Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings

DRAFT dated September 30, 2019

(Not yet) Approved by the AISC Committee on Specifications

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
130 East Randolph Street, Suite 2000, Chicago, Illinois 60601
www.aisc.org
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 by ASCE/SEI 41, these Provisions are intended to be used in conjunction with the Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341.

The 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. It is intended that ASCE/SEI 41 adopt these Provisions to replace 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 contributions and assistance of more than 50 additional professional volunteers working in task committees are also hereby acknowledged.

The Symbols, Glossary, Abbreviations to these Provisions are an integral part of the Provisions. The Symbols, Glossary, Abbreviations are consistent with those used in ASCE/SEI 41 for ease of adoption by ASCE/SEI 41, and for ease of use with ASCE/SEI 41. 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.

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

ii
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.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_b$</td>
<td>Gross area of rivet or bolt, in.$^2$ (mm$^2$)</td>
<td>C5.3a.42(a)(i)</td>
</tr>
<tr>
<td>$A_{cf}$</td>
<td>Area of column flange, in.$^2$ (mm$^2$)</td>
<td>C4.4a.2</td>
</tr>
<tr>
<td>$A_{conn}$</td>
<td>Cross-sectional area of BRB connection, in.$^2$ (mm$^2$)</td>
<td>C3.4a.1.b</td>
</tr>
<tr>
<td>$A_{core}$</td>
<td>Cross-sectional area of BRB core, in.$^2$ (mm$^2$)</td>
<td>C3.4a.1.b</td>
</tr>
<tr>
<td>$A_e$</td>
<td>Effective net area of horizontal angle leg, in.$^2$ (mm$^2$)</td>
<td>C5.3a.42(a)(ii)</td>
</tr>
<tr>
<td>$A_e$</td>
<td>Effective net area of split-tee stem, in.$^2$ (mm$^2$)</td>
<td>C5.3a.42(b)(iii)</td>
</tr>
<tr>
<td>$A_e$</td>
<td>Effective net area of flange plate, in.$^2$ (mm$^2$)</td>
<td>C5.3a.42(c)</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross area of the cross section, in.$^2$ (mm$^2$)</td>
<td>C3.2b</td>
</tr>
<tr>
<td>$A_{flang}$</td>
<td>Gross area of flange plate, in.$^2$ (mm$^2$)</td>
<td>C5.3a.2(c)</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross area of gusset plate, in.$^2$ (mm$^2$)</td>
<td>C7.2b</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross area of horizontal angle leg, in.$^2$ (mm$^2$)</td>
<td>C5.3a.42(a)(ii)</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross area of smaller member, in.$^2$ (mm$^2$)</td>
<td>C5.3b.3(b)</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross area of split-tee stem, in.$^2$ (mm$^2$)</td>
<td>C5.3a.42(b)</td>
</tr>
<tr>
<td>$A_{flang}$</td>
<td>Gross area of flange plate, in.$^2$ (mm$^2$)</td>
<td>C5.3a.1(c)</td>
</tr>
<tr>
<td>$A_{flang}$</td>
<td>Gross area of smaller member, in.$^2$ (mm$^2$)</td>
<td>C5.3b.4</td>
</tr>
<tr>
<td>$A_{flang}$</td>
<td>Gross cross sectional area of plate with Whitmore width using 30° projection, in.$^2$ (mm$^2$)</td>
<td>C7.2b</td>
</tr>
<tr>
<td>$A_{flang}$</td>
<td>Gross cross sectional area of plate with Whitmore width using 30° projection, in.$^2$ (mm$^2$)</td>
<td>C7.3b.4</td>
</tr>
<tr>
<td>$A_{nt}$</td>
<td>Net area subject to tension, in.$^2$ (mm$^2$)</td>
<td>C7.3b.3</td>
</tr>
<tr>
<td>$A_{nt}$</td>
<td>Net area subject to shear, in.$^2$ (mm$^2$)</td>
<td>C7.3b.3</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Effective shear area of the cross section, in.$^2$ (mm$^2$)</td>
<td>C2.2b4a.1.b</td>
</tr>
<tr>
<td>$B_w$</td>
<td>Whitmore width using 30° projection, effective gusset plate width, in. (mm)</td>
<td>C7.2b</td>
</tr>
<tr>
<td>$B_w$</td>
<td>Whitmore width using 32° projection, in. (mm)</td>
<td>C7.3b.4</td>
</tr>
<tr>
<td>$E$</td>
<td>Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)</td>
<td>C2.2b4a.1.b</td>
</tr>
<tr>
<td>$E_{ci}$</td>
<td>Modulus of elasticity of cast iron = 15,000 ksi (100 000 MPa)</td>
<td>I2</td>
</tr>
<tr>
<td>$E_{ci}$</td>
<td>Modulus of elasticity of wrought iron = 25,000 ksi (170 000 MPa)</td>
<td>I2</td>
</tr>
<tr>
<td>$E_{ci}$</td>
<td>Modulus of elasticity of cast iron = 45 200 000 ksi (340 000 MPa)</td>
<td>I3.1</td>
</tr>
<tr>
<td>$E_{fl}$</td>
<td>Flexural stiffness of a beam with partially restrained connections, kip in.$^2$ (N mm$^2$)</td>
<td>C5.2a.2</td>
</tr>
<tr>
<td>$E_{fl}$</td>
<td>Flexural stiffness of a column or brace, kip in.$^2$ (N mm$^2$)</td>
<td>C3.2b</td>
</tr>
<tr>
<td>$F_{cr}$</td>
<td>Critical stress, ksi (MPa)</td>
<td>I3</td>
</tr>
<tr>
<td>$F_{cr,LB}$</td>
<td>Critical stress of the plate computed using $F_{cr,LB}$, ksi (MPa)</td>
<td>C7.3b.4</td>
</tr>
<tr>
<td>$F_e$</td>
<td>Elastic buckling stress determined according to Equation I3-4, ksi (MPa)</td>
<td>I3.1</td>
</tr>
<tr>
<td>$F_{ESR}$</td>
<td>Weld filler metal classification strength, ksi (MPa)</td>
<td>C7.3a.4</td>
</tr>
<tr>
<td>$F_{nv}$</td>
<td>Nominal shear stress for bearing-type connections, given in Specification Section I3.6, ksi (MPa)</td>
<td>C5.3a.42(a)(i)</td>
</tr>
<tr>
<td>$F_{nv}$</td>
<td>Nominal shear stress for weld metal, given in Specification Section I2, ksi (MPa)</td>
<td>C5.3a.42(c)</td>
</tr>
<tr>
<td>$F_e$</td>
<td>Expected tensile strength of bolt or rivet, taken as $F_{nv}$</td>
<td>C5.3a.42(a)(ii)</td>
</tr>
<tr>
<td>$F_{nv}$</td>
<td>Nominal shear stress for bearing-type connections, given in Specification Section I3 ksi (MPa)</td>
<td>C5.3a.42(a)(iii)</td>
</tr>
<tr>
<td>$F_{g}$</td>
<td>Specified minimum tensile strength</td>
<td>C7.3b.2</td>
</tr>
<tr>
<td>$F_{ue}$</td>
<td>Expected tensile strength, ksi (MPa)</td>
<td>A5.2a</td>
</tr>
<tr>
<td>Expression</td>
<td>Definition</td>
<td></td>
</tr>
<tr>
<td>------------------------------------------------</td>
<td>-----------------------------------------------------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>$F_{ul,B}$</td>
<td>Lower-bound tensile strength, ksi (MPa)</td>
<td></td>
</tr>
<tr>
<td>$F_{we}$</td>
<td>Expected shear strength of bolt or rivet, taken as $E_{w}$/2</td>
<td></td>
</tr>
<tr>
<td>$F_y$</td>
<td>Specified minimum yield stress, ksi (MPa)</td>
<td></td>
</tr>
<tr>
<td>$F_{ye}$</td>
<td>Expected yield stress, ksi (MPa)</td>
<td></td>
</tr>
<tr>
<td>$G_{ul,B}$</td>
<td>Lower-bound yield stress, ksi (MPa)</td>
<td></td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)</td>
<td></td>
</tr>
<tr>
<td>$I$</td>
<td>Moment of inertia about the axis of bending, in.$^4$ (mm$^4$)</td>
<td></td>
</tr>
<tr>
<td>$I_b$</td>
<td>Moment of inertia of beam about the axis of bending, in.$^4$ (mm$^4$)</td>
<td></td>
</tr>
<tr>
<td>$L$</td>
<td>Moment of inertia of a column or brace about the axis of bending, in.$^4$ (mm$^4$)</td>
<td></td>
</tr>
<tr>
<td>$K$</td>
<td>Effective length factor</td>
<td></td>
</tr>
<tr>
<td>$K_{IC}$</td>
<td>Fracture toughness parameter per Table C5.4-2</td>
<td></td>
</tr>
<tr>
<td>$K_e$</td>
<td>Elastic shear stiffness of the beam, kip/in. (N/mm)</td>
<td></td>
</tr>
<tr>
<td>$K_w$</td>
<td>Elastic shear stiffness of a stiffened plate wall, kip/in. (N/mm)</td>
<td></td>
</tr>
<tr>
<td>$K_u$</td>
<td>Rotational stiffness, kip-in./rad (N-mm/rad)</td>
<td></td>
</tr>
<tr>
<td>$L$</td>
<td>Laterally unbraced length of member, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{avg}$</td>
<td>Average unrestrained length of gusset plate, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{CL}$</td>
<td>Centerline length of beam taken between joints, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{max}$</td>
<td>Average unrestrained length of gusset plate, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_c$</td>
<td>Length of column or brace between supports, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_e$</td>
<td>Effective length for buckling, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{ef}$</td>
<td>Length of beam taken as the clear span between column flanges, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{com}$</td>
<td>Centerline length of beam taken between joints, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{core}$</td>
<td>Effective length for buckling, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{ee}$</td>
<td>End-to-end brace length, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{rot}$</td>
<td>Rotational clearance, defined in Figure C7.4-2</td>
<td></td>
</tr>
<tr>
<td>$L_v$</td>
<td>Length of beam between shear supports, clearance length between supports that resist translation in the direction of the shear force, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{vert}$</td>
<td>Vertical clearance between the brace and beam flange, in. (mm) for middle gusset plate connections, as shown in Figure C7.4-2</td>
<td></td>
</tr>
<tr>
<td>$L_u$</td>
<td>Unrestrained gusset plate length to the nearest edge member at Whitmore width end, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_p$</td>
<td>Unrestrained gusset plate length to the nearest edge member at Whitmore width center, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$L_{ul}$</td>
<td>Unrestrained gusset plate length to the nearest edge member at Whitmore width end, in. (mm)</td>
<td></td>
</tr>
<tr>
<td>$M_{CE}$</td>
<td>Expected flexural strength, kip-in. (N-mm)</td>
<td></td>
</tr>
<tr>
<td>$M_{CT}$</td>
<td>Expected flexural strength about the y-axis, kip-in. (N-mm)</td>
<td></td>
</tr>
<tr>
<td>$M_{CT,TB}$</td>
<td>Expected lateral-torsional buckling flexural strength about the y-axis, kip-in. (N-mm)</td>
<td></td>
</tr>
<tr>
<td>$M_{CL}$</td>
<td>Lower-bound flexural strength, kip-in. (N-mm)</td>
<td></td>
</tr>
<tr>
<td>$M_{CL,e}$</td>
<td>Lower-bound flexural strength of connection at the face of column, as shown in Figure C7.4-2</td>
<td></td>
</tr>
<tr>
<td>$M_{ucl,p}$</td>
<td>Expected lateral-torsional buckling flexural strength about the y-axis, kip-in. (N-mm)</td>
<td></td>
</tr>
</tbody>
</table>

**Note:** The above expressions are used in various sections of the document and are referenced accordingly.
Lower-bound lateral-torsional buckling flexural strength about the x-axis, kip-in. (N-mm) ..........C3.4a.2.a.2
Lower-bound flexural strength about the y-axis, kip-in. (N-mm) .................................................. C3.4a.2.a.2
Lateral-torsional buckling flexural strength about the x-axis, determined in accordance with Section C3.3a.2 or C3.3b.2 at \( P_{UF} = 0 \), kip-in. (N-mm) ..........C3.4a.2.a.2
Expected flexural strength about the y-axis, kip-in. (N-mm) .................................................. C3.4a.2.a.2
Expected flexural strength about the y-axis, kip-in. (N-mm) .................................................. C3.4a.2.a.2
Flexural strength about the y-axis, determined in accordance with Section C3.3a.2 or C3.3b.2 at \( P_{UF} = 0 \), kip-in. (N-mm) ..........C3.4a.2.a.2
Nominal flexural strength, kip-in. (N-mm) .................................................................................. C2.3a.1
Expected plastic flexural strength reduced for the effect of axial force about the axis of bending defined in Section C2.3a.1, kip-in. (N-mm) .................................................. C2.4a.1.a
Projected to the face of column, kips (N) ................................................................. C5.4a.1.a(1)
Expected plastic flexural strength about the x-axis, determined in accordance with Section C3.3a.2 at \( P = P_{UF} = 0 \), kip-in. (N-mm) ..........C3.4a.2.a.1
Expected plastic flexural strength about the y-axis, determined in accordance with Section C3.3a.2 at \( P = P_{UF} = 0 \), kip-in. (N-mm) ..........C3.4a.2.a.1
Expected plastic flexural strength reduced for the effect of axial force (compression or tension), kip-in. (N-mm) ....................... C3.3a.2
Expected plastic flexural strength of beam at the plastic hinge location, determined in accordance with Section C2.3a at the plastic hinge location, kip-in. (N-mm) ..........C3.3a.2
Bending moment about the x-axis, determined in accordance with ASCE/SEI 41, Section 7.5.2.1.1, kip-in. (N-mm) ..........C3.4a.2.a.1
Bending moment about the y-axis, determined in accordance with ASCE/SEI 41, Section 7.5.2.1.1, kip-in. (N-mm) ..........C3.4a.2.a.1
Bending moment for force-controlled flexure about the x-axis, kip-in. (N-mm) ..........C3.4a.2.a.2
Bending moment for force-controlled flexure about the y-axis, kip-in. (N-mm) ..........C3.4a.2.a.2
Bending moment about the x-axis, computed in accordance with ASCE/SEI 41, Section 7.5.2.1.1, kip-in. (N-mm) ..........C3.4a.2.a.2
Bending moment about the y-axis, computed in accordance with ASCE/SEI 41, Section 7.5.2.1.1, kip-in. (N-mm) ..........C3.4a.2.a.2
Bending moment about the y-axis, kip-in. (N-mm) .................................................. C3.4a.2.a.2
Expected first yield moment of beam, kip-in. (N-mm) .................................................. C5.4a.1.a(1)
Least number of bolts or rivets connecting the top or bottom angle to the beam flange ..........C5.3a.42(a)(i)
Least number of bolts or rivets connecting the flange of the top or bottom split-tee to the column flange ..........C5.3a.42(b)
Axial force, kips (N) ..........C3.2b
Axial force (compression or tension), kips (N) ..........C3.3a.2
Expected compressive strength, kips (N) ..........C3.3a.1
Expected compressive and tensile strength for a buckling-restrained brace ..........C3.3a.1
Expected tensile strength of horizontal angle leg, kips (N) ..........C5.3a.42(a)(ii)
Lower-bound compressive strength, kips (N) ..........C3.3b.1
The axial force component of the gravity load as determined by ASCE/SEI 41, Equation 7-3 of ASCE/SEI
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_n$</td>
<td>Nominal compressive strength, kips (N)</td>
</tr>
<tr>
<td>$P_a$</td>
<td>Nominal axial strength, kips (N)</td>
</tr>
<tr>
<td>$P_T$</td>
<td>Nominal compressive strength, kips (N)</td>
</tr>
<tr>
<td>$P_{UD}$</td>
<td>Tensile force in the member, determined in accordance with ASCE/SEI 41, Section 7.5.2.1.1, kips (N)</td>
</tr>
<tr>
<td>$P_{UF}$</td>
<td>Axial force (compression or tension) in the member, determined in accordance with ASCE/SEI 41, Section 7.5.2.1.2, kips (N)</td>
</tr>
<tr>
<td>$P_{ye}$</td>
<td>Expected axial yield strength, kips (N)</td>
</tr>
<tr>
<td>$P_{ye,cf}$</td>
<td>Expected axial yield strength of the column flange, kips (N)</td>
</tr>
<tr>
<td>$P_{LB}$</td>
<td>Lower-bound axial yield strength, kips (N)</td>
</tr>
<tr>
<td>$Q$</td>
<td>Force demand, kips (N) or kip- in. (N-mm)</td>
</tr>
<tr>
<td>$Q_{CE}$</td>
<td>Expected component strength, kips (N) or kip-in. (N-mm)</td>
</tr>
<tr>
<td>$Q_{CL}$</td>
<td>Lower-bound component strength, kips (N) or kip-in. (N-mm)</td>
</tr>
<tr>
<td>$Q_{CL}$</td>
<td>Lower-bound shear strength, kips (N)</td>
</tr>
<tr>
<td>$Q_{UD}$</td>
<td>Deformation-controlled action caused by gravity loads and earthquake forces, determined in accordance with ASCE/SEI 41, Section 7.5.2.1.2</td>
</tr>
<tr>
<td>$Q_{CF}$</td>
<td>Force-controlled demand, kips (N)</td>
</tr>
<tr>
<td>$Q_s$</td>
<td>Expected component yield strength, kips (N) or kip-in. (N-mm)</td>
</tr>
<tr>
<td>$R_y$</td>
<td>Nominal strength, specified in the Specification</td>
</tr>
<tr>
<td>$R_y$</td>
<td>Ratio of the expected tensile strength to the specified minimum tensile strength, $F_y$</td>
</tr>
<tr>
<td>$R_y$</td>
<td>Ratio of the expected yield stress to the specified minimum yield stress, $F_y$</td>
</tr>
<tr>
<td>$S$</td>
<td>Elastic section modulus about the axis of bending, in. $^3$ (mm$^3$)</td>
</tr>
<tr>
<td>$S_p$</td>
<td>Elastic section modulus of beam, in. $^3$ (mm$^3$)</td>
</tr>
<tr>
<td>$S_n$</td>
<td>Nominal diaphragm strength, kip/in (N/mm)</td>
</tr>
<tr>
<td>$S_{nb}$</td>
<td>Nominal shear strength per unit length of a diaphragm controlled by out-of-plane buckling, kip/in (N/mm)</td>
</tr>
<tr>
<td>$S_{sf}$</td>
<td>Nominal shear strength per unit length of diaphragm controlled by connections, kip/in (N/mm)</td>
</tr>
<tr>
<td>$S_x$</td>
<td>Elastic section modulus of the smaller member taken about the $x$-axis, in. $^3$ (mm$^3$)</td>
</tr>
<tr>
<td>$S_y$</td>
<td>Elastic section modulus of the smaller member taken about the $y$-axis, in. $^3$ (mm$^3$)</td>
</tr>
<tr>
<td>$T_{CE}$</td>
<td>Expected tensile strength, kips (N)</td>
</tr>
<tr>
<td>$T_{CL}$</td>
<td>Lower-bound tensile strength, kips (N)</td>
</tr>
<tr>
<td>$T_{GL}$</td>
<td>Gross section yield strength, kips (N)</td>
</tr>
<tr>
<td>$T_{CL}$</td>
<td>Block shear rupture strength, kips (N)</td>
</tr>
<tr>
<td>$V_{CE}$</td>
<td>Expected shear strength, kips (N)</td>
</tr>
<tr>
<td>$V_{CL}$</td>
<td>Lower-bound shear strength, kips (N)</td>
</tr>
<tr>
<td>$V_n$</td>
<td>Nominal shear strength, kips (N)</td>
</tr>
<tr>
<td>$V_{pz}$</td>
<td>Panel-zone shear at the development of a hinge (expected first yield) at the critical location of the connection, kips (N)</td>
</tr>
<tr>
<td>$V_{pz}$</td>
<td>Panel zone shear, kips (N)</td>
</tr>
<tr>
<td>$V_{pe}$</td>
<td>Nominal shear strength, $V_n$, in the absence of axial force, from Seismic Provisions Section F3, with $F_{ye}$ substituted for $F_y$, kips (N)</td>
</tr>
<tr>
<td>$V_{ye}$</td>
<td>Expected plastic shear strength of the section reduced for the effect of axial force (compression or tension), kips (N)</td>
</tr>
<tr>
<td>$V_{ye}$</td>
<td>Expected shear strength of the panel zone reduced for the effect of axial force (compression or tension), kips (N)</td>
</tr>
<tr>
<td>$Z_p$</td>
<td>Plastic section modulus of beam, in. $^3$ (mm$^3$)</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Modeling parameter shown in Figure C1.1</td>
</tr>
</tbody>
</table>

