength of the link may be increased by adding cover plates to the beam flanges, adding doubler plates or stiffeners to the web, or changing the brace configuration.

E3. BUCKLING-RESTRAINED BRACED FRAMES (BRBF)

1. General

Buckling-restrained braces (BRB) are expected to withstand significant inelastic deformations without strength or stiffness degradation when subjected to earthquake loading. It is recommended that evaluation of buckling-restrained braced frames (BRBF) consider the rotational stiffness and deformation limitations of the gusset plate connections in series with the BRB elements. This limitation would mean that a typical bay would have beams, columns, BRB elements, and fully restrained (FR) or partially restrained (PR) moment-frame connections modeled at the end of the braces. Section E3 focuses on the modeling and permissible performance parameters of the BRB elements; refer to Chapter D and Section E1 for moment-frame and concentrically braced frame provisions, respectively.

2. Stiffness

2b. Nonlinear Static Procedure

Item (c) specifies that the residual strength beyond modeling parameter b may be reduced to near zero in recognition of the fact that the stiffness cannot be set to zero in the analytical model because of limitations of typical analytical software.

3. Strength

The compressive overstrength arises because of friction and confinement that are caused by the interaction of the core and the casing system.

5. Retrofit Measures

Potential retrofit measures for existing BRBF components would be to add additional seismic force-resisting elements to reduce the demand on the existing BRBF system or to replace the BRB element. As the BRBF system is a rather new system, an example of where this may be needed would be in upgrading an existing building to a higher performance level than was originally intended, for example, from life safety to immediate occupancy.

E4. STEEL PLATE SHEAR WALLS

1. General

A steel plate shear wall system develops its seismic resistance through shear stress in the wall. Although structures with steel plate shear walls are not common, they have been used to retrofit a few essential structures where immediate occupancy and operational performance levels are required after a large earthquake. Because of their stiffness, the steel plate shear walls attract much of the seismic shear. It is essential that the new load paths be carefully established.

The provisions for steel plate walls in ASCE/SEI 41 (ASCE, 2017) and in these Provisions assume that the steel plates are sufficiently stiffened to prevent buckling. The design professional is referred to Timler (2000) and the *Seismic Provisions* (AISC, 2022a) for additional information regarding the behavior and design of steel plate shear walls.

2. Stiffness

2c. Nonlinear Dynamic Procedure

This procedure is not recommended in most cases.

CHAPTER F

STRUCTURAL STEEL FRAMES WITH INFILLS

F1. GENERAL

In many cases, infill walls are unreinforced or lightly reinforced, and their strength and ductility may be inadequate. Before the loss of the wall, the steel frame adds confining pressure to the wall and enhances its resistance. The actual effective forces on the steel frame components, however, are probably minimal. As the frame components attempt to develop force, they deform and the stiffer concrete or masonry components on the far side of the member pick up load. However, beam end connections, column splices, and steel frame connections at the foundation should be investigated for forces caused by interaction with the infill as in procedures specified for concrete frames in ASCE/SEI 41, Chapter 10 (ASCE, 2017).

The stiffness and resistance provided by concrete and/or masonry infills may be much larger than the stiffness of the steel frame acting alone with or without composite actions. Gaps or incomplete contact between the steel frame and the infill may negate some or all of this stiffness. These gaps may be between the wall and columns of the frame or between the wall and the top beam enclosing the frame. Different strength and stiffness conditions result from different discontinuity types and locations. Therefore, the presence of any gaps or discontinuities between the infill walls and the frame are first determined and then subsequently considered in the design and retrofit process. The resistance provided by infill walls may also be included if proper evaluation of the connection and interaction between the wall and the frame is made and if the strength, ductility, and properties of the wall are properly included.

The stiffness provided by infill masonry walls is excluded from the design and retrofit process unless integral action between the steel frame and the wall is verified. If complete or partial interaction between the wall and frame is verified, the stiffness is increased accordingly. The seismic performance of unconfined masonry walls is far inferior to that of confined masonry walls; therefore, the resistance of the attached wall can be used only if strong evidence as to its strength, ductility, and interaction with the steel frame is provided.