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
A factor of 2 for weld lines on each side of the plate (in thickness direction).
Radius of gyration about γ-axis, in. (mm) .......................................................... Table C3.6
Design wall thickness of HSS member, in. (mm) .............................................. Table C3.6
Thickness of continuity plate, in. (mm) ............................................................. C5.4a.1.a(1)
Thickness of angle, in. (mm) .......................................................... C5.3a.42(a)(ii)
Thickness of beam flange, in. (mm) ............................................................. C5.4a.1.a(1)
Thickness of column flange, in. (mm) .......................................................... C4.4a.2
Thickness of flange, in. (mm) .......................................................... Table C3.6
Thickness of the flange of the split-tee, in. (mm) .......................................... C5.3a.42(b)(iv)
Thickness of the smaller flange or web, in. (mm) .......................................... C5.3b.43(b)
Total thickness of panel zone, including doubler plates, in. (mm) ............... C4.3a
Thickness of flange plate, in. (mm) ............................................................. C5.3a.42(c)
Thickness of gusset plate, in. (mm) ............................................................. C7.2b
Thickness of the split-tee stem, in. (mm) .................................................. C5.3a.42(b)(ii)
Thickness of web, in. (mm) .......................................................... C2.2a.4a.1.b
Thickness of steel plate shear wall, in. (mm) ................................................. C6.2c
Length of the flange angle, in. (mm) .......................................................... C5.3a.42(a)(iv)
Length of split-tee, in. (mm) .......................................................... C5.3a.42(b)(iv)
Weld leg size, in. (mm) .......................................................... C7.3a.4
Axial deformation at expected compressive buckling strength, in. (mm) .......... C3.4a.1.b
Plastic axial deformation, in. (mm) ............................................................. C3.4a.1.b
Axial deformation at expected tensile yield strength, in. (mm) ..................... C3.4a.1.b
Deformation ........................................................................................................ E3.4a
Yield axial deformation, in. (mm) ............................................................. C3.4a.1.b
Ratio of smaller depth of the connecting beams at a panel zone to the thickness of the column flange = \( \frac{d_A}{d} \) .......................................................... C4.4a.2
Post-elastic hardening slope shown in Figure C1 ............................................. C1
Compression strength adjustment factor ....................................................... C3.3a.1
Total shear deformation, rad .......................................................... C2.4b.2.b
Initial shear deformation, rad .......................................................... G3.4a.2
Plastic shear deformation, rad .......................................................... C2.4a.2.a.b
Permissible plastic shear deformation of the panel zone, rad ................. C4.4a.2
Yield shear deformation, rad ............................................................. C2.4a.1.b
Total chord rotation, rad .......................................................... C2.4b.1.b
Welded gusset plate rotation capacity, rad ........................................... C7.4a.4
Plastic chord rotation, rad .......................................................... C2.4a.1.b
Plastic chord rotation demand, rad ............................................................. C3.4a.2.b
Plastic rotation angle, rad ........................................................................... C5.4a.1.a(2)
Yield chord rotation .......................................................... C2.4a.1.b
Knowledge factor ....................................................................................... \( b 1.0 A 4.1 \)
Width-to-thickness ratio for the element, as defined in the Seismic Provisions Table C3.21
respectively, as defined in Seismic Provisions: Table D1.1, with \( R_y F_y \) replaced by \( F_y e \) ........................................................................................................ Table C2.1
Lower-bound tensile strength of splices made with partial-joint-penetration groove welds, ksi (MPa) .......................................................... C5.3b.43(b)
Weld stress demand on splice, ksi (MPa) .................................................. C5.3b.3(b)
Stiffness reduction parameter, as given in Specification Chapter C ............... C3.2b
Stiffness reduction parameter, as given in Equations C3.13a and C3.13b ........ C3.4a.2.b
Strain-hardening adjustment factor ............................................................. C3.3a.1

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
GLOSSARY

Note:
Terms designated with † are common AISI-AISC terms that are coordinated between the two standards development organizations.

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 designed.

User Note: The applicable building code is the building code under which the evaluation is performed and retrofit is designed; it should not be taken as the building code under which the structure was originally designed.

Assembly. Two or more interconnected components.

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

Base. The level at which the horizontal seismic ground motions are considered to be imparted to the structure.

Beam. Nominally horizontal structural member that has the primary function of resisting bending moments.

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

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) 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–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.

Chords and collectors. Chords and collectors are 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. See diaphragm chord.

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

Column. Nominally vertical structural member that has the primary function of resisting axial compressive force.

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

Composite. Condition in which steel and concrete elements and members work as a unit in the distribution of internal forces.
Concentrically braced frame (CBF). Braced frame element in which component worklines 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.

Connection. Combination of structural elements and connecting materials used to transmit forces between two or more members.

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 a panel zone.

Deformation-controlled action. An action that has an associated deformation that is allowed to exceed the yield value of the element being evaluated. The extent of permissible deformation beyond yield is based on component capacity modification factors.

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

Design earthquake. A user-specified earthquake for the evaluation or retrofit of a building that has ground-shaking criteria described in ASCE/SEI 41, Chapter 2.

Design strength. Resistance factor multiplied by the nominal strength.

Diagonal bracing. Inclined components designed to carry axial force, enabling a structural frame to act as a truss to resist lateral forces.

Diaphragm chord. A boundary component perpendicular to the applied force that is provided to resist tension or compression caused by the diaphragm moment. See chords and collectors.

Diaphragm collector. A component parallel to the applied force that transfers lateral forces from the diaphragm of the structure to vertical elements of the seismic force-resisting system. See chords and collectors.

Diaphragm. A horizontal (or nearly horizontal) structural element, such as a floor or roof system, used to transfer inertial lateral forces to vertical elements of the seismic force-resisting system.

Drift. Horizontal deflection at the top of the story relative to the bottom of the story.

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

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 variability in material strength as well as strain-hardening and plastic section development.

Fastener. Generic term for bolts, rivets, or other connecting devices.

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

Fully restrained moment connection. Connection capable of transferring moment with negligible rotation between connected members.

HSS (hollow structural section). Square, rectangular, or round hollow structural steel section produced in accordance with one of the product specifications in Section A2(3).

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.”

In-plane wall. See shear wall.

Joint. A region where ends, surfaces, or edges of two or more components are attached; categorized by type of fastener or weld used and method of force transfer.

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.
Lightweight concrete. Structural concrete with an equilibrium density of 115 lb/ft$^3$ (1800 kg/m$^3$) or less as determined by ASTM C567.

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.

Moment-resisting frame (MRF). A frame capable of resisting horizontal forces caused by the members (beams and columns) and connections resisting forces primarily by flexure.

Nominal strength. The capacity of a structure or component (without a resistance factor or safety factor) to resist the effects of loads, as determined by (a) computations using specified material strengths and dimensions, and formulas derived from accepted principles of structural mechanics; or (b) field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

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.

Occupancy. The purpose for which a building, or part thereof, is used or intended to be used, designated in accordance with the governing regulation, building code, or policy.

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

Panel zone. Area of a column at a beam-to-column connection delineated by the adjacent beam and column flanges. Web region of beam-to-column connection delineated by the extension of beam and column flanges through the connection, transmitting moment through a shear panel.

Partially restrained moment connection. Connection capable of transferring moment with rotation between connected members that is not negligible.

Performance Level: A limiting structural damage state, used in the definition of Performance Objectives.

Performance Objective: One or more pairings of a selected seismic hazard level with both 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.

Pipe. See HSS.

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 member resistance (or required strength). Forces, stresses, and deformations acting on a structural component, determined by either structural analysis (for the load combinations found in ASCE/SEI 41) as appropriate, or as specified by the Specification and these Provisions.

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

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

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.

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

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.

Seismic hazard intensity. A user-specified earthquake for the evaluation or retrofit of a building that has ground-shaking criteria described in ASCE/SEI 41, Chapter 2.

Seismic hazard level. Ground-shaking demands of specified severity, developed on either a probabilistic or deterministic basis.

Shear wall†. Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.

Specified minimum tensile strength. Lower limit of tensile strength specified for a material as defined by ASTM.

Specified minimum yield stress†. Lower limit of yield stress specified for a material as defined by ASTM.

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 limiting structural damage state; used in the definition of Performance Objectives.

Structural system. An assemblage of structural components that are joined together to provide regular interaction or interdependence.

Subassembly. A portion of an assembly.

Target displacement. An estimate of the maximum expected displacement of the roof of a building calculated for the design earthquake seismic hazard intensity.

Tensile strength (of material)†. Maximum tensile stress that a material is capable of sustaining as defined by ASTM.

Tubing. See HSS.

V-braced frame. A concentrically braced frame (CBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span.

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

Yield strength†. Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.
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)
- AWS (American Welding Society)
- BRB (buckling-restrained brace)
- BRBF (buckling-restrained braced frame)
- CBF (concentrically braced frame)
- CJP (complete joint penetration)
- CP (collapse prevention)
- 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)
- LS (life safety)
- PR (partially restrained)
- SEI (Structural Engineering Institute)
- 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. Condition Assessment
A5. Material Properties
A6. Subassembly Tests

A1. SCOPE

The Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings, hereafter referred to as these Provisions, shall govern the seismic evaluation and retrofit of structural steel, composite, wrought iron, and cast iron components of existing buildings subject 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.

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. Where required by ASCE/SEI 41, these Provisions are intended to be used in conjunction with the Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341, hereafter referred to as the Seismic Provisions.

These Provisions includes 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.

The strength of existing and new components shall be determined by considering the applicable provisions of Chapters B through K of the Specification for Structural Steel Buildings, ANSI/AISC 360, hereafter referred to as the Specification. Additionally, the existing and new components shall be evaluated considering the requirements in these Provisions.

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
There are specific instances in these Provisions where an alternate formulation for capacity is specified. In such cases, the alternate provides for lower capacity than would be obtained from the Specification for the specific action or condition being referenced.

Nominal strengths of existing and new components not provided in these Provisions shall be determined in accordance with the Seismic Provisions and the Specification for Structural Steel Buildings, ANSI/AISC 360, hereafter referred to as the Specification.

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 360-16 Specification for Structural Steel Buildings
ANSI/AISC 341-16 Seismic Provisions for Structural Steel Buildings
ANSI/AISC 358-16 Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

(c) American Iron and Steel Institute (AISI)

ANSI/AISI S100-16 North American Specification for the Design of Cold-Formed Steel Structural Members

(d) American Society of Civil Engineers (ASCE)

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

(e) ASTM International (ASTM)

ASTM A36/A36M Standard Specification for Carbon Structural Steel
ASTM A242/A242M Standard Specification for High-Strength Low-Allow Structural Steel
ASTM A307 Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60,000 PSI Tensile Strength
ASTM A500/A500M Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
ASTM A568/A568M Standard Specification for Steel, Sheet, Carbon, Structural, and High-Strength, Low-Alloy, Hot-Rolled, and Cold-Rolled, General Requirements for
A3. GENERAL REQUIREMENTS

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

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

Testing of components and assemblies of components shall be in accordance with Section A6.

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

User Note: Because of these Provisions' unique application to existing buildings, the requirements herein cite ASTM standards that have been withdrawn, which means that the standard is considered obsolete and is no longer maintained by ASTM. Availability of withdrawn standards may be limited. The Commentary provides information regarding alternative sources for information specified in the withdrawn standards of interest to users of these Provisions.
A4. CONDITION ASSESSMENT

1. General

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

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

Review of available construction documents shall be performed to identify the vertical and lateral load-carrying systems, critical components of these systems, and any modifications to these systems, their components, and the overall configuration of the structure. Where such documentation fails to provide adequate information to identify these aspects of the structure, such documentation shall be supplemented by field survey drawings prepared in absence of design drawings as required by the data collection requirements of ASCE/SEI 41, Section 6.2.

**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 structural steel components and documentation of the presence of any degradation.
(b) Verification of the presence and configuration of representative structural steel 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 mathematical-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 are to be assessed in accordance with Section A4.4.
2. **Visual Usual Condition Assessment**

If design drawings are available, and the design drawings specify the details of the connections, at least one structural steel connection of each type and a portion of each connected component shall be exposed. If no deviations from the available drawings exist, the sample shall be considered representative. If deviations from the available drawings exist, then removal of additional coverings from connections of that type and its connected components shall be performed until the extent of deviations is determined.

Where the available design drawings do not specify the details of the connections, assessment of connections shall be conducted in accordance with Section A4.3.

3. **Comprehensive Condition Assessment**

In the absence of construction drawings, at least three structural steel connections of each type, or all connections of each type, shall be identified and a portion of each connected component shall be exposed for the primary structural components. If no deviations within a connection type group are observed, the sample shall be considered representative. If deviations within a connection type group are observed, then additional connections of the same type and their connected components shall be exposed until the extent of deviations is determined.

4. **Component Properties**

The following properties of representative structural steel 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 of components at critical sections
(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 cross-sectional dimensions of components, connecting elements, and fasteners is permitted.

**User Note:** Documented cross-sectional dimensions of components and fasteners can be found listed 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 for the particular structure 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 material tests, 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: yield strength, tensile strength, deformability, and notch toughness.

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

Where materials testing is required by these Provisions or by ASCE/SEI 41, Section 6.2, testing to quantify material properties shall be as specified in Section A5.3.

The extent of testing of structural steel shall comply with the requirements of Section A5.4.

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

### 2. Default Material Properties

2a. **Structural Steel Materials from 1901 and After**

Materials in structural steel components of buildings constructed in 1901 and after shall be classified based on year of construction, the applicable ASTM specification and material grade and, if applicable, shape group in accordance with Table A5.1. The default lower-bound material properties for structural steel are permitted to be determined as follows:

(a) Where available construction documents identify material by ASTM Specification, and the ASTM Specification for the year of construction is listed in Table A5.1, default lower-bound material properties for structural steel, \( F_{yLB} \) and \( F_{uLB} \), shall be taken in accordance with Table A5.1 for material conforming to the specifications listed therein, where \( F_{yLB} \) is the lower-bound yield stress and \( F_{uLB} \) is the lower-bound tensile strength. Default expected material properties, \( F_{ye} \) and \( F_{ue} \), are permitted to be determined by multiplying default lower-bound values by the applicable factor taken from Table A5.2, where \( F_{ye} \) is the expected yield stress and \( F_{ue} \) is the expected tensile strength.

**User Note:** The values for default lower-bound properties listed in Table A5.1 are not always the same as specified minimum values as specified in the corresponding specification. Table A5.1 includes lower-bound values that are generally greater than specified minimum values.

(a)(b) For the available construction documents identify material by specification but the material specifications is not listed in Table A5.1, lower-bound material properties shall be taken as the minimum specified properties in accordance with the material specification, the Specification, or the value specified by available construction documents. Default expected material properties are permitted to be determined as \( F_{ye} = 1.10 F_{yLB} \) and \( F_{ue} = 1.10 F_{uLB} \). \( F_{ye} \) is the expected yield stress and \( F_{ue} \) is the expected tensile strength.
User Note: The values for default lower-bound properties listed in Table A5.1 are not always the same as specified minimum values as specified in the corresponding specification. Table A5.1 includes lower-bound values that are generally greater than specified minimum values.

(b) Where the available construction documents identify material by ASTM specification, grades that are known, but are not the ASTM specification is not listed in Table A5.1, if and the steel ASTM specification is permitted in the Seismic Provisions for use in structural steel seismic force-resisting systems, the default lower-bound material property is permitted to be taken as the minimum specified property in accordance with the ASTM specification, and (2) then $R_y$ and $R_t$, as specified in the Seismic Provisions, are permitted to be used to translate from lower-bound values, $F_{yLB}$ and $F_{uLB}$, to expected values, $F_{ye}$ and $F_{ue}$ respectively.

User Note: The translation factors specified in the Seismic Provisions are not the same as the factors listed in Table A5.2. The factors $R_y$ and $R_t$ from the Seismic Provisions are applied to specified minimum values, whereas the factors in Table A5.2 are applied to default lower-bound values.

(c)(d) Where the available construction documents specify minimum yield stress and minimum tensile strength, lower-bound material properties are permitted to be taken as the values specified by available construction documents. Default expected material properties are permitted to be determined as $F_{ye}=1.10 F_{yLB}$ and $F_{ue}=1.10 F_{uLB}$.


Default lower-bound material properties, $F_{yLB}$ and $F_{uLB}$, for cast iron, wrought iron, and pre-1901 standardized structural steel are permitted to be taken in accordance with Table A5.3. Default expected material properties are permitted to be determined as $F_{ye}=1.10 F_{yLB}$ and $F_{ue}=1.10 F_{uLB}$.

2c. Weld Metal

1. Default Lower-Bound Tensile Strength

The default lower-bound tensile strength for weld metal is permitted to be taken as the specified tensile strength of weld metal determined as follows:

(a) Where available construction documents indicate the filler metal classification strength, that value is permitted to be assumed as the specified minimum tensile strength for weld metal.

(b) For construction dated in 1980 and earlier predating 1970, 60 ksi (415 MPa) is permitted to be assumed as the specified minimum tensile strength for weld metal, and for construction after 1980, 70 ksi is permitted to be assumed as the specified minimum tensile strength for weld metal.

2. Default Lower-Bound Charpy V-Notch Toughness

The default lower-bound Charpy V-notch toughness for weld metal is permitted to be taken as follows:

(a) 10 ft-lb at 70°. For all construction predating 1997, and construction after 1997 when
there is no indication the weld is a demand critical weld, where no evidence is available that filler metal is rated for higher Charpy V-notch toughness 10 ft-lb at 70°, is permitted.

(b) 40 ft-lb at 70°. For construction after 1997, where there is indication the available construction documents specify that the weld is a demand critical weld, 40 ft-lb at 70°.

2db. Rivet Material

The default lower-bound tensile strength for rivet material is permitted to be taken as the specified minimum tensile strength of rivet material determined as follows:

(a) Where available construction documents indicate the ASTM Specification type and grade of rivet, the specified minimum tensile strength of the rivet material shall be determined from the listed specification or determined from Table A5.1 for the Specification types and grades of rivets listed.

(b) Where available construction documents indicate the specified minimum tensile strength of rivet material, that value is permitted to be used.

(c) Where rivet material information is not shown on available construction documents, rivet grade is permitted to be determined in accordance with Specification Appendix 5, Section 5.2.6, and the specified minimum tensile strength of the rivet material based upon the grade so determined.

Default expected tensile strength for rivet material is permitted to be determined by multiplying default lower-bound tensile strength by a factor of 1.10.

2ee. Bolt Material

The default lower-bound tensile strength for bolt material is permitted to be taken as the specified minimum tensile strength of bolt material determined as follows:

(a) Where available construction documents indicate the specification type and grade of bolts, the specified minimum tensile strength of the bolt material shall be determined from the listed specification.

(b) Where available construction documents indicate the specified minimum tensile strength of bolt material, that value is permitted to be used.