CHAPTER G
DIAPHRAGMS

The steel deck diaphragm provisions provided for the 2020 edition of these Provisions represent an expansion from available steel deck diaphragm provisions in ASCE/SEI 41 (ASCE, 2017). In previous editions, if connections controlled the strength of a steel deck diaphragm, then the diaphragm had to be considered force-controlled. As a result, because of the form of current strength calculations, essentially all steel deck diaphragms, with or without concrete fill, had to be considered force-controlled in ASCE/SEI 41. This is not consistent with the available experimental data, which demonstrates that ductility exists in these diaphragm systems. To that end, all existing experimental data was gathered and assessed to provide acceptance criteria and nonlinear modeling parameters for steel deck diaphragms, as detailed in Wei et al. (2019). Provisions were developed such that the engineer could consider steel deck diaphragms as either deformation-controlled or force-controlled—depending on whether ductility in the diaphragm is being considered.

Reliable provisions exist for determining the stiffness and strength of bare steel deck diaphragms and steel deck diaphragms with concrete fill in AISI *North American Specification for the Design of Cold-Formed Steel Structural Members*, ANSI/AISI S310 (AISI, 2020a). However, the current scope of ANSI/AISI S310 is limited to concrete fill with only temperature and shrinkage reinforcing steel and headed steel shear studs. If the steel reinforcement in the concrete fill is intended to provide an elevated shear capacity, or provide performance as a chord or collector, then ANSI/AISI S310 is silent and other standards provide only limited guidance. As a result, these provisions have been separated into three cases: Section G1 for bare steel deck diaphragms, Section G2 for steel deck diaphragms with reinforced concrete structural topping that are outside the scope of the stiffness and strength provisions of ANSI/AISI S310, and Section G3 for steel deck diaphragms with unreinforced structural topping and nonstructural topping, which are within scope for ANSI/AISI S310. The provisions for Section G2 follow the design philosophy of the *Seismic Provisions* (AISC, 2022a). Research and standardization work is underway to increase the scope of ANSI/AISI S310, therefore it is anticipated that the provisions of Sections G2 and G3 will evolve in the future.

G1. BARE STEEL DECK DIAPHRAGMS

1. General

Bare steel deck diaphragms are usually used for roofs of buildings where there are very light gravity loads other than support of roofing materials. Load transfer to frame elements that act as chords or collectors in modern frames is through arc spot or arc seam welds, screws, or power-actuated fasteners. Load transfer between deck sheets in modern frames is through screws or arc spot welds in nestable deck, or through proprietary clinching, top arc seam welds, or button punching, in interlocking deck.

Additionally, these provisions could be extrapolated to roof deck that is similar to steel deck, that is, through fastened roof deck.

2. Stiffness

Provisions for calculating stiffness are available in ANSI/AISI S310 (AISI, 2020a). Tabulated stiffness values may be found from the SDI *Diaphragm Design Manual* (SDI, 2015) or from manufacturer catalogs. A database of tested bare deck diaphragms has also been assembled and may be used for determining stiffness (SDII database site).

3. Strength

Provisions for finding the nominal strength of bare steel deck diaphragms are available in ANSI/AISI S310 (AISI, 2020a). Provisions are provided for limit states associated with the connectors and those associated with the panel. Connector limit states are more common in conventional configurations. Tabulated strength values may be found from the *Diaphragm Design Manual* (SDI, 2015) or from manufacturer catalogs. Appropriate nominal connection capacity calculations are embedded within the ANSI/AISI S310 provisions and generally rely on the connection provisions of AISI *North American Standard for the Design of Cold-Formed Steel Structural Members*, ANSI/AISI S100/S2-20 (AISI, 2020b).

The mean strength for the generalized force-deformation response of bare steel deck diaphragms was established by equating the energy under an elastic-perfectly plastic model up to the deformation consistent with 80% post-peak capacity with the actual tested force-deformation response in a cantilever diaphragm test, as detailed in Wei et al. (2019). The expected strength (mean resistance) was determined by comparing the established mean strength with the provisions of ANSI/AISI S310 (Wei et al., 2019). The lower-bound (mean minus one standard deviation) strength was determined in a similar manner, with judgment applied when the data was sparse.