(c) Where bolt material information is not shown on available construction documents, bolt grade is permitted to be determined in accordance with Specification Appendix 5, Section 5.2.6, and the specified minimum tensile strength of the bolt material based upon the grade so determined.
# TABLE A5.1
Default Lower-Bound Material Strengths for Structural Steel

<table>
<thead>
<tr>
<th>Date of Specification</th>
<th>Specification</th>
<th>Remarks</th>
<th>Lower-Bound Yield Stress ( F_{y, LB} )</th>
<th>Lower-Bound Tensile Strength ( F_{u, LB} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1901–1908</td>
<td>ASTM A9</td>
<td>Rivet steel</td>
<td>30 (207)</td>
<td>50 (345)</td>
</tr>
<tr>
<td>1909–1923</td>
<td>ASTM A9</td>
<td>Medium steel</td>
<td>30 (207)</td>
<td>60 (414)</td>
</tr>
<tr>
<td>1924–1931</td>
<td>ASTM A7 and ASTM A9</td>
<td>Structural steel</td>
<td>30 (207)</td>
<td>55 (379)</td>
</tr>
<tr>
<td>1932</td>
<td>ASTM A140-32T issued as a tentative revision to ASTM A9 (Buildings)</td>
<td>Plates, shapes, bars</td>
<td>33 (228)</td>
<td>60 (414)</td>
</tr>
<tr>
<td>1933</td>
<td>ASTM A140-32T discontinued and ASTM A9 (Buildings) revised Oct. 30, 1933</td>
<td>Structural steel</td>
<td>30 (207)</td>
<td>55 (379)</td>
</tr>
<tr>
<td>1934–1967</td>
<td>ASTM A7 and ASTM A9</td>
<td>Structural steel</td>
<td>33 (228)</td>
<td>60 (414)</td>
</tr>
<tr>
<td>1961–Present</td>
<td>ASTM A572, Gr. 50</td>
<td>Structural steel</td>
<td>50 (345)</td>
<td>65 (448)</td>
</tr>
<tr>
<td>Group 1</td>
<td>W-Shapes in Group 1d</td>
<td>44 (303)</td>
<td>62 (427)</td>
<td></td>
</tr>
<tr>
<td>Group 2</td>
<td>W-Shapes in Group 2d</td>
<td>41 (283)</td>
<td>59 (407)</td>
<td></td>
</tr>
<tr>
<td>Group 3</td>
<td>W-Shapes in Group 3d</td>
<td>39 (269)</td>
<td>60 (414)</td>
<td></td>
</tr>
<tr>
<td>Group 4</td>
<td>W-Shapes in Group 4d</td>
<td>37 (255)</td>
<td>62 (427)</td>
<td></td>
</tr>
<tr>
<td>Group 5</td>
<td>W-Shapes in Group 5d</td>
<td>41 (283)</td>
<td>70 (483)</td>
<td></td>
</tr>
</tbody>
</table>

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
<table>
<thead>
<tr>
<th>Group</th>
<th>Shapes in Group</th>
<th>c ksi</th>
<th>d ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1990-1998 Present</td>
<td>ASTM A36 and Dual Grade Structural steel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Group 1</td>
<td>W-Shapes in Group 1d</td>
<td>49 (338)&lt;sup&gt;c&lt;/sup&gt;</td>
<td>66 (455)&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>Group 2</td>
<td>W-Shapes in Group 2d</td>
<td>50 (345)&lt;sup&gt;c&lt;/sup&gt;</td>
<td>67 (462)&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>Group 3</td>
<td>W-Shapes in Group 3d</td>
<td>52 (359)&lt;sup&gt;c&lt;/sup&gt;</td>
<td>70 (483)&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>Group 4</td>
<td>W-Shapes in Group 4d</td>
<td>49 (338)&lt;sup&gt;c&lt;/sup&gt;</td>
<td>70 (483)&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>1998-Present</td>
<td>ASTM A992 Structural steel</td>
<td>50 (345)</td>
<td>65 (448)</td>
</tr>
<tr>
<td>All</td>
<td>ASTM A53, Gr. B Pipe</td>
<td>45 (310)&lt;sup&gt;c&lt;/sup&gt;</td>
<td>60 (414)&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>All</td>
<td>ASTM A500, Gr. B Round HSS</td>
<td>48 (381)&lt;sup&gt;c&lt;/sup&gt;</td>
<td>60 (414)&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td>Rectangular HSS</td>
<td>50 (345)&lt;sup&gt;c&lt;/sup&gt;</td>
<td>62 (427)&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>All</td>
<td>ASTM A500, Gr. C Round HSS</td>
<td>50 (345)&lt;sup&gt;c&lt;/sup&gt;</td>
<td>62 (427)&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td>Rectangular HSS</td>
<td>50 (345)&lt;sup&gt;c&lt;/sup&gt;</td>
<td>62 (427)&lt;sup&gt;c&lt;/sup&gt;</td>
</tr>
<tr>
<td>All</td>
<td>ASTM A1085 Gr. A (50 ksi) Rectangular and Round HSS</td>
<td>50 (345)</td>
<td>65 (448)</td>
</tr>
</tbody>
</table>

**User Note:** Lower-bound values listed in this table are not always the same as specified minimum values and therefore should not be used in place of specified minimum values.

**User Note:** Where applicable, the indicated values are representative of material extracted from the flanges of wide-flange shapes.

**User Note:** Values are based on specified minimum values, unless value is noted with superscript "c."

**User Note:** Values are based on mean minus one standard deviation values from statistical data and further reduced from statistical data.

**User Note:** W-shape size groupings are listed in Commentary Table C-A5.1.
TABLE A5.23
Factors to Calculate Translate Lower-Bound Properties to Expected Properties from Lower-Bound Properties for Structural Steel

<table>
<thead>
<tr>
<th>Property</th>
<th>Year/Date of Specification</th>
<th>Specification</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength</td>
<td>Before 1961</td>
<td>ASTM A7</td>
<td>1.00</td>
</tr>
<tr>
<td>Yield Stress</td>
<td>Before 1961</td>
<td>ASTM A7</td>
<td>1.15</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>1961–1980</td>
<td>ASTM A36</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>1980–1994</td>
<td>ASTM A36</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>1994–Present</td>
<td>ASTM A36</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A572 Gr. 42</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A572 Gr. 50</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A500 Gr. B &amp; C, Round &amp; Rectangular</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A1085 Gr. A, Rectangular</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>1994–Present</td>
<td>ASTM A36</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A992</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A53 Gr. B, Pipe</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A500 Gr. B &amp; C, Round &amp; Rectangular</td>
<td>1.40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A572 Gr. 42</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A572 Gr. 50</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM A1085 Gr. A, Rectangular</td>
<td>1.25</td>
</tr>
</tbody>
</table>

**User Note:** The factors listed in this table are specifically intended for use with the lower-bound values listed in Table A5.1.
### TABLE A5.32
Default Lower-Bound Material Strengths for Historical Structural Metals

<table>
<thead>
<tr>
<th>Year of Construction</th>
<th>Material</th>
<th>Lower-Bound Yield Stress, $F_{y, LB}$ ksi (MPa)</th>
<th>Lower-Bound Tensile Strength, $F_{u, LB}$ ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Any Before 1920</td>
<td>Cast Iron</td>
<td>See Chapter I</td>
<td></td>
</tr>
<tr>
<td>Any Before 1920</td>
<td>Wrought Iron</td>
<td>18 (124)</td>
<td>25 (170)</td>
</tr>
<tr>
<td>Pre-Before 1901</td>
<td>Pre-Standardized Structural Steel</td>
<td>24 (165)</td>
<td>36 (248)</td>
</tr>
</tbody>
</table>

**User Note:** Lower-bound values are not the same as specified minimum values and should not be used in place of specified minimum values.

### 3. Test Requirements

The determination of in-place material properties shall be accomplished through removal of samples and laboratory testing. Laboratory testing of samples to determine in-place material properties of structural steel components shall be performed in compliance with consensus standards published by ASTM, AISI, and AWS, and in accordance with Specification Appendix 5.

### 3a. Sampling and Repair of Sampled Locations

The determination of in-place material properties shall be accomplished through removal of samples and laboratory testing. Sampling shall take place in regions of components where the decreased section strength caused by the sampling remains higher than the capacity required in the component at the reduced section to resist the design loads. Alternately, where the reduced 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/ restored by repairs to the section.

Where a weld or a portion of a weld is to be removed, details regarding weld removal shall be defined. Where a weld is removed or where repairs are otherwise necessary to compensate for removed material, details describing the repairs shall be prepared/defined. As part of these repairs, the location where the material was removed shall be ground smooth. The repair shall be designed to provide equivalent or greater strength and ductility compared to the existing condition.

If a connector such as a bolt or rivet is removed for testing, a bolt of the same nominal diameter and of at least the same tensile strength as the existing bolt or rivet shall be reinstalled at the time of sampling.

If a weld or a portion of a weld is removed for testing, details regarding weld removal shall be supplied. When repairs are necessary to compensate for the removed material, details describing the retrofit methodology shall be provided.
Where required by ASCE/SEI 41 or these Provisions, testing for structural steel shall meet the requirements for usual testing in Section A5.4b or comprehensive testing in Section A5.4c.

3ba. Interpretation of Test Results

Expected in-place material properties for structural steel shall be taken as mean test values. Lower-bound in-place material properties shall be based on mean test values minus one standard deviation, except that where the material is conclusively identified as conforming to a defined standard material specification, in-place lower-bound properties need not be taken as less than the minimum-specified minimum properties for that specification.

4. Extent of Testing

The extent of in-place structural steel and wrought iron materials testing required to determine in-place material properties 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 for structural steel if material properties are available from original construction documents that include certified material test reports or certified reports of tests made in accordance with ASTM A6/A6M or A568/A568M. The structural steel material properties results of material tests obtained from such reports are permitted to be taken as in-place material properties when statistically analyzed in accordance with Section A5.3a. If such in-place properties differ from default lower-bound structural steel material properties given in Tables A5.1 and A5.2, material properties for evaluation and retrofit shall be selected such that the largest demands on components are generated.

4b. Usual Testing

The minimum number of tests to determine the in-place yield and tensile strengths of structural steel material properties for usual data collection shall be based on the following criteria:

(a) Where default yield stress and default tensile strength, both lower bound and expected, for structural steel materials from 1901 and after are unambiguously established in accordance with Sections A5.2a, subparagraphs (a), (b), or (c), it is permitted to use these default yield stresses and default tensile strengths as in-place yield stresses and in-place tensile strengths, respectively, without additional testing. If design drawings containing ASTM specification and material grade information are available, use of Table A5.1 to determine material properties is permitted without additional testing.

(b) If design drawings containing material property information are available, but the material properties are not listed in Table A5.1, use of minimum-specified material properties is permitted without additional testing.

(c) In the absence of construction documents defining material properties if no knowledge of the materials...
used exists, at least one strength coupon from each structural steel component type shall be removed for testing to determine yield stress and tensile strengths. At regions where samples have been extracted, the region where the material was removed shall be ground smooth and be patched or otherwise retrofitted to provide equivalent or greater strength and ductility compared to the existing condition.

(d) In the absence of construction documents defining weld filler metal classification and welding processes used if no knowledge of the weld filler metal classification and welding processes used exists, default values for weld strength shall be permitted to be used provided specified steel grade or weldability assessment in accordance with AWS D1.1 indicates that existing steel is weldable.

4c. Comprehensive Testing

The minimum number of tests to determine the in-place yield and tensile strengths of structural steel material properties for comprehensive data collection shall be based on the following criteria:

(a) Where default yield stress and default tensile strength, both lower bound and expected, for structural steel materials from 1901 and after are unambiguously established in accordance with Sections A5.2a, subparagraphs (a), (b) or (c), it is permitted to use these default yield stresses and default tensile strengths as in-place yield stresses and in-place tensile strengths, respectively, without additional testing. If original construction documents containing ASTM specification and material grade information are available, use of Table A5.1 to determine material properties is permitted without additional testing, with the following exception. Where ASTM A36/A36M is specified and the building was constructed between 1987 and 1990, an evaluation using default values from Table A5.1 for both pre-1990 and post-1990 A36/A36M steel shall be conducted, or material testing in accordance with Section A5.4c(b) shall be performed.

(b) If original construction documents defining material properties 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 coupons and three bolts and rivets shall be randomly removed from each component type. At regions where samples have been extracted, the region where the material was removed shall be ground smooth and be patched or otherwise retrofit to provide equivalent or greater strength and ductility as the existing condition.

(c) If no knowledge of the materials properties used exists, at least two tensile strength coupons and two bolts and rivets shall be removed from each component type for every four floors or every 200,000 ft$^2$ (19 000 m$^2$). 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. At regions where samples have been extracted, the region where the material was removed shall be ground smooth and be patched or otherwise retrofit to provide equivalent or greater strength and ductility as the existing condition.

(d) In the absence of construction documents defining weld filler metal classification and welding processes used, default values for weld strength shall be permitted to be used provided specified steel grade or weldability assessment in accordance with AWS D1.1 indicates that existing steel is weldable. Alternatively, at least two weld metal samples for each component type having welded joints shall be obtained for laboratory testing. The sample shall consist of both local base and weld metal to determine the composite strength of the connection. Weld metal samples shall also be tested for Charpy V-notch impact strength in accordance with the requirements of AWS D1.1 using a temperature consistent with the lowest ambient temperature to which the weld may be exposed to. At regions where samples have been extracted, the region where the material was removed shall be ground smooth and be patched or otherwise retrofit to provide equivalent or greater strength and ductility as the existing condition.
condition.

(e) For historical structural wrought iron or pre-1901 standardized structural steel, at least three tensile strength coupons shall be extracted for each component type for every four floors or 200,000 ft² (19 000 m²) of construction. If initial tests provide material properties that are consistent with properties given in Table A5.13, tests shall be required for every six floors or 300,000 ft² (28 000 m²) of construction only. If these tests provide material properties that are nonuniform, additional tests shall be performed until the extent of different materials is established.

For other material properties, a minimum of three tests shall be conducted.

4d. Increased Amount of Testing

The results of any structural steel and wrought iron material testing performed shall be compared to the default lower-bound values in Tables A5.1 and A5.32 for the particular era of building construction. The amount of testing shall be doubled if the in-place expected and in-place lower-bound yield stress and tensile strengths determined from testing are lower than the default lower-bound values.

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 STEEL COMPONENTS

Every structural component with seismic force or deformations in an existing steel 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. This chapter addresses the required characteristics of steel components, seismic force, or deformations, 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 design drawings. If some sectional material loss or deterioration has occurred, the loss shall be quantified by direct measurements 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 extent of data collected, condition assessment, and materials testing performed 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, \( \kappa \), 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 to determine component strengths are permitted in conjunction with the linear analysis procedures in 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 shall be calculated in accordance with the general requirements in ASCE/SEI 41, Section 7.5, this chapter, Chapter C, and any framesystem-specific requirements set forth in Chapters D through I. Component strength shall be taken as the nominal strength provided for in Chapters B through K of the Specification, but using the expected or lower-bound properties, from Chapter A of these Provisions. Additionally, the existing and new components shall be evaluated considering the requirements in these Provisions. Where conditions are not covered by the Specification or these Provisions, designs are permitted to be based on testing or analysis, subject to the approval of the authority having jurisdiction. Where the component strength for a given action is determined by calculations not provided in these Provisions, procedures contained in the Specification or Seismic Provisions for determining the nominal strength, as modified in these Provisions, shall be used. Where alternative definitions of component strength are used, they shall be justified by testing or analysis.

User Note: Nominal strengths determined in accordance with the Specification or Seismic Provisions do not incorporate a resistance factor (i.e., \( \phi \)). When taking the component strength to be the nominal strength provided in the Specification, the component strength is not factored by a resistance factor (i.e., \( \phi \)) or a safety factor (i.e., \( \Omega \)) when evaluating the component in these Provisions.

3a. Deformation-Controlled Actions

Strengths for deformation-controlled actions on steel components shall be taken as expected component strengths, \( Q_{CE} \), obtained experimentally or calculated using accepted principles of structural mechanics. Expected component strength obtained experimentally shall be defined as the mean resistance expected over the range of deformations to which the component is likely to be subjected. 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 steel components shall be taken as lower-bound component strengths, \( Q_{CL} \), obtained experimentally or calculated using established principles of structural mechanics. Lower-bound component strength obtained experimentally shall be defined as mean resistance minus one standard deviation. Where calculations are used to determine lower-bound component strength, lower-bound material properties shall be used.

The lower-bound strength of components controlled by elastic buckling shall be the nominal strength multiplied by 0.85.

4. Permissible Performance Parameters

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 deformation, depending on the analysis type selected, and represent the capacity of an action for a given performance level.

Component permissible performance parameters shall be computed 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 of these Provisions.

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 deformation, depending on the analysis type selected, and represent the capacity of an action 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, \( m_c \).

For nonlinear analysis procedures, the permissible deformation 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. Additionally, for nonlinear analysis procedures, the permissible deformation for a force-controlled action shall be taken as the yield deformation, or the deformation at the onset of buckling, computed using lower-bound material properties.

B3. RETROFIT MEASURES

Seismic retrofit measures shall satisfy the requirements of these Provisions and the applicable provisions of ASCE/SEI 41.
If replacement of the structural steel existing component is selected as the retrofit measure, the new component shall be designed in accordance with these Provisions and detailed and constructed in accordance with the applicable building code.

Where welding to existing structural steel components is required as part of a retrofit, weldability of existing steel shall be determined in accordance with AWS D1.1/D1.1M, unless it is confirmed that the existing material conforms to a weldable material specification. The welding procedures shall be determined based on the chemistry of the base material and filler material, as specified in AWS D1.1/D1.1M, clause 8, and based on a fillet weld break test. In buildings constructed after 1960, material conforming to ASTM A36/A36M, ASTM A242/A242M, ASTM A307, ASTM A572/A572M, ASTM A913/A913M, ASTM A972/A972M, and ASTM A992/A992M shall be deemed to be weldable. If the chemistry of the base material is not known, it is permitted to perform a weldability fillet weld break test.

All new welds, added to resist seismic forces, shall conform to Structural Welding Code—Seismic Supplement (AWS D1.8/D1.8M), hereafter referred to as AWS D1.8/D1.8M. Welds shall be designated as (1) demand critical; (2) in accordance with AWS D1.8/D1.8M requirements, but not designated as demand critical; or (3) in accordance with the requirements of Structural Welding Code—Steel (AWS D1.1/D1.1M); depending on their placement in accordance with AWS D1.8/D1.8M, Figures C-1.1, C-1.2, and C-1.3, and other requirements for seismic force-resisting system elements in the Seismic Provisions and AWS D1.8/D1.8M. Complete-joint-penetration groove welds designed to yield due to seismic loading, and the failure of which would result in substantial loss of seismic force-resisting system integrity, shall be designated demand critical. All welding of new elements, added to resist seismic forces, shall conform to AWS D1.8/D1.8M and shall be designated as demand critical.
CHAPTER C

COMPONENT PROPERTIES AND REQUIREMENTS

This chapter addresses the stiffness and strength of steel components 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 steel components are given for three structural performance levels, as defined in ASCE/SEI 41, Chapter 2, as follows, 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 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, component stiffnesses shall represent all phenomena specific to that component, 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.

For the nonlinear static procedure, when constructing the nonlinear force-deformation model, the force-deformation behavior of a component, as depicted in Figure C1.1 for Type 1 response as defined in ASCE/SEI 41, Section 7.5, shall be determined in accordance with this section. Alternatively, this model may be derived from testing or analysis in accordance with ASCE/SEI 41, Section 7.6.

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.

For component actions that exhibit a total deformation at Point C greater than two times the yield deformation, the post-elastic hardening slope, $\alpha_h$, is the ratio of the inelastic stiffness to the elastic stiffness, and it can be positive or negative. If $\alpha_h$ is negative, then the peak inelastic strength is less than the yield strength.

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

For the nonlinear dynamic procedure, the complete hysteretic behavior of a component shall be determined.
by testing or analysis, or by other procedures approved by the authority having jurisdiction. If test data are
not available for the formulation of component behavior, it is permitted to use the component force-
deformation parameters described in this Chapter for modeling the force-deformation behavior, in other
words, the backbone curve, and applying hysteretic rules for corresponding component actions. When
constructing a backbone curve from test data in accordance with ASCE/SEI 41, Section 7.6, the hysteretic
load and deformation curve shall not cross beyond the backbone curve. The characteristics of the hysteretic
loops, including post-elastic hardening slope, $\alpha_h$, cyclic stiffness degradation in unloading and reloading,
cyclic strength degradation, and in-cycle strength degradation, shall be represented in the response model.
If cyclic degradation slopes vary for a group of similar components, then the response model shall be
constructed as the best fit to the data for the class of components.
C2. BEAMS

1. General

The component characteristics of steel \textit{and composite steel-concrete} beams subjected to seismic forces or deformations from flexural and/or shear actions with no 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 determined in accordance with ASCE/SEI, Section 7.5.2.1.2, $P_{CF}$, does not exceed 10% of the nominal axial strength, $P_n$, determined in accordance with \textit{Specification}, Chapters D or E, as applicable.

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 shear supports that resist translation in the direction of the shear force, $L_v$, of the beam is greater than $2.6 \frac{M_{CE}}{V_{CE}}$, 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 $1.6 \frac{M_{CE}}{V_{CE}}$, the beam shall be designated as shear-controlled. Otherwise, for lengths of $L_v$ between $1.6 \frac{M_{CE}}{V_{CE}}$ and $2.6 \frac{M_{CE}}{V_{CE}}$, 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.

User Note: Beams with a calculated axial load equal to or exceeding 10% of the expected axial yield strength of the section, $P_{yw}$, shall be evaluated as a column or brace in accordance with Section C3.

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 \textit{Specification} unless superseded by supplemental provisions of this section or system-specific sections in Chapters D through I.

The force-deformation model for a \textit{steel} beam shall account for all significant sources of deformations (e.g., axial, flexural, and shear) that affect its behavior.

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 equivalent width of the concrete floor slab, as permitted in \textit{Specification} Section I3.1a, can 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, as stipulated in ASCE/SEI 41, Chapter 2.

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.

2.2c. Shear Stiffness

The elastic shear stiffness, $K_e$, of beams shall be determined from the analytical model or another rational method.

For composite beams, the shear stiffness shall be taken as that of the steel section alone, unless otherwise justified by rational analysis. When not explicitly included in the structural analysis, the elastic shear stiffness, $K_e$, of the beam is permitted to be determined from Equation C2-1, unless justified by a rational analysis.

$$K_e = \frac{12EI}{L^3(1+\eta)}$$  \hspace{1cm} (C2-1)

where:

- $A_s$ = effective shear area of the cross section, in.\(^2\) (mm\(^2\))
- $E$ = modulus of elasticity of steel = 29,000 ksi (200,000 MPa)
- $G$ = shear modulus of elasticity of steel = 11,200 ksi (77,200 MPa)
- $I$ = moment of inertia about the axis of bending, in.\(^4\) (mm\(^4\))
- $L_v$ = length of beam between shear supports in the direction of the shear force, in. (mm)
- $d_b$ = depth of beam, in. (mm)
- $t_w$ = thickness of web, in. (mm)
- $\eta = \frac{12EI}{L_v^2GA_t}$  \hspace{1cm} (C2-2)

3. Strength

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

3a. Deformation-Controlled Action

1. Expected Flexural Strength

The expected flexural strength, $M_{CE}$, of a steel beam 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 and be sufficiently braced laterally to develop the expected plastic flexural strength, $M_{pl}$, of the section given in Specification Chapter Section F2.1, except that $F_{ye}$ shall be substituted for $F_y$. In this case, the expected component strength, $Q_{CE} = Q_{y} = M_{CE}$, otherwise $Q_{CE} < Q_{y}$.
where $Q_y$ is the expected component yield strength.