4. Permissible Performance Parameters

Prior to 2020, connection-based limit states were considered as force-controlled only. In 2020, based on an evaluation of the ductility and hysteretic response of bare steel deck diaphragms in full-scale cantilever tests, this position was updated and permissible performance parameters were provided to allow engineers to treat these limit states as deformation-controlled (Wei et al., 2019). The permissible performance parameters and modeling parameters are broken down by limit state and the connector configuration. In most cases, available cantilever testing did not cycle far enough to provide a reliable prediction of the residual strength, modeling parameter c. Connector testing under large cycles and judgment was used to develop these final values—in general, power-actuated fasteners provide substantial residual capacity while welds do not. See Wei et al. (2019) for the complete development.

The component capacity modification factors, m, for panel buckling are based on ASCE/SEI 41 and reflect some engineering judgment as this mode of failure is not common in testing. Modeling parameters for the panel buckling limit state are not provided as it is not a common mode of deformation; however, it is possible to develop them from engineering mechanics.

The fastener spacing limits, provided in Tables G1.1 and G1.2, were initially incorporated into the initial edition of the SDI *Diaphragm Design Manual* (DDM01)

(SDI, 1981) and all subsequent editions. Additionally, this limit is also found in the Steel Deck Institute Standards for roof deck (ANSI/SDI RD) and floor deck (ANSI/SDI NC and ANSI/SDI C) (SDI, 2017a, 2017b, 2017c). The limit in ANSI/AISI S310 (AISI, 2020a) and DDM01 (SDI, 1981) through DDM04 (SDI, 2015), and the SDI Standards is based on practical deck installation limitations. This limit prevents having a deck side seam from having an unconnected length of over 5 ft (1.5 m), thus limiting the relative vertical movement of an unconnected side seam when an installer might stand on one deck panel and not the adjacent panel. The majority of the initial testing by Luttrell, as found in DDM01, was conducted without any side-lap fasteners [for welded diaphragms, 89 of 107 assemblies had no side-lap attachment on spans up to 6 ft 8 in. (2 m)]. This testing, along with subsequent testing, validates the analytic method used for calculating diaphragm strength and stiffness that forms the basis of ANSI/AISI S310. This testing validates the ability to predict the performance of diaphragms with various numbers of side-lap fasteners, from highly connected to no side-lap connections.

5. Retrofit Measures

The stiffness and strength provisions of ANSI/AISI S310 (AISI, 2020a) are sensitive to deck profile and gauge, structural fastener type and spacing, and side-lap fastener type and spacing. All of these are potential parameters to be considered in a retrofit. The following measures may be effective in retrofitting bare steel deck diaphragms:

(a) Adding steel headed stud anchors for transfer of load to chord or collector elements;

(b) Strengthening existing chords or collectors by the addition of new steel plates to existing frame components;

(c) Adding puddle welds or other shear connectors at panel perimeters;

(d) Adding diagonal steel bracing to form a horizontal truss to supplement diaphragm strength;

(e) Adding structural concrete; and

(f) Adding connections between deck and supporting members.

G2. STEEL DECK DIAPHRAGMS WITH REINFORCED CONCRETE STRUCTURAL TOPPING

1. General

Steel deck diaphragms with reinforced structural concrete topping are used on floors and roofs of buildings where there are substantial gravity loads and significant shear demands that require reinforcing in the structural concrete topping beyond that for temperature and shrinkage steel. ANSI/AISI S310 (AISI, 2020a) and Section G3 provide solutions for lightly reinforced (welded wire fabric or only temperature and shrinkage steel reinforcing) and plain structural concrete topping; this section applies to fully reinforced (composite) slabs. Overall, the approach that is adopted, consistent with *Seismic Provisions* Section D1.5, is to treat the reinforced concrete above the top of the deck flute as a reinforced slab and use ACI 318 (alternatively testing is also permitted). This approach is also adopted for cast-in-place concrete diaphragms in ASCE/SEI 41, Chapter 10.

3. Strength

In addition to considering the strength of the reinforced concrete slab above the deck in shear, the strength of the composite steel headed stud or steel channel anchors are also considered. Engineers are directed to *Specification* Chapter I for this calculation.

4. Permissible Performance Parameters

All permissible performance parameters and modeling parameters are aligned with ASCE/SEI 41, Chapter 10, for cast-in-place concrete diaphragms. In turn, these provisions are based on reinforced concrete shear walls.

5. Retrofit Measures

See the commentary discussion in ASCE/SEI 41, Chapter 10, and Commentary Section G3.

G3. STEEL DECK DIAPHRAGMS WITH UNREINFORCED STRUCTURAL CONCRETE TOPPING OR LIGHTWEIGHT INSULATING CONCRETE

1. General

Steel deck diaphragms with structural concrete topping are frequently used on floors and roofs of buildings where there are typical floor gravity loads. Concrete has structural properties that significantly add to diaphragm stiffness and strength. Concrete reinforcing is minimal, ranging from light welded wire reinforcement grids to a regular grid of small reinforcing bars (size No. 3 or No. 4). Plain concrete is also acceptable. Steel decking is typically composed of corrugated sheet steel from 22 gauge down to 14 gauge. Rib depths vary from 1½ to 3 in. (38 to 75 mm) in most cases. Attachment of the steel deck to the steel frame is usually accomplished using arc spot or arc seam welds at 1 to 2 ft (0.3 to 0.6 m) on center. For partially composite behavior, steel headed stud anchors are welded to the frame before the concrete is cast.

Steel deck diaphragms with nonstructural fill are typically used on roofs of buildings where there are small gravity loads. The fill, such as lightweight insulating concrete (e.g., vermiculite), usually does not have usable structural properties and is most often unreinforced. Consideration of any composite action is undertaken with caution and only after extensive investigation of field conditions. Material properties, force transfer mechanisms, and other similar factors are verified where such composite action is relied upon.

Load transfer to frame elements that act as chords or collectors in modern frames is usually through puddle welds or headed studs. In older construction where the frame is encased for fire protection, load transfer is made through the concrete-to-steel bond.

2. Stiffness

Provisions for calculating stiffness are available in ANSI/AISI S310 (AISI, 2020a). Tabulated stiffness values may be found from the SDI *Diaphragm Design Manual* (SDI, 2015) or from manufacturer catalogs. A small database of tested filled deck diaphragms has also been assembled and may be used for determining stiffness (O'Brien et al., 2017a, 2017b).

3. Strength

Provisions for finding the nominal strength of filled steel deck diaphragms are available in ANSI/AISI S310 (AISI, 2020a). Tabulated strength values may be found from the *Diaphragm Design Manual* (SDI, 2015) or from manufacturer catalogs. The mean strength for the generalized force-deformation response of steel deck diaphragms with concrete fill was established by equating the energy under an elastic-perfectly plastic model up to the deformation consistent with 80% post-peak capacity with the actual tested force-deformation response in a cantilever diaphragm test, as detailed in Wei et al. (2019). The expected strength (mean resistance) was determined by comparing the established mean strength with the provisions of ANSI/AISI S310 (Wei et al., 2019). The lower-bound (mean minus one standard deviation) strength was determined in a similar manner, with judgment applied when the data was sparse.

4. Permissible Performance Parameters

Prior to 2020, filled decks were considered to be force-controlled only. In 2020, based on an evaluation of the ductility and hysteretic response of filled steel deck diaphragms in full-scale cantilever tests, this position was updated and permissible performance parameters were provided to allow engineers to treat these limit states as deformation-controlled (Wei et al., 2019). Cracking in the concrete fill initiates at small strains and thus the immediate occupancy (IO) permissible performance parameters are relatively small. Additionally, relatively large values of m and permissible performance parameters should be understood in the context of these small strains.

5. Retrofit Measures

The following measures may be effective in retrofitting steel deck diaphragms with structural concrete topping:

(a) Adding steel headed stud anchors to transfer forces to chord or collector elements;

(b) Strengthening existing chords or collectors by the addition of new steel plates to existing frame components or attaching new plates directly to the slab by embedded bolts or epoxy; and

(c) Adding diagonal steel bracing to supplement diaphragm strength.

The following measures may be effective in retrofitting steel deck diaphragms with nonstructural topping:

(1) Adding steel headed stud anchors to transfer forces to chord or collector elements;

(2) Strengthening existing chords or collectors by the addition of new steel plates to existing frame components or attaching new plates directly to the slab by embedded bolts or epoxy;

(3) Adding puddle welds at panel perimeters of diaphragms;

(4) Adding diagonal steel bracing to supplement diaphragm strength; and

(5) Replacing nonstructural fill with structural concrete.