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

2. Expected Shear Strength

The expected shear strength, $V_{CE}$, of a steel beam 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 $Q_{CE} = V_{CE}$.

For beams expected to experience inelastic action through shear yielding, the yielding zone shall be sufficiently stiffened or the web shall have adequate compactness to prevent shear buckling before shear yielding. In this case, the expected shear strength shall be determined using equations for nominal strength, $V_n$, given in Seismic Provisions Section F3, except that $F_{yLB}$ shall be substituted for $F_y$, and $Q_{CE} = Q_y = V_{CE}$, 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, $M_{CL}$, of a steel beam shall be determined using equations for nominal strength, $M_n$, given in Specification Chapter F, except that the lower-bound yield strength determined in accordance with Section Chapter A, $F_{yLB}$, shall be substituted for $F_y$, and the the lower-bound component strength, $Q_{CL} = M_{CL}$, For the limit state of shear yielding, $M_{CL}$ shall not be taken greater than $V_{CL} L_e / 2$, or as required by analysis based on support conditions, where the lower-bound shear strength, $V_{CL}$ is computed determined in accordance with Section C2.3b.2.

2. Lower-bound Shear Strength

The lower-bound shear strength, $V_{CL}$, of a steel beam shall be determined using equations for nominal strength, $V_n$, given in Specification Chapter G, except that $F_{yLB}$ 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 steel 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 of a beam is considered
deformation-controlled, the flexural behavior shall be evaluated using Equation 7-36 of ASCE/SEI 41 with the expected flexural strength, \( Q_{CE} = M_{CE} \), determined in accordance with Section C2.3a.1 and \( m \) 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, \( m_e \), determined from Equation C2-31.

\[
m_e = m - (m-1) \left[ \frac{M_{pe} - M_{CE}}{M_{pe} - (0.7F_{ye}) S} \right] \geq 1.0 \tag{C2-31}
\]

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 Sections C2.4a.2.

For beams fully encased in concrete where confining reinforcement is provided to ensure that the concrete remains in place during an earthquake 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, \( m \).

---

### TABLE C2.1
Component Capacity Modification Factor, \( m \), for Linear Analysis Procedures—Beams Subjected to Flexure \(^{a,b}\)

<table>
<thead>
<tr>
<th>Section Compactness (^c)</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IO</td>
<td>LS</td>
</tr>
<tr>
<td>( a ) Highly ductile ((\lambda \leq \lambda_{hd}))</td>
<td>2</td>
<td>6</td>
</tr>
<tr>
<td>( b ) Non-moderately ductile ((\lambda &gt; \lambda_{md}))</td>
<td>1.25</td>
<td>2</td>
</tr>
<tr>
<td>( c ) Other</td>
<td>Linear interpolation between the values on lines ( a ) and ( b ) for flange, wall and web slenderness shall be performed, and the lowest resulting value shall be used.</td>
<td></td>
</tr>
</tbody>
</table>

\(^a\) Regardless of the modifiers applied, \( m \) need not be taken less than 1.0.

\(^b\) Values tabulated are applicable for flexure-controlled beams with \( L_i \geq 2.6 M_{CE} / V_{CE} \). Linearly interpolate values to 1.0 when \( L_i \leq 1.6 M_{CE} / V_{CE} \). Values of \( m \) shall be 1.0 when \( L_i \leq 1.6 M_{CE} / V_{CE} \) or \( 1.6 M_{CE} / V_{CE} < L_i < 2.6 M_{CE} / V_{CE} \). \( m \) shall be linearly interpolated between the tabulated values and 1.0.

\(^c\) The limiting slenderness parameters for highly and moderately ductile compression elements, \( \lambda_{hd} \) and \( \lambda_{md} \), respectively, are defined in Seismic Provisions Table D1.1, with \( R_y F_y \) replaced by \( F_{ye} \).
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 Section E1.4. 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 experiment testing or analysis. For beams, it is permitted to take a strain hardening slope $\alpha_h$ for flexural action of $\alpha_h$ 3% of the elastic slope is permitted. Further modification of the curve is permitted if a greater strain hardening slope value for $\alpha_h$ is justified by experiment testing or analysis.

When the flexural behavior of a beam is considered deformation-controlled, the plastic chord rotation, $\theta_p$, 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, $\theta_y$, of the beam shall be determined from Equation C2-42. Otherwise, if the beam is shear-controlled or shear-flexure-controlled, $\theta_y$ shall be taken as the shear yield deformation, $\gamma_y$, determined from Section C2.4a.2.b.

$$\theta_y = \frac{M_{CF}L_{CL} (1 + \eta)}{6EI} \quad \text{ (C2-42)}$$

where

- $A_s$ = effective shear area of the cross section, in.$^2$ (mm$^2$) [for a wide-flange section in strong-axis bending, $A_s = d_b t_w$]
- $E$ = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
- $G$ = shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)
- $I$ = moment of inertia about the axis of bending, in.$^4$ (mm$^4$)
- $L_{CL}$ = centerline length of beam taken between joints, in. (mm)
- $d_b$ = depth of beam, in. (mm)
- $t_w$ = thickness of web, in. (mm)
- $\eta = \frac{12EI}{L_{CL} G A_t}$ \quad \text{ (C2-23)}

User Note: Equation C2-42 is based on a beam that is rotationally restrained at both ends with no end zones and assumes that the effects from transverse loads are negligible. Therefore, the inflection point is located at mid-span. When other end conditions and/or transverse loadings may shift the inflection point, $\theta_y$ should be determined by analysis; see Commentary for more information.

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 $\eta$ as zero.

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
User Note: Shear deformation (accounted for by $\eta$) in a flexure-controlled beam with a length greater than $10M_{CE} / V_{CE}$ is generally small and can be neglected in Equation C2-42.

If $M_{CE} < M_{pe}$, then the values in Table C3.4 shall be multiplied by the factor, $\chi$, determined from Equation C2-54.

$$\chi = \left[ 1 - \frac{M_{pe} - M_{CE}}{M_{pe} - (0.7F_{ye})S} \right] \geq 0$$  (C2-54)

### TABLE C2.2

**Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Beams Subjected to Flexure**

<table>
<thead>
<tr>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic chord rotation angle $a$ and $b$, rad</td>
<td>Plastic chord rotation angle, rad</td>
</tr>
<tr>
<td>Residual strength ratio $c$</td>
<td>IO</td>
</tr>
</tbody>
</table>

**Section Compactness**

- **a1**. Highly ductile ($\lambda \leq \lambda_{md}$)
  - $a = 9\theta_y$
  - $b = 11\theta_y$
  - $c = 0.6$
  - 0.25$a$ | $a$ | $b$

- **b2**. Non-moderately ductile ($\lambda > \lambda_{md}$)
  - $a = 4\theta_y$
  - $b = 6\theta_y$
  - $c = 0.2$
  - 0.25$a$ | 0.75$a$ | $a$

- **c3**. Other
  - Linear interpolation between the values on lines $a$ and $b$ for flange, wall and web slenderness shall be performed, and the lower resulting value shall be used.

*Values tabulated values are applicable for flexure-controlled beams with $L_v \geq 2.6 M_{CE} / V_{CE}$. Linearly interpolate values to 0.0 when $L_v \leq 1.6 M_{CE} / V_{CE}$. Values shall be taken as 0.0 when $L_v \leq 1.6 M_{CE} / V_{CE}$. For 1.6 $M_{CE} / V_{CE} < L_v < 2.6 M_{CE} / V_{CE}$, values shall be linearly interpolated between the tabulated values and 0.0.*

*The limiting width-to-thickness ratios, $\lambda_{fl}$ and $\lambda_{wb}$, are defined in *Seismic Provisions* Table D1.1, with $R_y F_y$ replaced by $F_{ye}$.

### 2. Shear Actions

#### a. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior of a beam is considered deformation-controlled, the shear behavior shall be evaluated using ASCE/SEI 41, Equation 7-36, with the expected shear strength, $Q_{CE} = V_{CE}$, determined in accordance with Section C2.3a.2 and $m$ taken from Table C2.3.
### TABLE C2.3
Component Capacity Modification Factor, \( m \), for Linear Analysis Procedures—Beams Subjected to Shear \(^{a,b,c,d}\)

<table>
<thead>
<tr>
<th>Length of Beam, ( L_v )</th>
<th>( m )</th>
<th>\begin{array}{lll} \text{Primary Component} &amp; \text{Secondary Component} \ \text{LS} &amp; \text{CP} &amp; \text{LS} &amp; \text{CP} \ \end{array}</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L_v \leq \frac{1.6 M_{ce}}{V_{ce}} ) (Shear-Controlled)</td>
<td>1.5</td>
<td>9</td>
</tr>
<tr>
<td>( L_v \geq \frac{2.6 M_{ce}}{V_{ce}} ) (Flexure-Controlled)</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>( \frac{1.6 M_{ce}}{V_{ce}} &lt; L_v &lt; \frac{2.6 M_{ce}}{V_{ce}} ) (Shear-Flexure-Controlled)</td>
<td>Linear interpolation between the values on lines ( a ) and ( b ) shall be performed</td>
<td></td>
</tr>
</tbody>
</table>

\(^{a}\) 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 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 Tables C2.4, shall be used for beams. Alternatively, these relationships are permitted to be derived from experiment testing or analysis.

For beams, it is permitted to take \( \alpha_h \), a strain-hardening slope for shear action, as 6% of the elastic slope is permitted. Further modification of the curve is permitted if a greater strain-hardening slope value for \( \alpha_h \) is justified by experiment testing or analysis.

When the shear behavior of a beam is considered deformation-controlled, the plastic shear deformation, \( \gamma_p \), predicted by analysis shall be not greater than the permissible plastic shear deformation provided in Table C2.4 for a given performance level. The shear yield deformation, \( \gamma_y \), of a beam shall be determined from Equation C2.65.

\[
\gamma_y = \frac{V_{ce}}{K_e L_v} 
\]  

(C2.65)

where

\[
L_v = \text{clear length between supports that resist translation in the direction of the shear force, in (mm)}
\]

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

\[
K_e = \text{elastic shear stiffness of the beam determined in accordance with Section C2.2b}
\]
TABLE C2.4
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Beams Subjected to Shear

<table>
<thead>
<tr>
<th>Length of Beam, ( L_v )</th>
<th>Modeling Parameters</th>
<th></th>
<th>Expected Deformation Capacity</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Shear Deformation, rad</td>
<td>Residual Strength Ratio</td>
<td>Plastic Shear Deformation, rad</td>
<td></td>
</tr>
<tr>
<td>( L_v \leq \frac{1.6 M_{CE}}{V_{CE}} ) (Shear-Controlled)</td>
<td>0.15</td>
<td>0.17</td>
<td>0.8</td>
<td>0.005</td>
</tr>
<tr>
<td>( L_v &gt; \frac{2.6 M_{CE}}{V_{CE}} ) (Flexure-Controlled)</td>
<td>0.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \frac{1.6 M_{CE}}{V_{CE}} &lt; L_v &lt; \frac{2.6 M_{CE}}{V_{CE}} ) (Shear-Flexure-Controlled)</td>
<td>Linear interpolation between the values on lines a and b</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Deformation is the rotation angle between the beam and column or portion of beam outside the shear yielding zone.
* 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.
* Assumes ductile detailing for beam in the shear yielding zone in accordance with the Seismic Provisions.
* Values shall be taken as 0.0 when \( L_v \geq 2.6 M_{CE} / V_{CE} \). For \( 1.6 M_{CE} / V_{CE} < L_v < 2.6 M_{CE} / V_{CE} \), values shall be linearly interpolated between the tabulated values and 0.0.

4b. Force-Controlled Actions

1. Flexural Action

   a. Linear Analysis Procedures

   When linear analysis procedures are used and the flexural behavior of a beam is considered force-controlled, the flexural behavior shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound flexural strength, \( Q_{CL} = M_{CL} \), determined in accordance with Section C2.3b.1.

b. Nonlinear Analysis Procedures

   When nonlinear analysis procedures are used and the flexural behavior of a beam is considered...
force-controlled, the flexural behavior shall be evaluated using ASCE/SEI 41, Equation 7.38, with the lower-bound flexural strength, $Q_{CL} = M_{CL}$, determined in accordance with Section C2.3b.1.

Alternatively, when a force-controlled action, for the beam, is explicitly modeled with a nonlinear force-deformation behavior, it is permitted to evaluate the beam based on deformation. For such an evaluation, the When nonlinear analysis procedures are used and the flexural behavior of a beam is considered force-controlled, total chord rotation, $\theta$, of the beam predicted by analysis shall not exceed $\theta_y$ determined from Equation C2-42.

The lower-bound flexural strength, $Q_{CL} = M_{CL}$, determined in accordance with Section C2.3b.1 shall not be less than the maximum force determined by ASCE/SEI 41, Section 7.5.3.2.3.

2. Shear Action

a. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior of a beam is considered force-controlled, the shear behavior shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound shear strength, $Q_{CL} = V_{CL}$, determined in accordance with Section C2.3b.2.

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the shear behavior of a beam is considered force-controlled, the shear behavior shall be evaluated using ASCE/SEI 41, Equation 7.38, with the lower-bound shear strength, $Q_{CL} = V_{CL}$, determined in accordance with Section C2.3b.2.

Alternatively, when a force-controlled action, for the beam, is explicitly modeled with a nonlinear force-deformation behavior, it is permitted to evaluate the beam based on deformation. When nonlinear analysis procedures are used and the shear behavior of a beam is considered force-controlled, For such an evaluation, total shear deformation, $\gamma$, of the beam predicted by analysis shall not exceed $\gamma_y$ determined from Equation C2-65.

The lower-bound shear strength, $Q_{CL} = V_{CL}$, determined in accordance with Section C2.3b.2 shall not be less than the maximum force determined by ASCE/SEI 41, Section 7.5.3.2.3.
C3. MEMBERS SUBJECTED TO AXIAL OR COMBINED LOADING

1. General

The component characteristics of steel and composite steel-concrete columns or braces, 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 \( P_{UF} \), determined in accordance with ASCE/SEI, Section 7.5.2.1.2, equals or exceeds 10% of \( P_n \), determined in accordance with Specification, Chapters D or E, as applicable.

User Note: Beams with a calculated axial load \( P_{UF} \) equal to or exceeding 10% of the expected axial yield strength of the section, \( P_{ye} \), \( P_n \) should be evaluated as a column or brace in accordance with Section C3. Most beams in braced frames meet this requirement.

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

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

Deformation-controlled braces that buckle in compression shall use the modified version of the generalized force-deformation relation in Figure C3.1 for both the compressive and tensile response. This modified 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.

![Fig. C3.1. Generalized force-deformation relation for buckling steel braces and their connections.](image-url)
2. Stiffness

The stiffness of steel 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 frame-specific sections in Chapters D through I.

The force-deformation model for a column or brace shall include all significant sources of deformation, including axial, flexural and shear, as applicable. The force-deformation model for a brace shall be consistent with the requirements of Section E1.

2a. Axial Stiffness

For buckling braces, the axial stiffness shall be modeled using the specific requirements for linear and nonlinear analysis in Section E2. Elastic stiffness of a buckling brace shall be calculated or modeled considering 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 consider only the steel brace 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 as 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 shall be permitted to be considered adequately confined to provide full composite action.

2b. Flexural Stiffness

The flexural stiffness of a column or brace, $EI_c$, with $P > 0.5P_{ye}$, shall be modified by $\tau_P$, the stiffness reduction parameter, as given in Specification Chapter C,

\[
\tau_P = \frac{1}{2} \left( \frac{P}{A_gF_{ye}} \right)
\]

where

- $A_g$ = gross area of the cross section, in.$^2$ (mm.$^2$)
- $I_c$ = moment of inertia of a column or brace about the axis of bending, in.$^4$ (mm.$^4$)
- $P$ = axial force, kips (N)
- $P_{ye} = A_gF_{ye}$ = expected axial yield strength, kips (N)
- $A_{gross}$ = gross area of the cross section, in.$^2$ (mm.$^2$)
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**

The elastic shear stiffness, $K_e$, of members subject to axial or combined loading shall be determined from the analytical model or another rational method.

For composite members, the shear stiffness shall be taken as that of the steel section alone, unless otherwise justified by rational analysis. When not explicitly included in the structural analysis, the elastic shear stiffness, $K_e$, of a column or brace is permitted to be determined from Equation C2-1, modified to include the effect of an axial load, unless otherwise justified by a rational analysis.

### 3. Strength

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

#### 3a. Deformation-Controlled Action

##### 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, $L_e$, for calculation of member slenderness, $L_e/r$, shall be determined using the end-to-end brace length, $L_{ee}$.

$$K = \text{effective length factor}$$

$$L_e = KL_{ee} = \text{effective length, in. (mm)}$$

$$r = \text{radius of gyration, in. (mm)}$$

The length of a brace shall be taken as the distance between brace ends.

For buckling braces, the effective length factor, $K$, shall be taken as follows:

(a) For braces with rotation-restrained end connections, as defined in Section C7, $K = 0.65$.

(b) For rotation-accommodating connections, as defined in Section C7, $K = 1.0$.

(c) For intersecting braces in X-braced frames, $K = 0.7$.

Other values of $K$ are permitted to be justified by analysis.

The expected tensile strength, $T_{CE}$, of a steel column or brace shall be determined using equations for
nominal axial strength, \( P_n \), given in Specification Chapter D, except that \( F_{ye} \) shall be substituted for \( F_y \).

The expected compressive and tensile strength for a buckling-restrained brace, \( P_{CE} \), 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 A3.1. 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 BRB casing system, connections, and adjoining members 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 \( \omega P_{CE} \). Factors \( \beta \) and \( \omega \) 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 of a steel column or brace shall be determined in accordance with Section C2.3a.1.

For columns or braces expected to exhibit nonlinearity through flexural yielding, the expected flexural strength, \( M_{CE} \), of the column or brace, or its cross section at the hinge locations, shall be determined from Equation C3-1 as follows:

\[
\begin{align*}
M_{CE} &= M_{pe} \\
&= M_{pe} \left( 1 - \frac{|P|}{2P_{pe}} \right) \quad (C3-1)
\end{align*}
\]

When \( \frac{|P|}{P_{pe}} < 0.2 \)

\[
M_{CE} = M_{pe} \left( 1 - \frac{|P|}{2P_{pe}} \right)
\]

When \( \frac{|P|}{P_{pe}} \geq 0.2 \)

\[
M_{CE} = M_{pe} \left( 1 - \frac{|P|}{8P_{pe}} \right)
\]

\( 2020 \text{ Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings} \\
\text{Draft Ballot 3 Dated September 30, 2019} \\
\text{AMERICAN INSTITUTE OF STEEL CONSTRUCTION} \)
$M_{CE} = M_{pe} - \begin{cases} 
|P| < 0.2 & \frac{|P|}{P_{ce}} \left( 1 - \frac{|P|}{2P_{ce}} \right) \\
|P| \geq 0.2 & \frac{P}{P_{ce}} \left( 1 - \frac{|P|}{P_{ce}} \right) 
\end{cases}$ (C3-1)

where

$F_{ye}$ = expected yield stress, ksi (MPa)

$M_{pe}$ = expected plastic flexural strength, determined in accordance with Section C2.3a.1, kip-in. (N-mm)

$M_{pce}$ = expected plastic flexural strength reduced for the effect of axial force (compression or tension), kip-in. (N-mm)

$P$ = axial force (compression or tension), kips (N)

User Note: When 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 of a steel column or brace shall be determined in accordance with Section C2.3a.2.

For columns or braces expected to exhibit nonlinearity through shear yielding of the web, the expected shear strength, $V_{CE}$, along the yielding zone shall be determined from Equation C3-2 as follows:

When $|P| < 0.2$

$V_{CE} = V_{pe} - V_{pe}$ (C3-3)

When $|P| > 0.2$

$V_{CE} = V_{pe} = V_{pe} \sqrt{1 - \left( \frac{|P|}{P_{pe}} \right)^2}$ (C3-4)

\begin{align*}
&\left\{ \begin{array}{ll}
&|P| \leq 0.2 \quad V_{pe} \\
&|P| > 0.2 \quad V_{pe} \sqrt{1 - \left( \frac{|P|}{P_{pe}} \right)^2}
\end{array} \right.
\end{align*} (C3-2)

where

$V_{pe}$ = nominal shear strength, $V_{pe}$, in the absence of axial force, from Seismic Provisions Section F3, with $F_{pe}$ substituted for $F_y$, kips (N)
3b. Force-Controlled Action

1. Lower-Bound Axial Strength

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

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

2. Lower-Bound Flexural Strength

The lower-bound flexural strength, $M_{CL}$, of a steel 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 steel 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 steel 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 of a column or brace is considered deformation-controlled, the axial behavior shall be evaluated using ASCE/SEI 41, Equation 7-36, with the expected component strength, $Q_{CE} = P_{CE}$, determined in accordance with Section C3.3a.1 and $m$ taken from Table C3.1 or Table C3.2, as appropriate.
### TABLE C3.1
Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—Columns and Buckling-Restrained Braces Subjected to an Axial Force

<table>
<thead>
<tr>
<th>Component</th>
<th>IO</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>1. Columns in Tension</td>
<td>1.25</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>2. Buckling-Restrained Braces $^{a, b}$</td>
<td>2.3</td>
<td>5.6</td>
<td>7.5</td>
</tr>
</tbody>
</table>

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 buckling-restrained brace (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.