G4. HORIZONTAL STEEL TRUSS DIAPHRAGMS

1. General

Steel truss diaphragm elements are typically found in conjunction with vertical framing systems that are of structural steel framing. Steel truss diaphragms are more common in long-span situations, such as special roof structures for arenas, exposition halls, auditoriums, and industrial buildings. Diaphragms with large span-to-depth ratios may often be stiffened by the addition of steel trusses. The addition of steel trusses for diaphragms identified to be deficient may provide a proper method of enhancement.

Steel truss diaphragms may be made up of any of the various structural shapes. Often, the truss chord elements consist of wide-flange shapes that also function as floor beams to support the gravity loads of the floor. For lightly loaded conditions, such as industrial steel deck roofs without concrete fill, the diagonal members may consist of threaded rod elements, which are assumed to act only in tension. For steel truss diaphragms with large loads, diagonal elements may consist of wide-flange members, hollow structural sections, or other structural elements that act in both tension and compression. Truss element connections are generally concentric to provide the maximum lateral stiffness and ensure that the truss members act under pure axial load. These connections are generally similar to those of gravity load-resisting trusses.

5. Retrofit Measures

The following measures may be effective in retrofitting steel truss diaphragms:

(a) Diagonal components may be added to form additional horizontal trusses as a method of strengthening a weak existing diaphragm;

(b) Existing chord components may be strengthened by the addition of steel headed stud anchors to enhance composite action;

(c) Existing steel truss components may be strengthened by methods specified for braced steel frame members;

(d) Truss connections may be strengthened by the addition of welds, new or enhanced plates, and bolts; and

(e) Structural concrete fill may be added to act in combination with steel truss diaphragms after verifying the effects of the added weight of concrete fill.

G5. ARCHAIC DIAPHRAGMS—SHALLOW BRICK ARCHES SPANNING BETWEEN STRUCTURAL STEEL FLOOR BEAMS

1. General

Archaic steel diaphragm elements are almost always found in older steel buildings in conjunction with vertical systems of structural steel framing. The masonry arches were typically covered with a very low strength concrete fill, usually unreinforced. In many instances, various archaic diaphragm systems were patented by contractors.

2. Stiffness

2b. Nonlinear Analysis Procedures

Inelastic properties of archaic diaphragms should be chosen with caution for seismic analyses. For the case of archaic diaphragms, inelastic models similar to those of archaic timber diaphragms in unreinforced masonry buildings may be appropriate. Inelastic deformation limits of archaic diaphragms should be lower than those prescribed for a concrete-filled diaphragm.

5. Retrofit Measures

The following measures may be effective in retrofitting archaic diaphragms:

(a) Adding diagonal members to form a horizontal truss;

(b) Strengthening existing steel members by adding steel headed stud anchors to enhance composite action; and

(c) Removing weak concrete fill and replacing it with a structural concrete topping slab after verifying the effects of the added weight of concrete fill.

G6. CHORD AND COLLECTOR ELEMENTS

1. General

Where reinforcing acts as the chord or collector, load transfer occurs through bond between the reinforcing bars and the concrete.

5. Retrofit Measures

The following measures may be effective in retrofitting chord and collector elements:

(a) Strengthen the connection between diaphragms and chords or collectors;

(b) Strengthen steel chords or collectors with steel plates attached directly to the slab with embedded bolts or epoxy, and strengthen slab chords or collectors with added reinforcing bars; and

(c) Add chord members.

CHAPTER H
STRUCTURAL STEEL PILE FOUNDATIONS

H1. GENERAL

Steel piles of wide-flange shape (H-piles) or hollow structural sections or pipes, with and without concrete infills, can be used to support foundation loads. Piles driven in groups should have a pile cap to transfer loads from the superstructure to the piles.

In poor soils or soils subject to liquefaction, bending of the piles may be the only dependable resistance to lateral loads.

H4. PERMISSIBLE PERFORMANCE PARAMETERS

Nonlinear methods require the use of specialized software for determining actions on the piles. FEMA 274 (FEMA, 1997b) is a useful reference for additional information.

H5. RETROFIT MEASURES

Retrofit measures for concrete pile caps are specified in ASCE/SEI 41 (ASCE, 2017), Chapter 10. Criteria for the retrofit of foundation elements are specified in ASCE/SEI 41, Chapter 8. One method that may be effective in retrofitting steel pile foundations consists of driving additional piles near existing groups and then adding a new pile cap to increase stiffness and strength of the pile foundation. Monolithic behavior gained by connecting the new and old pile caps with epoxied dowels may also be effective. In most cases, it is not possible to retrofit the existing piles.