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

<table>
<thead>
<tr>
<th>Component</th>
<th>IO $^{c, d, e}$</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>Braces in Compression $^{c, d, e}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. W, I, 2L in-plane, 2C in-plane</td>
<td>1.25</td>
<td>0.5$n$</td>
<td>0.75$n$</td>
</tr>
<tr>
<td>$n = 5.6 \left( \frac{\lambda}{\lambda_{ho}} \right)^{-1.7} \left( \frac{L}{f_{y}r_{p}} \right)^{0.45}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>b2. 2L out-of-plane, 2C out-of-plane</td>
<td>1.25</td>
<td>0.5$n$</td>
<td>0.75$n$</td>
</tr>
<tr>
<td>$n = 4.7 \left( \frac{\lambda}{\lambda_{ho}} \right)^{-1.7} \left( \frac{L}{f_{y}r_{p}} \right)^{0.45}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>c3. Rectangular HSS</td>
<td>1.25</td>
<td>0.5$n$</td>
<td>0.75$n$</td>
</tr>
<tr>
<td>$n = 3.0 \left( \frac{\lambda}{\lambda_{ho}} \right)^{1.0} \left( \frac{L}{f_{y}r_{p}} \right)^{1.0}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>d4. Round HSS and pipe</td>
<td>1.25</td>
<td>0.5$n$</td>
<td>0.75$n$</td>
</tr>
<tr>
<td>$n = 4.7 \left( \frac{\lambda}{\lambda_{ho}} \right)^{-1.7} \left( \frac{L}{f_{y}r_{p}} \right)^{0.45}$</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

*2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*

*Draft Ballot 3 Dated September 30, 2019*

*American Institute of Steel Construction*
TABLE C3.2
Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—Buckling Braces Subjected to an Axial Force

<table>
<thead>
<tr>
<th>Component</th>
<th>IO</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>a. Single angle (L)</td>
<td>$n = 12 \left( \frac{A}{A_{eq}} \right)^{1.7}$</td>
<td>1.25</td>
<td>0.5$n$</td>
</tr>
</tbody>
</table>
| a. W, L, 2L in-plane, 2C in-plane | $n = 3.6 \left( \frac{A}{A_{eq}} \right)^{1.7} L/c \left( \frac{E} {2
\nu P_{y}} \right)^{1.4}$ | 1.25 | 0.5$n$ | 0.75$n$ | 0.6$n$ | 0.9$n$ |
| b. 2L out-of-plane, 2C out-of-plane | $n = 2.0 \left( \frac{\lambda}{\lambda_{hd}} \right)^{1.7} L/c \left( \frac{E} {2\nu P_{y}} \right)^{0.45}$ | 1.25 | 0.5$n$ | 0.75$n$ | 0.6$n$ | 0.9$n$ |
| c. Rectangular HSS | $n = 4.7 \left( \frac{A}{A_{eq}} \right)^{1.7} L/c \left( \frac{E} {2\nu P_{y}} \right)^{0.24}$ | 1.25 | 0.5$n$ | 0.75$n$ | 0.6$n$ | 0.9$n$ |
| d. Round HSS and pipe | $n = 2.0 \left( \frac{\lambda}{\lambda_{hd}} \right)^{1.7} L/c \left( \frac{E} {2\nu P_{y}} \right)^{0.45}$ | 1.25 | 0.5$n$ | 0.75$n$ | 0.6$n$ | 0.9$n$ |
| e. Single angle (L) | $n = 7.2 \left( \frac{A}{A_{eq}} \right)^{1.7}$ | 1.25 | 0.5$n$ | 0.75$n$ | 0.6$n$ | 0.9$n$ |

$A_{eq}$ = $A_{eq}$ effective length for buckling, in. (mm)
$A$ = cross-sectional area, in.² (mm²)
$E$ = modulus of elasticity, ksi (MPa)
$\nu$ = Poisson’s ratio
$P_{y}$ = yield load, kips (N)
$L/c$ = ratio of gusset plate thickness, in. (mm)
$\lambda_{hd}$ = slenderness ratio of the element, as defined in the Seismic Provisions.

Where HSS or pipe braces are filled with concrete and $\lambda/\lambda_{hd}$ is less than or equal to 2.5; $\lambda/\lambda_{hd}$ need not exceed 1.0 for computing $n$.

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.

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

In addition to consideration of connection design strength in accordance with Section E1.4, values for bracing shall be modified for connection robustness using $n_p$ per Section C7.

The limiting slenderness parameters for highly and moderately ductile compression elements, $\lambda_{hd}$ and $\lambda_{m}$, respectively, are defined in Seismic Provisions Table G11.1, with $P_{y}$ replaced by $P_{y}\nu$.

The component modification factor, $m$, for IO shall not exceed $m$ for LS.

The component modification factors for highly and moderately ductile compression elements, $m_{h}$ and $m_{m}$, respectively, are defined in Seismic Provisions Table G11.1, with $P_{y}$ replaced by $P_{y}\nu$.

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, a strain-hardening slope is permitted to take $a_h$ for tension action of as 3% of the elastic slope is permitted. Further modification of the curve is permitted if a greater strain-hardening slope value for $a_h$ is justified by testing or analysis.

User Note: For a buckling-restrained brace, point C in Figure C1.1 is $\omega Q_y$ for tension and $\beta \omega Q_y$ for compression. Refer to Section C3.3a.1 and the Seismic Provisions to determine the compression strength adjustment factor $\beta$ and the strain-hardening adjustment factor $\omega$.

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, $\Delta_p$, 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, $\Delta_y$, of a column or brace shall be determined as follows:

For tension:

$$\Delta_y = \Delta_T = \frac{P_{EC} L_c}{E A_T}$$  \hspace{1cm} (C3-35)

For compression:

$$\Delta_y = \Delta_C = \frac{P_{EC} L_c}{E A_C}$$  \hspace{1cm} (C3-46)

For buckling-restrained braces:

$$\Delta_y = \Delta_T + \frac{P_{EC} L_{conn}}{E A_{conn}} + \frac{P_{EC} L_{core}}{E A_{core}}$$  \hspace{1cm} (C3-57)

where

- $A_{conn} =$ cross-sectional area of BRB connection, in.$^2$ (mm$^2$)
- $A_{core} =$ cross-sectional area of BRB core, in.$^2$ (mm$^2$)
- $L_c =$ length of column or brace between supports, in. (mm)
- $L_{conn} =$ length of BRB connection, in. (mm)
- $L_{core} =$ length of BRB core, in. (mm)
- $\Delta_C =$ axial deformation at expected compressive buckling strength, in. (mm)
- $\Delta_T =$ axial deformation at expected tensile yield strength, in. (mm)

User Note: The term $Q_y$, and associated, $\Delta_y$, 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.
### TABLE C3.3
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Columns and Buckling-Restrained Braces Subjected to Axial Force

<table>
<thead>
<tr>
<th>Component</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Axial Deformation, in. (mm)</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td>Columns in Tension</td>
<td>a 5ΔT</td>
<td>b 7ΔT</td>
</tr>
<tr>
<td>Buckling-Restrained Braces</td>
<td>c 13.3Δy</td>
<td>b 13.3Δy</td>
</tr>
</tbody>
</table>

* Maximum strain of the buckling-restrained brace core shall not exceed 2.5%.
* If testing to demonstrate compliance with Section E3.4 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

<table>
<thead>
<tr>
<th>Component</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Axial Deformation, in. (mm)</td>
<td>Strength Ratio at Maximum Deformation</td>
</tr>
<tr>
<td>Braces in Compression</td>
<td>d</td>
<td>f</td>
</tr>
<tr>
<td><strong>a₁</strong> W, I, 2L in-plane, 2C in-plane</td>
<td>n = 5.6(ΔΔ₀/Δ₀)⁻¹.⁷(Δ₀/Δ₀)⁰.₈₈</td>
<td>nΔ₀₀₀</td>
</tr>
<tr>
<td><strong>a₂</strong> 2L out-of-plane, 2C out-of-plane</td>
<td>n = 4.7(ΔΔ₀/Δ₀)⁻¹.⁷(Δ₀/Δ₀)⁰.₄₅</td>
<td>nΔ₀₀₀</td>
</tr>
<tr>
<td><strong>a₃</strong> Rectangular HSS braces</td>
<td>nΔ₀₀₀</td>
<td></td>
</tr>
</tbody>
</table>

---

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
American Institute of Steel Construction
\[
n = 3.0 \left( \frac{\lambda}{\lambda_{\text{hd}}} \right)^{-1.7} \left( \frac{L_c/f_c}{E/F_y} \right)^{1.0}
\]

### a5. Round HSS and pipe braces

\[
n = 4.7 \left( \frac{\lambda}{\lambda_{\text{hd}}} \right)^{-1.7} \left( \frac{L_c/f_c}{E/F_y} \right)^{0.45}
\]

### a6. Single angle (L)

\[
n = 12 \left( \frac{\lambda}{\lambda_{\text{hd}}} \right)^{-1.7}
\]

#### Braces in tension

<table>
<thead>
<tr>
<th>Type</th>
<th>Formula</th>
</tr>
</thead>
</table>
| a1. W, I, 2L in-plane, 2C in-plane | \[
n = 3.4 \left( \frac{\lambda}{\lambda_{\text{hd}}} \right)^{-1.7} \left( \frac{L_c/f_c}{E/F_y} \right)^{0.4}
\] |
| b2. 2L out-of-plane, 2C out-of-plane | \[
n = 2.8 \left( \frac{\lambda}{\lambda_{\text{hd}}} \right)^{-1.7} \left( \frac{L_c/f_c}{E/F_y} \right)^{0.45}
\] |
| c3. Rectangular HSS braces | \[
n = 4.7 \left( \frac{\lambda}{\lambda_{\text{hd}}} \right)^{-1.7} \left( \frac{L_c/f_c}{E/F_y} \right)^{0.24}
\] |
| d4. Round HSS and pipe braces | \[
n = 2.8 \left( \frac{\lambda}{\lambda_{\text{hd}}} \right)^{-1.7} \left( \frac{L_c/f_c}{E/F_y} \right)^{0.45}
\] |
| e5. Single angle (L) | \[
n = 7.2 \left( \frac{\lambda}{\lambda_{\text{hd}}} \right)^{-2.8}
\] |

\( \lambda_{\text{hd}} \) = effective length for buckling in \( \text{mm} \)
\( r \) = radius of gyration about axis of bending \( \text{mm} \)
\( \lambda = \) local slenderness (i.e., width-to-thickness ratio) as defined in the Seismic provisions

The limiting width-to-thickness ratio, \( \lambda_{\text{hd}} \), is defined in Seismic Provisions Table D1.1, with \( R_e \) replaced by \( F_y \). For concrete-filled HSS or pipe braces, \( \lambda_{\text{hd}} \) shall be determined for the corresponding hollow section.

- 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.
- Where HSS or pipe braces are filled with concrete and \( \lambda_{\text{hd}} \) is less than or equal to 2.5, \( \lambda_{\text{hd}} \) need not exceed 1.0 for computing \( n \).
- Connectors for built-up members: Where the connectors for built-up braces do not satisfy the requirements of Seismic...
2. Flexural Action

a. Linear Analysis Procedures Concurrent with Axial Action

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 steel columns and braces under combined axial and bending stress, the 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 C3-6 or C3-7. If the column or brace is in compression, values for \( m \) shall be taken from Table C3.5. If the column or brace is in tension, values for \( m \) shall be taken from Table C3.5, line a, and the compactness requirements shall be neglected.

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.

---

**TABLE C3.5**

Component Capacity Modification Factor, \( m \), for Linear Analysis Procedures—Columns and Braces Subjected to Flexure with Axial Compression or Tension\(^{a,b}\)

<table>
<thead>
<tr>
<th>Axial Load Ratio and Section Compactness</th>
<th>IO</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>For ( P_{UB}/P_{y} &lt; 0.2 )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( a ) highly ductile (( L \leq L_{ph} ))</td>
<td>2</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>( b ) non-moderately</td>
<td>1.25</td>
<td>1.25</td>
<td>2</td>
</tr>
</tbody>
</table>

---

\( ^{a} \) For tension-only bracing, the values shall be divided by 2.0.

\( ^{b} \) In addition to consideration of connection design strength in accordance with Section E1.4, values for braces shall be modified for connection robustness per Section C7.

\( ^{c} \) The limiting width-to-thickness ratio, \( \lambda_{hd} \), is defined in Seismic Provisions Table D1.1, with \( R_y F_y \) replaced by \( F_{ye} \). For concrete-filled HSS or pipe braces, \( \lambda_{hd} \) shall be determined for the corresponding hollow section.

\( ^{d} \) 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,b

<table>
<thead>
<tr>
<th>Axial Load Ratio and Section Compactness</th>
<th>IO</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>ductile ($\lambda &gt; \lambda_{md}$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>For $0.6 \geq P_{rel} P_{rel} \geq 0.2$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\lambda_{hd}$, highly ductile ($\lambda \leq \lambda_{md}$)</td>
<td></td>
<td>1.5 (1 - 0.5) $P_{rel} P_{rel}$</td>
<td>7.5 (1 - 0.5) $P_{rel} P_{rel}$</td>
</tr>
<tr>
<td>$\lambda_{nm}$, non-moderately ductile ($\lambda &gt; \lambda_{md}$)</td>
<td></td>
<td>0.375 (1 - 0.5) $P_{rel} P_{rel}$</td>
<td>0.375 (1 - 0.5) $P_{rel} P_{rel}$</td>
</tr>
<tr>
<td>Other</td>
<td></td>
<td>4.5 (1 - 0.5) $P_{rel} P_{rel}$</td>
<td>4.5 (1 - 0.5) $P_{rel} P_{rel}$</td>
</tr>
</tbody>
</table>

^a The limiting width-to-thickness ratios, $\lambda_{md}$ and $\lambda_{md}$, are defined in Seismic Provisions Table D1.1, with $R F_{y}$ replaced by $P_{rel}^{y}$.

^b For columns in concentrically braced frames with V- or inverted V-bracing, the value for $m$ is permitted to be multiplied by 1.25.

---

For When $\frac{P_{rel}}{P_{rel}} < 0.2k$

\[
\left[ \frac{P_{rel}}{2P_{rel}} \right] \left[ \frac{M_{UDx}}{m_{M_{pes}}} + \frac{M_{UDy}}{m_{M_{pes}}} \right] \leq \kappa \tag{C3-68}
\]

For When $\frac{P_{rel}}{P_{rel}} \geq 0.2k$

\[
\left[ \frac{P_{rel}}{8P_{rel}} \right] \left[ \frac{M_{UDx}}{m_{M_{pes}}} + \frac{M_{UDy}}{m_{M_{pes}}} \right] \leq \kappa \tag{C3-29}
\]

where

$M_{pes} = $ expected plastic flexural strength about the $x$-axis determined in accordance with Section C3.3a.2 at $P = P_{rel} = 0$, kip-in. (N-mm)

$M_{pes} = $ expected plastic moment flexural strength about the $y$-axis determined in accordance with Section C3.3a.2 at $P = P_{rel} = 0$, kip-in. (N-mm)

$M_{UDx} = $ bending moment about the $x$-axis determined in accordance with ASCE/SEI 41, Section 7.5.2.1.1, kip-in. (N-mm)

---

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
2. Member Strength

A steel column or brace in compression shall satisfy Equation C3-8 or C3-9, and Equation C3-10 for a given performance level.

For When $\frac{P_{ef}}{P_{ef}} < 0.2\kappa$

\[
\frac{P_{ef}}{2P_{ef}} \left(\frac{M_{CExLTB}}{m_{x},M_{CExLTB}} + \frac{M_{CExLTB}}{m_{y},M_{CExLTB}}\right) \leq \kappa
\]  
(C3-80)

For When $\frac{P_{ef}}{P_{ef}} \geq 0.2\kappa$

\[
\frac{P_{ef}}{9P_{ef}} \left(\frac{8}{m_{y},M_{CExLTB}} + \frac{M_{CExLTB}}{m_{x},M_{CExLTB}}\right) \leq \kappa
\]  
(C3-91)

and

\[
\frac{P_{ef}}{P_{ef}} \leq 0.75\kappa
\]  
(C3-1012)

where

- $M_{CExLTB}$ = expected lateral-torsional buckling flexural strength about the x-axis, kip-in. (N-mm)
- $M_{CExLTB}$ = lower-bound lateral-torsional buckling flexural strength about the x-axis, kip-in. (N-mm)
- $M_{CExLTB}$ = lateral-torsional buckling flexural strength about the x-axis determined in accordance with Section C3.3a.2 or C3.3b.2 at $P_{ef} = 0$. If flexure is deformation-controlled, $M_{CExLTB} = M_{CExLTB}$; otherwise flexure is force-controlled and $M_{CExLTB} = M_{CExLTB}$, kip-in. (N-mm)
- $M_{Exy}$ = expected flexural strength about the y-axis, kip-in. (N-mm)
- $M_{Exy}$ = lower-bound flexural strength about the y-axis, kip-in. (N-mm)
- $M_{Cy}$ = flexural strength about the y-axis determined in accordance with Section C3.3a.2 or C3.3b.2. If flexure is deformation-controlled, $M_{Cy} = M_{Exy}$; otherwise flexure is force-controlled and $M_{Cy} = M_{Exy}$, kip-in. (N-mm)
If $M_{CELTB} < M_{pe}$, then $m$ for bending about the $x$-axis in Equation C3-8 and C3-9 shall be replaced by $m$ determined from Equation C2-3 taking $M_{CE} = M_{CELTB}$.

A steel column or brace in tension shall satisfy Equation C3-8 or C3-9, except that $P_{CL}$ shall be taken as the expected tensile strength, $T_{CE}$, if the axial action in the column or brace is deformation-controlled or the lower-bound tensile strength, $T_{CL}$, if the axial action in the column or brace 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, the column or brace shall satisfy Equation C3-11 for each performance level.

$$\frac{P_{UD}}{m_{T}T_{CE}} \leq \kappa \quad (C3-11)$$

where

- $P_{UD}$ = tensile force in the member determined in accordance with ASCE/SEI 41, Section 7.5.2.1.1, kips (N)
- $m_{T}$ = component capacity modification factor, $m$, for column or brace in tension taken from Table C3.3

b. Nonlinear Analysis Procedures

When constructing the nonlinear force-deformation model for use in the nonlinear analysis procedures, the generalized force-deformation curve for 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 $a_{h}$, a strain-hardening slope for flexural action, as 3% of the elastic slope is permitted. Further modification of the curve is permitted if a greater strain-hardening slope value for $a_{h}$ is justified by testing or analysis. When the flexural behavior of a column or brace is considered deformation-controlled, the plastic chord rotation demand, $\theta_{p}$, 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 is flexure-controlled, the yield chord rotation, $\theta_{y}$, of the column shall be determined from Equation C3-12. Otherwise, if the column or brace is shear-controlled or shear-flexure-controlled, $\theta_{y}$ shall be taken as $\gamma_{y}$ determined from Section C3.4a.3.b.

---

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
\[ \theta_y = \frac{M_{pl} L}{6 \tau_b E I} \quad (C3-1214) \]

where

- \( L \) = laterally unbraced length of member, in. (mm)
- \( \tau_b \) = stiffness reduction parameter, determined as follows:

1. When \( |P|/P_{cr} \leq 0.5 \)
   \[ \tau_b = 1.0 \quad (C3-15a) \]
2. When \( |P|/P_{cr} > 0.5 \)
   \[ \tau_b = 4 \frac{|P|}{P_{cr}} \left( 1 - \frac{|P|}{P_{cr}} \right) \quad (C3-15b) \]

**User Note:** The underlying assumptions for Equation C3-12-14 are the same as Equation C2-21. When other end conditions may shift the inflection point away from midspan, it is permitted to determine \( \theta_y \) by analysis. See Commentary for more information.

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 \( \eta \) as zero.

**User Note:** Shear deformation (accounted for by \( \eta \)) in a flexure-controlled column or brace with a length greater than \( 10M_{pl} / V_{CE} \), including axial force interaction, is generally small and can be neglected in Equation C3-12-14.

### TABLE C3.6

<table>
<thead>
<tr>
<th>Axial Load Ratio and Section Compactness</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
</table>
### Plastic Rotation Angle, rad

<table>
<thead>
<tr>
<th></th>
<th>IO</th>
<th>LS</th>
<th>CP</th>
</tr>
</thead>
</table>

#### Columns in Compression

1. **Highly ductile** \((\leq \lambda_{md})\)

\[
a = 0.8 \left(1 - \frac{P_G}{P_{wy}}\right)^{0.2} \left(0.1 \frac{L}{r_f} + 0.8 \frac{h}{t_w}\right)^{-1} - 0.0035 \geq 0
\]

\[
b = 7.4 \left(1 - \frac{P_G}{P_{wy}}\right)^{0.2} \left(0.5 \frac{L}{r_f} + 2.9 \frac{h}{t_w}\right)^{-1} - 0.006 \geq 0
\]

\[
c = 0.9 - 0.9 \frac{P_G}{P_{wy}}
\]

2. **Non-moderately ductile** \((\lambda > \lambda_{md})\)

\[
a = 1.2 \left(1 - \frac{P_G}{P_{wy}}\right)^{0.2} \left(1.4 \frac{L}{r_f} + 0.1 \frac{h}{t_w} + 0.9 \frac{b_f}{2t_f}\right)^{-1} - 0.0023 \geq 0
\]

\[
b = 2.5 \left(1 - \frac{P_G}{P_{wy}}\right)^{0.2} \left(0.1 \frac{L}{r_f} + 0.2 \frac{h}{t_w} + 2.7 \frac{b_f}{2t_f}\right)^{-1} - 0.0097 \geq 0
\]

\[
c = 0.5 - 0.5 \frac{P_G}{P_{wy}}
\]

3. **Other:**

Linear interpolation between the values on lines 1 and 2 shall be performed, and the lower resulting value shall be used.

#### Columns in Tension

1. For \(|P_d / P_{rw}| < 0.2\)

\[
a = 9 \beta_o \quad \beta_o = 0.6
\]

2. For \(|P_d / P_{rw}| \geq 0.2\)

\[
a = 16.5 (1 - 5/3 \frac{|P_d / P_{rw}|}{\beta_o})^o \quad \beta_o = 0.6 (1 - 5/3 \frac{|P_d / P_{rw}|}{\beta_o}) + 0.2 \geq 0.2
\]

\[
b = 0.25 \beta_o \quad \beta_o = 13.5 (1 - 5/3 \frac{|P_d / P_{rw}|}{\beta_o}) \geq 0 \quad 16.5 (1 - 5/3 \frac{|P_d / P_{rw}|}{\beta_o}) \geq 0
\]

3. **Other:**

Linear interpolation between the values on lines 1 and 2 shall be used.
### TABLE C3.6
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Columns and Braces Subjected to Flexure with Axial Compression or Tension

<table>
<thead>
<tr>
<th>Axial Load Ratio and Section Compactness</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Rotation Angle, rad</td>
</tr>
<tr>
<td>IO</td>
<td>LS</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_{G} )</td>
<td>Axial force component of the gravity load as determined by ASCE/SEI 41, Equation 7-3.</td>
</tr>
<tr>
<td>( d )</td>
<td>Full nominal depth of member, in. (mm)</td>
</tr>
<tr>
<td>( h )</td>
<td>Height of flange, for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)</td>
</tr>
<tr>
<td>( L )</td>
<td>For beams in concentrically braced frames with V- or inverted V-bracing, the permissible performance parameters are permitted to be multiplied by 1.25.</td>
</tr>
<tr>
<td>( P_{uy} )</td>
<td>Ultimate axial load of the member, in. (mm)</td>
</tr>
<tr>
<td>( r_{y} )</td>
<td>Radius of gyration about y-axis, in. (mm)</td>
</tr>
<tr>
<td>( t )</td>
<td>Design wall thickness of HSS member, in. (mm)</td>
</tr>
<tr>
<td>( t_{f} )</td>
<td>Thickness of flange, in. (mm)</td>
</tr>
</tbody>
</table>

Where the modeling parameter \( a \) is equal to zero or where \( P_{G} / P_{uy} > 0.6 \), the component shall remain elastic for flexure.

Columns or braces classified as deformation-controlled for flexure shall also satisfy Equation C3-8-10 or C3-9-11, and Equation C3-10-12 when the column or brace is in compression, in Section C3.4.2a, except that the axial force, \( P \), and bending moments about the x- and y-axes, \( M_{x} \) and \( M_{y} \), shall be substituted for \( P_{uy} \), \( M_{uy} \), and \( M_{by} \), respectively, developed at the target displacement for the nonlinear static procedure or at the instant of computation for the nonlinear dynamic procedure, and \( m \) shall be taken as unity.
3. Shear Action Concurrent with Axial Action

a. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior of a column or brace is considered deformation-controlled, the shear behavior strength shall be evaluated using ASCE/SEI 41, Equation 7-36, with the expected shear strength, $Q_{CE} = V_{CE}$, determined in accordance with Section C3.3a.3 and $m$ taken from Table C3.7.

### TABLE C3.7
Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—Columns and Braces Subjected to Shear $^{a,b,c,d}$

<table>
<thead>
<tr>
<th>Axial Load Ratio and Member Length</th>
<th>IO</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>For $</td>
<td>P_{Uf}</td>
<td>/P_{ye}</td>
<td>\leq 0.2$</td>
</tr>
<tr>
<td>a. $L_c \leq 1.6 \frac{M_{ce}}{V_{ce}}$ (Shear-Controlled)</td>
<td>1.5</td>
<td>9</td>
<td>13</td>
</tr>
<tr>
<td>b. $L_c &gt; 1.6 \frac{M_{ce}}{V_{ce}}$, Linear interpolation between the values on lines a and b</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>c. $L_c \leq 2.6 \frac{M_{ce}}{V_{ce}}$, Linear interpolation between the values on lines a and b</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>d. $L_c &gt; 2.6 \frac{M_{ce}}{V_{ce}}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>For $</td>
<td>P_{Uf}</td>
<td>/P_{ye}</td>
<td>&gt; 0.2$</td>
</tr>
<tr>
<td>a. $L_c \leq 1.6 \frac{M_{ce}}{V_{ce}}$ (Shear-Controlled)</td>
<td>0.75 (1 - 5/3</td>
<td>$</td>
<td>P_{Uf}</td>
</tr>
<tr>
<td>b. $L_c &gt; 1.6 \frac{M_{ce}}{V_{ce}}$, Linear interpolation between the values on lines a and b</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>c. $L_c \leq 2.6 \frac{M_{ce}}{V_{ce}}$, Linear interpolation between the values on lines a and b</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>d. $L_c &gt; 2.6 \frac{M_{ce}}{V_{ce}}$</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$^{a}$ 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 $L_c \geq 2.6 \frac{M_{ce}}{V_{ce}}$. For $1.6 \frac{M_{ce}}{V_{ce}} < L_c < 2.6 \frac{M_{ce}}{V_{ce}}$, $m$ shall be linearly interpolated between the tabulated values and 1.0.

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
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 Tables C2.4, shall be used for column or braces. Alternatively, these relationships may be derived from testing or analysis. For columns or braces, a strain-hardening slope is permitted to take \( a \) as 6% of the elastic slope is permitted. Further modification of the curve is permitted if a greater strain-hardening slope value for \( a \) is justified by testing or analysis. When the shear behavior of a column or brace is considered deformation-controlled, the plastic shear deformation demand, \( \gamma_p \), predicted by analysis shall be not greater than the permissible plastic shear deformation provided in Table C3.8 for a given performance level. The shear yield deformation of a column or brace, \( \gamma_y \), shall be determined from Equation C3.1416:

\[
\gamma_y = \frac{V_{CE}}{K_e} \tag{C3.1416}
\]

where

\( K_e = \) elastic shear stiffness of the column or brace determined from the analytical model or another rational method, including the effect of an axial load, 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)

| TABLE C3.8 |
|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Modeling Parameters | Expected Deformation Capacity | Modeling Parameters | Expected Deformation Capacity |
| Axial Load Ratio and Member Length | Plastic Rotation Deformation, rad | Residual Strength Ratio | Plastic Shear Deformation, rad |
| Axial Load Ratio and Member Length | \( a \) | \( b \) | \( c \) | IO | LS | CP |
| \( \frac{|P|}{P_{ye}} \leq 0.2 \) | 0.15 | 0.17 | 0.8 | 0.005 | 0.14 | 0.16 |
| \( \frac{L_c}{1.6 \frac{M_{ye}}{V_{ye}}} \) (Shear-Controlled) | Linear interpolation shall be used |
| \( \frac{L_c}{2.6 \frac{M_{ye}}{V_{ye}}} \) (Flexure-Controlled) | \( 0.0 \) |
| \( \frac{1.6 M_{ye}}{V_{ye}} \) (Shear-Flawed) | \( \frac{2.6 M_{ye}}{V_{ye}} \) (Flexure-Flawed) | Linear interpolation shall be used |

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
American Institute of Steel Construction
1457
4b. Force-Controlled Action

1459 1. Axial Action

1460 a. Linear Analysis Procedures

1461 When linear analysis procedures are used and the axial behavior of a column or brace is
1462 considered force-controlled, the axial behavior shall be evaluated using Equation 7-37 of
1463 ASCE/SEI 41 with the lower-bound axial strength, $Q_{CL} = P_{CL}$, determined in accordance with
1464 Section C3.3b.1.

1465 b. Nonlinear Analysis Procedures

1466 When nonlinear analysis procedures are used and the axial behavior of a column or brace is
1467 considered force-controlled, the axial behavior shall be evaluated using Equation 7-38 of
1468 ASCE/SEI 41 with the lower-bound axial strength, $Q_{CL} = P_{CL}$, determined in accordance with
1469 Section C3.3b.1.

1470 Alternatively, when a force-controlled action, for the column or brace, is explicitly modeled with a
1471 nonlinear force-deformation behavior, it is permitted to evaluate the column or brace based on
deflection. When nonlinear analysis procedures are used and the axial behavior of a column or
brace is considered force-controlled, for such an evaluation, the total axial deformation, $\Delta$, of the
column or brace predicted by analysis shall not exceed $\Delta_T$ in tension or $\Delta_C$ in compression
determined from Equations C3-35 and C3-46, respectively, except $T_{CL}$ and $P_{CL}$ shall be
substituted for $T_{CE}$ and $P_{CE}$, respectively.

1472 The maximum force determined by ASCE/SEI 41, Section 7.5.3.2.3, shall not exceed the lower-bound axial
strength, $Q_{CL} = P_{CL}$, determined in accordance with Section C3.3b.1.

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
2. Flexural Action Concurrent with Axial Action

a. Linear Analysis Procedures

When linear analysis procedures are used and the flexural behavior of a column or brace is considered force-controlled, the column or brace shall satisfy Equation C3-8 or C3-9, and C3-10 for a given performance level, determined with \( P_{ydB} \) and the lower-bound axial yield strength, \( P_{yLB} \), substituted for \( F_y \) and \( P_y \), respectively.

Columns or braces classified as force-controlled for flexure shall also satisfy Equations C3-6 and C3-7 for a given performance level, except that \( P_{UB} \), \( M_{UDx} \) and \( M_{UDy} \) shall be taken as \( P \), \( M_x \) and \( M_y \), respectively; the values for \( m \) shall be taken as unity; \( M_{pe} \) shall be taken as the lower-bound plastic flexural strength, \( M_{pLB} \), with \( F_{yLB} \) substituted for \( F_{ye} \); and \( P_y \), as determined in Section C3.2b, shall be taken as the lower-bound axial yield strength, \( P_{yLB} \), with \( F_{yLB} \) substituted for \( F_{ye} \).

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the flexural behavior of a column or brace is considered force-controlled, the column or brace shall satisfy Equation C3-8 or C3-9, and C3-10 when the column is in compression, except that \( P_{UB} \), \( M_{UB} \) and \( M_{UB} \) shall be taken as \( P \), \( M_x \) and \( M_y \), respectively, developed at the target displacement for the nonlinear static procedure or at the instant of computation for the nonlinear dynamic procedure, and \( m \) shall be taken as unity.

Columns or braces classified as force-controlled for flexure shall also satisfy Equations C3-6 and C3-7 for a given performance level, except that \( P_{UB} \), \( M_{UB} \) and \( M_{UB} \) shall be taken as \( P \), \( M_x \) and \( M_y \), respectively; the values for \( m \) shall be taken as unity; \( M_{pe} \) shall be taken as the lower-bound plastic flexural strength, \( M_{pLB} \), with \( F_{yLB} \) substituted for \( F_{ye} \); and \( P_y \), as determined in Section C3.2b, shall be taken as the lower-bound axial yield strength, \( P_{yLB} \), with \( F_{yLB} \) substituted for \( F_{ye} \).

3. Shear Action Concurrent with Axial Action

a. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior of a column or brace is considered force-controlled, the shear behavior shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound flexural strength, \( Q_{CL} = V_{CL} \), determined in accordance with Section C3.3b.2.

b. Nonlinear Analysis Procedures

When nonlinear analysis procedures are used and the shear behavior of a column or brace is considered force-controlled, the shear behavior shall be evaluated using ASCE/SEI 41, Equation 7-38, with the lower-bound flexural strength, \( Q_{CL} = V_{CL} \), determined in accordance with Section C3.3b.2.
Alternatively, when a force-controlled action, for the column or brace, is explicitly modeled with a nonlinear force-deformation behavior, it is permitted to evaluate the column or brace based on deformation. When nonlinear analysis procedures are used and the shear behavior of a column or brace is considered force-controlled, for such an evaluation, the total shear deformation, $\gamma$, of the column or brace predicted by analysis shall not exceed $\gamma_y$ determined from Equation C3-1416. The maximum force determined by ASCE/SEI 41, Section 7.5.3.2.3, shall not exceed the lower-bound shear strength, $Q_{CL} = V_{CL}$, determined in accordance with Section C3.3b.3.
1321 C4. PANEL ZONES

1322 1. General

1323 The component characteristics of panel zones at fully restrained 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 panel zone is the region of a column at a beam-to-column connection delineated by
the adjacent beam and column flanges.

1327 The shear behavior of a panel zone shall be designated as either deformation-controlled or force-controlled
in accordance with Chapters D through I.

1.329 2. Stiffness

1330 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.

1333 The force-deformation model for a panel zone shall account for all significant sources of deformations
(e.g., flexural, shear) that affect its behavior.

1335 Inclusion of 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.

1343 2a. Flexural Stiffness

1344 There are no additional requirements beyond those specified in Section C4.2.

1345 2b. Axial Stiffness

1346 There are no additional requirements beyond those specified in Section C4.2.

1347 2c. Shear Stiffness

1348 If the panel zone includes concrete encasement or backing, then the shear stiffness of the panel zone is
permitted to be determined using full composite action, including the effects of cracking, provided a
mechanism exists that provides sufficient for the transfer and distribution of forces to the surrounding
components.

1352 3. Strength

1353 The shear strength of a panel zone shall be determined in accordance with this section.
The shear strength of the concrete encasement or backing 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.

3. Deformation-Controlled Action

The expected shear strength, $V_{CE}$, of a panel zone shall be determined from Equations C4-1a and C4-1b, and $Q_{CE} = Q = V_{CE}$. The axial force, $P$ (compression or tension) in Equations C4-1a and C4-1b shall be computed in accordance with Section C3 and dependent on the analysis type selected.

(a) When $\frac{|P|}{P_{ce}} \leq 0.4$

$$V_{CE} = V_{ce} = 0.55F_{ye}d_c t_p$$  \hspace{1cm} (C4-1a)

(b) When $\frac{|P|}{P_{ce}} > 0.4$

$$V_{CE} = V_{ce} = 0.55F_{ye}d_c t_p \left(1.4 - \frac{|P|}{P_{ce}}\right)$$  \hspace{1cm} (C4-1b)

where

- $V_{ce} = \text{expected shear strength of the panel zone reduced for the effect of axial force (compression or tension), kips (N)}$
- $d_c = \text{depth of column, in. (mm)}$
- $t_p = \text{total thickness of panel zone, including doubler plates, in. (mm)}$

3b. Force-Controlled Action

The lower-bound shear strength, $V_{CL}$, of a panel zone shall be determined from Equations C4-1a and C4-1b except that $F_{yLB}$ shall be substituted for $F_{ye}$, 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 Action

1. Linear Analysis Procedures

When the linear analysis procedures are used and the shear behavior of a panel zone is considered deformation-controlled, the shear behavior shall be evaluated using Equation 7-36 of ASCE/SEI 41 with the expected panel zone shear strength, $Q_{CE} = V_{CE}$, determined in accordance with C4.3a and $m$ taken from Table C4.1 and adjusted as required by this section. The axial load, $P_{UF}$, shall be determined in accordance with Section C3.
### TABLE C4.1
Component Capacity Modification Factor, \( m \), for Linear Analysis Procedures—Panel Zones Subject to Shear

| Axial Load \( |P_f|/P_m \) | Primary Component | Secondary Component |
|-----------------|------------------|--------------------|
|                 | IO               | LS                 | CP                | LS               | CP                |
| For \( |P_f|/P_m \leq 0.4 \) | 1.5              | 84                 | 115.5             | 126              |
| For \( |P_f|/P_m > 0.4 \) | \( (2.5/3)(1-|P_f|/P_m)+1 \) | \( (35/3)(1-|P_f|/P_m)+1 \) | \( (50/3)(1-|P_f|/P_m)+1 \) | \( (55/3)(1-|P_f|/P_m)+1 \) |

**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  

*Regardless of the modifiers applied, \( m \) need not be taken as less than 1.0.*

The component capacity modification factor, \( m \), of the panel zone in Table C4.1, for the life safety (LS) and collapse prevention (CP) performance levels shall be multiplied by 0.52 when all of the following conditions are met:

1. \( V_{PZ}/V_{ye} > 1.10 \), where the panel zone shear, \( V_{PZ} \), is determined from Equation C5-19 and \( V_{ye} \) is determined in accordance with Section C4.3a,
2. welds are located at the edges of the panel zone where column flanges are susceptible to kinking, and
3. the beam flange-to-column flange connection is made with complete-joint-penetration (CJP) groove welds that do not satisfy the requirements of Seismic Provisions Section A3.4.

### 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. Alternatively, these relationships may be derived from experiment-testing or analysis. For panel zones, a strain-hardening slope is permitted to take \( \alpha_h \) for shear action of as 6% of the elastic slope is permitted. Further modification of the curve is permitted if a greater strain-hardening slope is justified by experiment testing or analysis.

When the shear strength of a panel zone is considered deformation-controlled, the plastic shear deformation demand, \( \gamma_p \), predicted by analysis shall be not greater than the permissible plastic shear deformation determined from Equations C4-3 or C4-4 or that provided in Table C4.2 for a given performance level. The shear yield deformation, \( \gamma_y \), of a panel zone shall be determined from Equation C4-2. The axial load, \( P \), shall be determined in accordance with Section C3.

\[
\gamma_y = \frac{F_{ey}}{G \frac{P}{P_{ey}}} \left( 1 - \frac{|P|}{P_{ey}} \right)^{2} \quad (C4-2)
\]
The permissible plastic shear deformation of the panel zone, \( \gamma_{p,pz} \), for the LS and CP performance levels shall be determined from Equations C4-3 or C4-4, when all of the following conditions are met:

(a) \( V_{PZ} / V_{ye} > 1.10 \), where \( V_{PZ} \) is determined from Equation C5.19 and \( V_{ye} \) is determined in accordance with Section C4.3a,

(b) Welds are located at the edge of the panel zone where column flanges are susceptible to kinking, and

(c) The beam flange-to-column flange connection is made with CJP groove welds.

(1) 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

\[
\gamma_{p,pz} = \frac{0.092 F_y}{G} \left( \alpha + \frac{3.45}{\alpha} \right) \left[ 1 - \frac{\left| P \right|^2}{2P_{p,cf}} \right] \leq 6f_y \quad \text{(C4-3)}
\]

or

\[
\gamma_{p,pz} = \frac{0.183 F_y}{G} \left( \alpha + \frac{3.45}{\alpha} \right) \left[ 1 - \frac{\left| P \right|^2}{2P_{p,cf}} \right] \leq 12f_y \quad \text{(C4-3)}
\]

(2) 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

\[
\gamma_{p,pz} = \frac{0.183 F_y}{G} \left( \alpha + \frac{3.45}{\alpha} \right) \left[ 1 - \frac{\left| P \right|^2}{2P_{p,cf}} \right] < 12f_y \quad \text{(C4-4)}
\]

\[
\gamma_{p,pz} = \frac{0.092 F_y}{G} \left( \alpha + \frac{3.45}{\alpha} \right) \left[ 1 - \frac{\left| P \right|^2}{2P_{p,cf}} \right] < 6f_y \quad \text{(C4-4)}
\]

where

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

\( P_{p,cf} = A_{cf} F_{y,c} = \) expected axial yield strength of the column flange, kips (N)

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

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

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

\( \alpha = d_b / b_{cf} \)

<table>
<thead>
<tr>
<th>Axial Load</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_{p,pz} )</td>
<td>( \frac{0.092 F_y}{G} \left( \alpha + \frac{3.45}{\alpha} \right) \left[ 1 - \frac{\left</td>
<td>P \right</td>
</tr>
</tbody>
</table>
4b. Force-Controlled Action

1. Linear Analysis Procedures

When linear analysis procedures are used and the shear behavior of a panel zone is considered force-controlled, the shear behavior shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound shear strength, \( Q_{CL} = V_{CL} \), 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 shear behavior shall be evaluated using ASCE/SEI 41, Equation 7-38, with the lower-bound shear strength, \( Q_{CL} = V_{CL} \), determined in accordance with Section C4.3b.