CHAPTER I

CAST AND WROUGHT IRON

I1. GENERAL

Historical gray cast iron is a very hard and brittle material with a high carbon content that can resist compression forces very well but can be highly susceptible to brittle tensile failures at forces that are a fraction of its compressive strength. The stress-strain relationship for historical cast iron lacks a distinct yield point in either tension or compression. Historical wrought iron is capable of developing yield strength and ductility in tension, although through-thickness tensile properties of wrought iron are noticeably lower than its tensile properties in the longitudinal (rolling) direction. Because of these unique characteristics, these two historical structural metals are addressed separately from structural steel in these Provisions. Procedures for field-identification of these two particular historical structural metals, and for distinguishing them from structural steel, are beyond the scope of these Provisions.

Cast iron in a structural system was used primarily in compression, mostly for columns in framing systems and occasionally as compression elements acting compositely with wrought iron tension elements in built-up beams. The earliest all-metal framing systems used wrought iron beams supported by cast iron columns. All-metal framing systems evolved over time into systems constructed entirely of wrought iron, and then to systems constructed entirely of structural steel (Paulson, 2013; Brockenbrough and Schuster, 2018).

The limitation on use of historical cast iron and historical wrought iron materials to framing components in combination with concrete or masonry walls arises from the unreliable nature of historical cast iron in tension and the lesser tensile properties of historical wrought iron in its through-thickness direction. These concerns led to a desire to limit deformation demands on components composed of these historical metals, and it is felt that this can be encouraged by limiting the use of these historical metals only to components in structures that also employ relatively stiff walls in an effort to control seismic deformation demands.

I3. STRENGTH

1. Cast Iron

The type of cast iron covered by Section I3.1 is historical gray cast iron as manufactured during the 1800s and early 1900s and as primarily used for columns. Historical gray cast iron lacks reliable tensile strength (Paulson, 2013) and, as a result, cast iron columns should not be used where flexural or axial actions, either alone or in combination, may result in net tensile stresses in the cast iron. Again, because of the lack of reliable tensile strength in historical cast iron materials, cast iron beams are not permitted to resist any seismic actions; cast iron beams, however, are rarely found in existing buildings in the United States, although they might be encountered in buildings predating the 1870s. Detailed structural assessment of cast iron members in general is beyond the scope of these Provisions.

The formula provided in these Provisions for determination of lower-bound compression strength of a cast iron column is adapted from a formula given in Paulson et al. (1996), which provides an analysis of historical compression tests from the 1880s and 1890s on full-size cast iron columns.

2. Wrought Iron

The type of wrought iron covered by Section I3.2 is historical wrought iron as manufactured between approximately the 1860s and the 1920s. Historical wrought iron is usually capable of achieving yield in tension and can develop a reliable tensile strength and post-yield ductility. However, tensile properties of historical wrought iron in the through-thickness direction (perpendicular to rolling direction) are lower than the tensile properties in the longitudinal (rolling) direction because of the slag that is inherent in historical wrought iron (Paulson, 2013). This may lead to concerns when assessing wrought iron connecting elements and requires careful consideration when welding historical wrought iron. Detailed structural assessment of historical wrought iron members in general is beyond the scope of these Provisions.

I4. PERMISSIBLE PERFORMANCE PARAMETERS

1. Cast Iron

In regard to resisting the deformations at the selected seismic hazard level, net tensile stresses from direct axial tension and axial-flexure interaction should not develop in the cast iron component. This is because historical cast iron members do not reliably resist tensile stresses (Paulson, 2013). The performance of historical cast iron under cyclic tensile stresses is not well documented but is believed to be extremely poor because of the metallurgical nature of historical cast iron.

2. Wrought Iron

In contrast to cast iron, wrought iron components can sustain tensile stresses and achieve yield. However, inelastic cyclic performance of wrought iron components is not well documented and is believed to be potentially poor, primarily because its through-thickness tensile properties are noticeably lower than its longitudinal tensile properties. As a result, wrought iron components are classified as force-controlled components in accordance with ASCE/SEI 41, Section 7.5.1.2.

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