Alternatively, when a force-controlled action, for the panel zone, is explicitly modeled with a nonlinear force-deformation behavior, it is permitted to evaluate the panel zone based on deformation. For such an evaluation when nonlinear analysis procedures are used and the shear behavior of a panel zone is considered force-controlled, the total shear deformation, \( \gamma \), of the panel zone predicted by analysis shall not exceed \( \gamma_y \) determined from Equation C4-2.

The lower-bound shear strength, \( Q_{CL} = V_{CL} \), determined in accordance with Section C4.3b shall not be less than the maximum force determined by ASCE/SEI 41, Section 7.5.3.2.3.
C5. **BEAM AND COLUMN CONNECTIONS**

1. **General**

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

The component characteristics of steel connections of structural components subject to seismic forces or deformations shall be determined in accordance with this Section.

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 and Beam-to-Beam 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 connection types for which permissible performance parameters are provided in Section C5. Modeling procedures, permissible performance parameters, and retrofit measures for moment frames with fully restrained FR (FR) and partially restrained (PR) beam-to-column connections shall be as determined in Section D2.

<table>
<thead>
<tr>
<th>Connection</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded unreinforced flange (WUF) (pre-1995)</td>
<td>Complete joint penetration (CJP) groove welds between beam flanges and column flanges or continuity plates placed between the column flanges when the beam frames into the weak way. The web may be bolted or welded to a shear tab attached to the column or directly welded to the column. A composite slab may or may not be present.</td>
</tr>
<tr>
<td>Bottom haunch in WUF with top flange weld replaced</td>
<td>The beam flanges are connected to the column flanges with existing CJP groove welds. The beam web is either bolted or welded to a single-plate shear connection, or the beam web is welded directly to the column. A composite slab may, or may not, be present. The existing top flange weld shall be removed and replaced with a CJP groove weld meeting the requirements of the Seismic Provisions Section A3.4b. The new bottom haunch shall be connected to both the beam flange and the column flange with CJP groove welds meeting the requirements of the Seismic Provisions Section A3.4b.</td>
</tr>
</tbody>
</table>
### TABLE C5.1
Beam-to-Column and Beam-to-Beam FR Connection Types

<table>
<thead>
<tr>
<th>Connection</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom haunch in WUF with existing top flange weld unaltered</td>
<td>The beam flanges are connected to the column flanges with existing CJP groove welds. The beam web is either bolted or welded to a single-plate shear connection, or the beam web is welded directly to the column. A composite slab may, or may not, be present. The new bottom haunch shall be connected to both the beam flange and the column flange with CJP groove welds meeting the requirements of the Seismic Provisions Section A3.4b. The existing top flange weld is permitted to remain unaltered.</td>
</tr>
<tr>
<td>Welded cover plate in WUF</td>
<td>The beam flanges are connected to the column flanges with existing CJP groove welds. The beam web is either bolted or welded to a single-plate shear connection, or the beam web is welded directly to the column. A composite slab may, or may not, be present. The existing beam flange welds shall be removed and replaced with CJP groove welds meeting the requirements of the Seismic Provisions Section A3.4b. The new cover plates shall be connected to the column flange with CJP groove welds meeting the requirements of the Seismic Provisions Section A3.4b.</td>
</tr>
<tr>
<td>Improved WUF—Bolted web (IWUF-B)</td>
<td>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. A composite slab may, or may not, be present. The existing CJP groove welds, connecting the beam flanges to the column, shall meet the requirements of the Seismic Provisions Section A3.4a. Alternatively, the existing CJP groove welds shall be removed and replaced with CJP groove welds meeting the requirements of the Seismic Provisions Section A3.4b.</td>
</tr>
<tr>
<td>AISC 358 WUF—Welded web (WUF-W)</td>
<td>The beam flanges are connected to the column flanges with CJP groove welds. The beam web is welded to the column and a single-plate shear connection. The single plate is welded to the column. A composite slab may, or may not, be present. This connection shall meet the requirements of ANSI/AISC 358, Chapter 8.</td>
</tr>
</tbody>
</table>

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
TABLE C5.1
Beam-to-Column and Beam-to-Beam FR Connection Types

<table>
<thead>
<tr>
<th>Connection</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded flange plates</td>
<td>The beam flanges are connected to the flange plates with fillet welds; 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 or welded to a single-plate shear connection, or the beam web is welded directly to the column. A composite slab may, or may not, be present. The existing CJP groove welds, connecting the flange plates to the column, shall meet the requirements of the Seismic Provisions Section A3.4a.</td>
</tr>
<tr>
<td>Reduced beam section</td>
<td>The beam flanges are connected to the column flanges with CJP groove welds. The net area of the beam flange is reduced to force plastic hinging away from the column face. The beam web is bolted or welded to a single-plate shear connection, with or without direct weld to the column. Composite slab may, or may not, be present. This connection shall meet the requirements of ANSI/AISC 358, Chapter 5.</td>
</tr>
<tr>
<td>Welded bottom haunch (existing)</td>
<td>The beam top flange is connected to the column flange with an existing CJP groove weld. The existing bottom haunch is connected to both the beam flange and the column flange with CJP groove welds. The beam web is bolted or welded to a single-plate shear connection, with or without direct weld to the column. A composite slab may, or may not, be present. The existing CJP groove welds, for the bottom haunch, shall be removed and replaced with CJP groove welds meeting the requirements of the Seismic Provisions Section A3.4b. The existing CJP groove weld, connecting the beam top flange to the column, shall meet the requirements of the Seismic Provisions Section A3.4b.</td>
</tr>
</tbody>
</table>
# TABLE C5.1
## Beam-to-Column and Beam-to-Beam FR Connection Types

<table>
<thead>
<tr>
<th>Connection</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded bottom haunch (retrofitted)</td>
<td>The beam flanges are connected to the column flanges with existing CJP groove welds. The beam web is bolted or welded to a single-plate shear connection, with or without direct weld to the column. A composite slab may, or may not, be present. The new bottom haunch shall be connected to both the beam flange and the column flange with CJP groove welds meeting the requirements of the Seismic Provisions Section A3.4b. The existing CJP groove weld, connecting the beam top flange to the column, shall meet the requirements of the Seismic Provisions Section A3.4a. Alternatively, the existing CJP groove weld, for the top flange, shall be removed and replaced with a CJP groove weld meeting the requirements of the Seismic Provisions Section A3.4b.</td>
</tr>
<tr>
<td>Welded top and bottom haunches (existing)</td>
<td>The existing top and bottom haunches are welded to both the beam flanges and column with CJP groove welds. The beam web is bolted or welded to a single-plate shear connection, with or without direct weld to the column. A composite slab may, or may not, be present. The existing haunches shall be connected to both the beam flanges and the column flange with CJP groove welds meeting the requirements of the 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 the Seismic Provisions Section A3.4b.</td>
</tr>
<tr>
<td>Welded top and bottom haunches (retrofitted)</td>
<td>The beam flanges are connected to the column flanges with existing CJP groove welds. The beam web is bolted or welded to a single-plate shear connection, with or without direct weld to the column. A composite slab may, or may not, be present. The new haunches shall be connected to both the beam flanges and the column flange with CJP groove welds meeting the requirements of the Seismic Provisions Section A3.4b. The existing flange welds are permitted to remain unaltered.</td>
</tr>
<tr>
<td>Welded cover—plated flanges</td>
<td>The beam flanges are connected to the cover plates with fillet welds, and are connected to the column with CJP groove welds. The cover plates are welded to the column with CJP groove welds. The beam web is either bolted or welded to a single-plate shear connection, or the beam web is welded directly to the column. A composite slab may, or may not, be present. The existing CJP groove welds, connecting both the beam flanges and cover plates to the column, shall meet the requirements of the Seismic Provisions Section A3.4a.</td>
</tr>
</tbody>
</table>
### TABLE C5.1
Beam-to-Column and Beam-to-Beam FR Connection Types

<table>
<thead>
<tr>
<th>Connection</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolted Flange Plate</td>
<td>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 or welded to a single-plate shear connection. A composite slab may, or may not, be present. The connection shall be considered FR, if it meets the requirements of ANSI/AISC 358, Chapter 7, or if the connection is stronger than the expected plastic flexural strength, $F_{yC}$, of the beam.</td>
</tr>
<tr>
<td>Double Split Tee</td>
<td>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 or welded to a single-plate shear connection. A composite slab may, or may not, be present. This connection is permitted to be considered FR, if it satisfies the strength and connection deformation requirements of Section C5.1a, or those given in ANSI/AISC 358.</td>
</tr>
<tr>
<td>Bolted End Plate in conformance with ANSI/AISC 358</td>
<td>The beam flanges are connected to the end-plate with CJP groove welds. The beam webs are welded to the end-plate with CJP groove welds or fillet welds. The end-plate is bolted to the column flange. A composite slab may, or may not, be present. This connection shall meet the requirements of ANSI/AISC 358, Chapter 6.</td>
</tr>
</tbody>
</table>

#### 1b. Partially Restrained Beam-to-Column Moment Connections and Beam-to-Beam 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 connection types 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 Section D2.
### TABLE C5.2
Beam-to-Column PR Connection Types

<table>
<thead>
<tr>
<th>Connection</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top and Bottom Flange Angle</td>
<td>Flange angles bolted or riveted to beam flanges and column flange without a composite slab.</td>
</tr>
<tr>
<td>Double Split-Tee</td>
<td>Split-tees bolted or riveted to beam flanges and column flange.</td>
</tr>
<tr>
<td>Composite Top and Flange Angle Bottom</td>
<td>Flange angle bolted or riveted to column flange and beam bottom flange with composite slab.</td>
</tr>
<tr>
<td>Bolted Flange Plates</td>
<td>Flange plate with CJP groove welds at column and bolted to beam flanges.</td>
</tr>
<tr>
<td>Bolted End-Plate</td>
<td>Stiffened or unstiffened end-plate welded to beam and bolted to column flange.</td>
</tr>
</tbody>
</table>
TABLE C5.2
Beam-to-Column PR Connection Types

<table>
<thead>
<tr>
<th>Connection</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Connection with Slab</td>
<td>Single-plate shear connection, composite slab.</td>
</tr>
<tr>
<td>Shear Connection without Slab</td>
<td>Single-plate shear connection, no composite slab.</td>
</tr>
</tbody>
</table>

1c. Column-to Base Connections

Column-to-base connections shall be made with welds, bolts, rivets, or a combination thereof.

2. Stiffness

The stiffness of steel connections shall be based on principles of structural mechanics and as specified in the Specification unless superseded by these Provisions.

The force-deformation model for a steel connection shall account for all significant sources of deformations (e.g., axial, flexural, shear) that affect its behavior.

User Note: Not all connection types 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 modelled as a continuous component from joint to joint.

2a. Beam-to-Column and Beam-to-Beam 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 Connections**

Modeling of connection rotational stiffness for fully restrained (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 Connections**

The rotational stiffness, $K_\theta$, of each partially restrained (PR) connection for use in modeling shall be determined by the procedure of this section, unless otherwise justified by testing, or by rational analysis. The deformation of the connection shall be included when calculating frame displacements.

In lieu of explicit connection modelling, it is permitted to adjust the flexural stiffness of a beam with PR connections, $EI_b$, to account for the flexibility of the end connections using Equation C5-1:

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

Where $EI_b$ = moment of inertia of the beam about the axis of bending, in.$^4$ (mm$^4$)
$L_{CL}$ = 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. **Brace-to-Beam and Brace-to-Column Connections**

Brace connections shall be analyzed as FR or PR where the flexural stiffness provided by the connection results in flexure action in the brace exceeding 10% of the expected plastic moment strength of the brace, including the effects from the concurrent axial action.

2cb. **Column-to-Base Connections**

The rotational stiffness, $K_{rb}$, of each base connection for use in modeling shall be determined by the procedure of this section, by testing, or by rational 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 steel connection shall be determined in accordance with this
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 actions shall be taken as the nominal strength for that action given in Specification Chapter J.

3a. Deformation-Controlled Action

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

The expected strengths for all applicable limit states for FR and PR moment connections shall be based on procedures specified in the Specification or Seismic Provisions, based on testing, principles of structural mechanics using expected material properties, or the requirements of this section; based on principles of structural mechanics using expected material properties, or based on the requirements of this section. Unless otherwise indicated in this section, the expected flexural strength, $M_{CE}$, of FR connections shall be determined using Equation (C5-2):

$$ Q_{CE} = M_{CE} = F_w Z_b $$  \hspace{1cm} (C5-2)

where

- $Z_b$ = plastic section modulus of beam, in.$^3$ (mm$^3$)

(a) For WUF (pre-1995) connections with beam depth of W24 (W610) and greater, $M_{CE}$ shall be determined as

$$ Q_{CE} = M_{CE} = F_w S_b $$  \hspace{1cm} (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 and Beam-to-Beam Connections

The expected strengths for all applicable limit states for PR moment connections shall be based on procedures specified in the Specification or Seismic Provisions, based on testing, principles of structural mechanics using expected material properties, or the requirements of this section; based on principles of structural mechanics using expected material properties, or based on the requirements of this section.

(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:

![Fig. C5.1. Top and bottom flange angle connection.](image)

(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:

$$Q_{CE} = M_{CE} = (F_{ve} A_b N_b) d_b$$  \hspace{1cm} \text{(C5-24)}

where

- $A_b =$ gross area of rivet or bolt, \(\text{in.}^2\) (\(\text{mm}^2\))
- $F_{nv} =$ nominal shear stress for bearing-type connections, given in Specification Section J3.6, ksi (MPa)
- $F_{ve} =$ expected shear strength of bolt or rivet, taken as $F_{nv}$, given in Specification Section J3, ksi (MPa)
- $N_b =$ 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:

$$Q_{CE} = M_{CE} = P_{CE} (d_b + t_b)$$  \hspace{1cm} \text{(C5-25)}

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 C5-46 and C5-57, kips (N).
where

\[ A_e = \text{effective net area of horizontal angle leg, in.}^2 \text{ (mm}^2) \]
\[ A_g = \text{gross area of horizontal angle leg, in.}^2 \text{ (mm}^2) \]
\[ F_{ue} = \text{expected tensile strength, ksi (MPa)} \]
\[ t_a = \text{thickness of angle, in. (mm)} \]

(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:

\[ Q_{CE} = M_{CE} = (F_{ue} A_t N_y) (d_b + b_a) \]  \hfill (C5-68)

where

\[ F_{te} = \text{expected tensile strength of bolt or rivet, taken as } F_{nt}, \text{ given in Specification Section J3, ksi (MPa)} \]
\[ b_a = \text{distance from the exterior flange face to the resultant tensile force of the bolt or rivet group, as shown in Figure C5.1, in. (mm)} \]

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 angles controls, the expected flexural strength of the connection, \( M_{CE} \), shall be determined as:

\[ Q_{CE} = M_{CE} = \frac{w t_a^2 F_{ue}}{2 \left( b_a - t_a \right)} (d_b + b_a) \]  \hfill (C5-92)

where

\[ w = \text{length of the flange angle, in. (mm)} \]

User Note: Equation C5-79 assumes the connection failure mode is plastic hinges developing in the vertical angle at the bolt centerline and at the face of the horizontal leg.

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

(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 C5-22.

(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:

\[
Q_{CE} = M_{CE} = (F_{ub} A_{c_0} N_b) (d_b + t_s)
\]

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 C5-35, where \( t_s \) is substituted for \( t_c \) and \( A_t \) and \( A_g \) are taken as the gross area and effective net area of the split-tee stem, respectively.

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

\[
Q_{CE} = M_{CE} = \left( \frac{w t_f^2 F_{ye}}{b_t - \frac{t_s}{2}} \right) (d_b + t_s)
\]

\[ \text{(C5-911)} \]
where
\[ b_t = \text{distance between the nearest row of fasteners in the flange of the split-tee and the centerline of the split-tee stem, as shown in Figure C5.2, in. (mm)} \]
\[ t_f = \text{thickness of flange of the split-tee, in. (mm)} \]
\[ w = \text{length of split-tee, in. (mm)} \]

User Note: Equation C5-2.11 assumes the connection failure mode is plastic hinges developing in the flange at the bolt centerline and at the face of the stem, above and below the stem.

(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, \( M_{c,r} \), of the connection shall be determined using Equation C5-53, 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 equal or exceeds the expected flexural strength of the connecting connected beam.

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 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 partially restrained PR connections

The expected strength for composite partially restrained PR connections shall be based on rational analysis procedures and experimental data derived from testing or analysis in accordance with ASCE/SEI 41, Section 7.6.
2. **Brace-to-Beam and Brace-to-Column Connections**

   The expected strengths for all applicable limit states for brace-to-beam and brace-to-column connections shall be based on procedures specified in the Specification or Seismic Provisions, based on testing, based on principles of structural mechanics using expected material properties, or based on the requirements of this section.

3. **Column-to-Base Connections**

   The expected strengths for all applicable limit states for column-to-base connections shall be based on procedures specified in the Specification or Seismic Provisions, based on testing, based on principles of structural mechanics using expected material properties, or based on the requirements of this section.

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 and Beam-to-Beam 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.

2. **Brace-to-Beam and Brace-to-Column Connections**

   The lower-bound strengths for all applicable limit states for brace-to-beam and brace-to-column connections shall be based on procedures listed in Section C5.3a.2 using lower-bound material properties instead of expected material properties.

32. **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.

423. **Column and Beam Splices**

   The lower-bound strengths for all applicable limit states for column and beam splices shall be based on procedures specified in the Specification or Seismic Provisions, based on testing, based on principles of structural mechanics using expected material properties, or based on the requirements of this section.

   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.

---

*2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*

Draft Ballot 3 Dated September 30, 2019

*American Institute of Steel Construction*
(a) With Complete-Joint-Penetration Groove Welded Splices

The lower-bound tensile strength of splices made with complete-joint-penetration (CJP) groove welds for a given action shall be determined in accordance with procedures given in the Specification for nominal strengths using the lower-bound yield strength, $F_{y_{LB}}$, for the yield strength, $F_y$.

(b) With Partial-Joint-Penetration Groove Welded Splices

The lower-bound tensile strength of splices made with partial-joint-penetration (PJP) groove welds, $\sigma_{cr}$, for a given action shall be determined in accordance with Equation C5-1012. Demand Weld stress demand on the splice, $\sigma_{UF}$, shall be determined as the maximum stress in the smaller section at the tip end of the partial-joint-penetration (PJP) groove weld or in accordance with Equation C5-1214. The demand, $\sigma_{UF}$, shall not exceed the lower-bound strength, $\sigma_{cr}$.

$$\sigma_{cr} = \frac{K_{IC}}{\sqrt{F} \left( \frac{a_0}{t_{f,u}} \right)} \leq F_{U} \left( \frac{1 - \frac{a_0}{t_{f,u}}}{\sqrt{a_0}} \right) \leq F_{y_{LB}} \quad (C5-1012)$$

where

$$\sigma_{UF} = \frac{P_{EF}}{A_g} = \left( \frac{M_{UF,x}}{S_x} \right) = \left( \frac{M_{UF,y}}{S_y} \right) \quad (C5-1214)$$

<table>
<thead>
<tr>
<th>Charpy V-Notch at Last, ft-lb (J)</th>
<th>$K_{IC}$, ksi in. (MPa in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 (6.8)</td>
<td>50 (1700)</td>
</tr>
<tr>
<td>10 (14)</td>
<td>100 (3500)</td>
</tr>
<tr>
<td>20 (27)</td>
<td>185 (6400)</td>
</tr>
<tr>
<td>40 (54)</td>
<td>300 (10 000)</td>
</tr>
</tbody>
</table>

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
where

\[ A_x = \text{gross area of smaller member, in.}^2 (\text{mm}^2) \]

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

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

(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 steel 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 C5.2-1 (Table C5.2M4M) shall be considered deformation-controlled and evaluated in accordance with ASCE/SEI 41, Equation 7-36, with the expected flexural strength, \( Q_{CE} = M_{CE} \), determined in accordance with Section C5.3a.1 and \( m \) taken from Table C5.2-1 (Table C5.2M4M) as modified in this section. Actions for limit states for which no values for \( m \) are provided shall be considered force-controlled.

The permissible flexural strength of FR beam-to-column moment connections shall be 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. Tabulated values for \( m \) in Table C5.2-1 (Table C5.2M4M) 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.

(a) If the connection does not satisfy at least one of the following three conditions, the tabulated value for \( m \) in Table C5.2-1 (Table C5.2M4M) shall be multiplied by 0.8.
5.2 $bf$ 
$cf$ 
bt $\geq$ 
(C5-1315)

or

(ii) 
$bf$ $t$ $< t_{bf}$ $< \frac{bf}{5.2}$ 
(C5-1416)

and

$bf$ $t$ $\geq \frac{bf}{2}$ 
(C5-1425)

or

(iii) 
$bf$ $t$ $< \frac{bf}{7}$ 
(C5-1618)

and

$bf$ $t$ $> \frac{bf}{7}$ 
(C5-1429)

where

$bf$ = width of beam flange, in. (mm)

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

(b) If the following condition is not met, the tabulated value for $m$ in Table C5.2-44 (Table C5.2-4M) shall be multiplied by 0.8. 
$V_{ye}$ shall be determined in accordance with Section C4.3a.

$0.6 \leq \frac{V_{PZ}}{V_{ye}} \leq 0.9$ 
(C5-1820)

where

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

$V_{ye}$ = expected yield shear strength of the panel zone computed in accordance with Section C4.3a, kips (N)

For $M_y$ at the face of the column, $V_{PZ}$ can is permitted to be estimated using Equation C5-1921.

$V_{PZ} = \sum_{i=beam} \left( \frac{L_{CL}}{L_{CL} - d_i} \right) \left( \frac{h_{max} - d_i}{h_{avg}} \right)$ 
(C5-1921)
where
\[ M_{\text{yield,beam}} = \text{expected first yield moment of the beam, } \frac{SF_{\text{ye}}}{\text{kip-in. (N-mm)}} \]

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

(c) If the beam flange and web slenderness satisfy the following conditions, the tabulated value for \( m \) in Table C5.2.4 (Table C5.2M4M) need not be modified.

\[ \frac{b_f}{2r_f} \leq 0.31 \sqrt{\frac{E}{F_{\text{ye}}} \quad (C5-229)} \]

and

\[ \frac{h}{t_w} \leq 2.45 \sqrt{\frac{E}{F_{\text{ye}}} \quad (C5-234)} \]

If the beam flange or web slenderness satisfy the following conditions, the tabulated value for \( m \) in Table C5.2.4 (Table C5.2M4M) shall be multiplied by 0.5.

\[ \frac{b_f}{2r_f} > 0.38 \sqrt{\frac{E}{F_{\text{ye}}} \quad (C5-242)} \]

or

\[ \frac{h}{t_w} > 3.76 \sqrt{\frac{E}{F_{\text{ye}}} \quad (C5-2325)} \]

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.

(d) If the clear span-to-depth ratio, \( L_c / d_b \), is greater-than \( \frac{L_c}{d_b} \), the tabulated value for \( m \) in Table C5.2.4 (Table C5.2M4M) shall be multiplied by the factor

\[ 1.4 - 0.04 \frac{L_c}{d_b} \left| \frac{8 - (L_c / d_b)}{3} \right| 0.5 \]
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 C5-2426.

$$M_{CLc} \geq M_{peb}$$  \hspace{1cm} \text{(C5-2426)}$$

where

- $M_{peb} =$ expected plastic flexural strength of beam, determined in accordance with Section C2.3a at the plastic hinge location, projected to the face of column, kips (N)
- $M_{CLc} =$ lower-bound flexural strength of connection at the face of the column, determined in accordance with Section C5.3b.1, kips (N)

**TABLE C5.24**

Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—FR Beam-to-Column Connections Subject to Flexure $^a,b$

<table>
<thead>
<tr>
<th>Component $^c$</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded unreinforced flange (WUF) (pre-1995) $^a$</td>
<td>1.0</td>
<td>4.3 - 0.083$d_b$</td>
</tr>
<tr>
<td>Bottom haunch in WUF with slab top flange weld removed and replaced per Table C5.1 $^a$</td>
<td>2.3</td>
<td>2.7</td>
</tr>
<tr>
<td>Bottom haunch in WUF without slab with existing top flange weld unaltered</td>
<td>1.8</td>
<td>2.1</td>
</tr>
<tr>
<td>Welded cover plate in WUF with existing flange weld removed and replaced per Table C15.1</td>
<td>1.4</td>
<td>1.7</td>
</tr>
<tr>
<td>Welded cover plate in WUF with existing flange weld unaltered $^c$</td>
<td>3.9 - 0.059$d_b$</td>
<td>4.3 - 0.067$d_b$</td>
</tr>
<tr>
<td>Improved WUF—Bolted web $^b$</td>
<td>2.0 - 0.016$d_b$</td>
<td>2.3 - 0.021$d_b$</td>
</tr>
</tbody>
</table>

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
### TABLE C5.24
Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—FR Beam-to-Column Connections Subject to Flexure $^{a,b}$

<table>
<thead>
<tr>
<th>Component $^{c}$</th>
<th>IO</th>
<th>LS</th>
<th>CP</th>
<th>LS</th>
<th>CP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Improved AISC 358 WUF-W, Welded web and other AISC 358 conforming connections.</td>
<td>3.1</td>
<td>4.2</td>
<td>5.3</td>
<td>5.3</td>
<td>6.7</td>
</tr>
<tr>
<td>Free flange</td>
<td>$4.5 - 0.065 d_b$</td>
<td>$6.3 - 0.098 d_b$</td>
<td>$8.1 - 0.129 d_b$</td>
<td>$8.4 - 0.129 d_b$</td>
<td>$11.0 - 0.172 d_b$</td>
</tr>
<tr>
<td>Reduced beam section $^{b}$</td>
<td>$3.5 - 0.016 d_b$</td>
<td>$4.9 - 0.025 d_b$</td>
<td>$6.2 - 0.032 d_b$</td>
<td>$6.5 - 0.032 d_b$</td>
<td>$8.4 - 0.032 d_b$</td>
</tr>
<tr>
<td>Welded Flange Plates</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Flange plate net section</td>
<td>2.5</td>
<td>3.3</td>
<td>4.1</td>
<td>5.7</td>
<td>7.3</td>
</tr>
<tr>
<td>b2. Other limit states</td>
<td>Force-controlled</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welded bottom haunch</td>
<td>2.3</td>
<td>3.1</td>
<td>3.8</td>
<td>4.6</td>
<td>5.9</td>
</tr>
<tr>
<td>Welded top and bottom haunch</td>
<td>2.4</td>
<td>3.1</td>
<td>3.9</td>
<td>4.7</td>
<td>6.0</td>
</tr>
<tr>
<td>Welded cover-plated flanges</td>
<td>2.5</td>
<td>2.8</td>
<td>3.4</td>
<td>3.4</td>
<td>4.2</td>
</tr>
</tbody>
</table>

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
TABULATED values shall be modified as indicated in Section C5.24a.1(a) through (d). Where values of $m$ are a function of $d_b$, they need not be taken as less than 1.0.
Refer to Table C5.1 for description of the connection.
### TABLE C5.2M4M
Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—FR Beam-to-Column Connections Subject to Flexure

<table>
<thead>
<tr>
<th>Component</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IO</td>
<td>LS</td>
</tr>
<tr>
<td>Welded unreinforced flange (WUF)—pre-1995</td>
<td>1.0</td>
<td>4.3 – 0.0033$d_b$</td>
</tr>
<tr>
<td>Bottom haunch in WUF with top flange weld removed and replaced per Table C5.1</td>
<td>2.3</td>
<td>2.7</td>
</tr>
<tr>
<td>Bottom haunch in WUF with existing top flange weld unaltered without slab</td>
<td>1.8</td>
<td>2.1</td>
</tr>
<tr>
<td>Welded cover plate in WUF with existing flange weld removed and replaced per Table C5.1</td>
<td>1.4</td>
<td>1.7</td>
</tr>
<tr>
<td>Welded cover plate in WUF with existing flange weld unaltered</td>
<td>3.9 – 0.0023$d_b$</td>
<td>4.3 – 0.0026$d_b$</td>
</tr>
<tr>
<td>Improved WUF—Bolted web</td>
<td>2.0 – 0.00063$d_b$</td>
<td>2.3 – 0.00083$d_b$</td>
</tr>
<tr>
<td>AISC 358 WUF-W and other AISC 358 conforming connections</td>
<td>Improved WUF—Welded web</td>
<td>3.1</td>
</tr>
<tr>
<td>Free flange</td>
<td>4.5 – 0.0002$d_b$</td>
<td>6.3 – 0.0003$d_b$</td>
</tr>
<tr>
<td>Reduced beam section</td>
<td>3.5 – 0.00063$d_b$</td>
<td>4.9 – 0.00098$d_b$</td>
</tr>
<tr>
<td>Welded Flange Plates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Flange plate net section</td>
<td>4.1</td>
<td>4.1</td>
</tr>
<tr>
<td>b2. Other limit states</td>
<td>Force-controlled</td>
<td></td>
</tr>
<tr>
<td>Welded bottom haunch</td>
<td>2.3</td>
<td>3.1</td>
</tr>
<tr>
<td>Welded top and bottom haunch</td>
<td>2.4</td>
<td>3.1</td>
</tr>
<tr>
<td>Welded cover—Plated flanged</td>
<td>2.5</td>
<td>2.8</td>
</tr>
</tbody>
</table>

* Tabulated values shall be modified as indicated in Section C5.4.1.a(1)(a through d).
* Where values of $m$ are a function of $d_b$, they need not be taken as less than 1.0.
* Refer to Table C5.1 for description of the connection.

#### (2) Nonlinear Analysis Procedures

For nonlinear analysis procedures, flexural behavior of FR connections identified in Table C5.3.5 (Table C5.3M) shall be considered deformation-controlled, and the plastic rotation angle, $\theta_p$, predicted by analysis shall be not greater than the permissible plastic rotation angle given in Table C5.3.5 (Table C5.3M).
Permissible flexural deformation of FR beam-to-column moment connections shall be 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. Tabulated permissible deformations in Table C5.3.5 (Table C5.3M) shall be modified in accordance with the conditions set forth in Sections C5.4a.1(a), (b), and (c), and the following condition(d). The modifications shall be cumulative.

If the clear span-to-depth ratio, $L_{cf}/d_b$, is less than 8, the plastic rotation angles provided in Table C5.3 (Table C5.3M) shall be multiplied by the factor

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

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 C5-2426.
### Table C5.35
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—
FR Beam-to-Column Connections Subject to Flexure \(^a, b\)

<table>
<thead>
<tr>
<th>Component (^c)</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Rotation Angle, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>(a)</td>
<td>(b)</td>
</tr>
<tr>
<td>WUF (pre-1995) (^b)</td>
<td>0.051–0.0013d_b</td>
<td>0.043–0.00060d_b</td>
</tr>
<tr>
<td>Bottom haunch in WUF with slab top flange weld removed and replaced per Table C5.1</td>
<td>0.026</td>
<td>0.036</td>
</tr>
<tr>
<td>Bottom haunch in WUF without slab with existing top flange weld unaltered</td>
<td>0.018</td>
<td>0.023</td>
</tr>
<tr>
<td>Welded cover plate in WUF</td>
<td>0.056–0.0011d_b</td>
<td>0.056–0.0011d_b</td>
</tr>
<tr>
<td>Welded cover plate in WUF with existing flange weld removed and replaced per Table C5.1</td>
<td>0.031</td>
<td>0.031</td>
</tr>
<tr>
<td>Welded cover plate in WUF with existing flange weld unaltered (^b)</td>
<td>0.056–0.0011d_b</td>
<td>0.056–0.0011d_b</td>
</tr>
<tr>
<td>Improved WUF—Bolted web (^b)</td>
<td>0.021–0.00030d_b</td>
<td>0.050–0.00060d_b</td>
</tr>
<tr>
<td>Improved AISC 358 WUF—Welded web and other AISC 358 conforming connections</td>
<td>0.041</td>
<td>0.054</td>
</tr>
<tr>
<td>Free flange</td>
<td>0.067–0.0012d_b</td>
<td>0.094–0.0016d_b</td>
</tr>
<tr>
<td>Reduced beam section (^b)</td>
<td>0.050–0.00030d_b</td>
<td>0.070–0.00030d_b</td>
</tr>
<tr>
<td>Welded flange plates (^a)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flange plate net section</td>
<td>0.03</td>
<td>0.06</td>
</tr>
<tr>
<td>Force-controlled</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welded bottom haunch</td>
<td>0.027</td>
<td>0.047</td>
</tr>
<tr>
<td>Welded top and bottom haunches</td>
<td>0.028</td>
<td>0.048</td>
</tr>
<tr>
<td>Welded cover—plated flanges</td>
<td>0.031</td>
<td>0.031</td>
</tr>
</tbody>
</table>

\(^a\) Values are applicable at the column face. Tabulated values shall be modified as indicated in Section C5.4a.1a(1a 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 C5.3M5M
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—FR Beam-to-Column Connections Subject to Flexure

<table>
<thead>
<tr>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic Rotation Angle, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td>Component</td>
<td>a</td>
</tr>
<tr>
<td>WUF (pre-1995)</td>
<td>0.051–0.000051d_b</td>
</tr>
<tr>
<td>Bottom haunch in WUF with top flange weld removed and replaced per Table C5.1</td>
<td>0.026</td>
</tr>
<tr>
<td>Bottom haunch in WUF with existing top flange weld unaltered without slab</td>
<td>0.018</td>
</tr>
<tr>
<td>Welded cover plate in WUF*</td>
<td>0.056–0.000043d_b</td>
</tr>
<tr>
<td>Welded cover plate in WUF with existing flange weld removed and replaced per Table C5.1</td>
<td>0.031</td>
</tr>
<tr>
<td>Welded cover plate in WUF with existing flange weld unaltered</td>
<td>0.056–0.000043d_b</td>
</tr>
<tr>
<td>Improved WUF—Bolted web</td>
<td>0.021–0.000012d_b</td>
</tr>
<tr>
<td>AISC 358 WUF-W and other AISC 358 conforming connections; Improved WUF—Bolted web</td>
<td>0.041</td>
</tr>
<tr>
<td>Welded web; Free flange*</td>
<td>0.067–0.000043d_b</td>
</tr>
</tbody>
</table>
### Table C5.3M5M
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—
FR Beam-to-Column Connections Subject to Flexure \(^n, b\)

<table>
<thead>
<tr>
<th>Component</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Modeling Parameters</td>
</tr>
<tr>
<td></td>
<td>Plastic Rotation Angle, rad</td>
</tr>
<tr>
<td></td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>Plastic Rotation Angle, rad</td>
</tr>
<tr>
<td>Reduced beam section (b)</td>
<td>(a)</td>
</tr>
<tr>
<td>Reduced beam section (b)</td>
<td>(0.050)</td>
</tr>
<tr>
<td>Welded flange plates</td>
<td>(0.0000047d_b)</td>
</tr>
<tr>
<td>Welded bottom haunch</td>
<td>(0.027)</td>
</tr>
<tr>
<td>Welded top and bottom haunches</td>
<td>(0.028)</td>
</tr>
<tr>
<td>Welded cover—plated flanges</td>
<td>(0.031)</td>
</tr>
</tbody>
</table>

\(^n\) 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.

---

### b. PR Beam-to-Column Moment Connections

#### (1) Linear Analysis procedures

For linear analysis procedures, the flexural behavior of PR connections identified in Table C5.4-4 (Table C5.4H5M) shall be considered deformation-controlled and evaluated in accordance with ASCE/SEI 41, Equation 7-36, with the expected flexural strength, \(Q_{CE} = M_{CE}\), determined in accordance with Section C5.3a.4.2 and \(m\) taken from Table C5.4-4 (Table C5.4H5M). Actions for limit states for which no values for \(m\) are provided shall be considered force-controlled.
### Table C5.46
Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—PR Beam-to-Column Connections Subject to Flexure

<table>
<thead>
<tr>
<th>Component</th>
<th>IO</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top and Bottom Flange Angle$^b$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Shear failure of rivet or bolt (limit state 1)$^c$</td>
<td>1.5</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>b2. Tension failure of horizontal leg of angle (limit state 2)</td>
<td>1.25</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>c3. Tension failure of rivet or bolt (limit state 3)$^c$</td>
<td>1.25</td>
<td>1.5</td>
<td>2.5</td>
</tr>
<tr>
<td>d4. Flexural failure of angle (limit state 4)</td>
<td>2</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>Double Split-Tee$^b$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Shear failure of rivet or bolt (limit state 1)$^c$</td>
<td>1.5</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>b2. Tension failure of rivet or bolt (limit state 2)$^c$</td>
<td>1.25</td>
<td>1.5</td>
<td>2.5</td>
</tr>
<tr>
<td>c3. Tension failure of split-tee stem (limit state 3)</td>
<td>1.25</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>d4. Flexural failure of split tee (limit state 4)</td>
<td>2</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>Bolted Flange Plate$^b$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Failure in net section of flange plate or shear failure of bolts or rivets$^b$</td>
<td>1.5</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>b2. Weld failure or tension failure on gross section of plate</td>
<td>1.25</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>Bolted End-End Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Yield of end plate</td>
<td>2</td>
<td>5.5</td>
<td>7</td>
</tr>
<tr>
<td>b2. Yield of bolts</td>
<td>1.5</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>c3. Failure of weld</td>
<td>1.25</td>
<td>1.5</td>
<td>2</td>
</tr>
</tbody>
</table>

Composite Top with Bottom Flange Angle$^b$
Table C5.46
Component Capacity Modification Factor, \( m \), for Linear Analysis Procedures—PR Beam-to-Column Connections Subject to Flexure

<table>
<thead>
<tr>
<th>Component</th>
<th>IO</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>a1. Failure of deck reinforcement</td>
<td>1.25</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>b2. Local flange yielding and web crippling of column</td>
<td>1.5</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>c3. Yield of bottom flange angle</td>
<td>1.5</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>d4. Tensile yield of rivets or bolts at column flange</td>
<td>1.25</td>
<td>1.5</td>
<td>2.5</td>
</tr>
<tr>
<td>e5. Shear yield of beam flange connections</td>
<td>1.25</td>
<td>2.5</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Shear connection Connection with slab “Slab”

<table>
<thead>
<tr>
<th></th>
<th>2.4</th>
<th>–</th>
<th>–</th>
<th>13.0–0.2 90(d_y)</th>
<th>17.0–0.387(d_y)</th>
</tr>
</thead>
</table>

Shear connection Connection without slab “Slab”

<table>
<thead>
<tr>
<th></th>
<th>8.9</th>
<th>0.19</th>
<th>–</th>
<th>–</th>
<th>13.0–0.2 90(d_y)</th>
<th>17.0–0.387(d_y)</th>
</tr>
</thead>
</table>

\(d_y\) = depth of bolt group, in.

*Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If \(d_y\) > 18 in, multiply \(m\) by 18/\(d_y\), but values need not be less than 1.0.

*For high-strength bolts, divide values by 2.0, but values need not be less than 1.25.
<table>
<thead>
<tr>
<th>Component</th>
<th>IO</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>Top and Bottom Flange Angle</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Shear failure of rivet or bolt (limit state 1)</td>
<td>1.5</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>b2. Tension failure of horizontal leg of angle (limit state 2)</td>
<td>1.25</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>c3. Tension failure of rivet or bolt (limit state 3)</td>
<td>1.25</td>
<td>1.5</td>
<td>2.5</td>
</tr>
<tr>
<td>d4. Flexural failure of angle (limit state 4)</td>
<td>2</td>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>Double Split-Tee</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Shear failure of rivet or bolt (limit state 1)</td>
<td>1.5</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>b2. Tension failure of rivet or bolt (limit state 2)</td>
<td>1.25</td>
<td>1.5</td>
<td>2.5</td>
</tr>
<tr>
<td>c3. Tension failure of split-tee stem (limit state 3)</td>
<td>1.25</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>Bolted Flange Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Failure in net section of flange plate or shear failure of bolts or rivets</td>
<td>1.5</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>b2. Weld failure or tension failure on gross section of plate</td>
<td>1.25</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>Bolted End-Plate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Yield of end plate</td>
<td>2</td>
<td>5.5</td>
<td>7</td>
</tr>
<tr>
<td>b2. Yield of bolts</td>
<td>1.5</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>c3. Failure of weld</td>
<td>1.25</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>Composite Top with Bottom Flange Angle</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a1. Failure of deck reinforcement</td>
<td>1.25</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>b2. Local flange yielding and web crippling of column</td>
<td>1.5</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>c3. Yield of bottom flange angle</td>
<td>1.5</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>d4. Tensile yield of rivets or bolts at column flange</td>
<td>1.25</td>
<td>1.5</td>
<td>2.5</td>
</tr>
<tr>
<td>e5. Shear yield of beam flange</td>
<td>1.25</td>
<td>2.5</td>
<td>3.5</td>
</tr>
</tbody>
</table>
Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—PR Beam-to-Column Connections Subject to Flexure

<table>
<thead>
<tr>
<th>Component</th>
<th>IO</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>Shear connection, Slab overweighted</td>
<td>2.4-</td>
<td>0.00043</td>
<td>$d_{bg}$</td>
</tr>
<tr>
<td>Shear connection, Slab thin</td>
<td>8.9-</td>
<td>0.0076</td>
<td>$d_{bg}$</td>
</tr>
</tbody>
</table>

$*d_{bg}$ = depth of the bolt group, mm.

Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If $d_{bg} > 450$ mm, multiply $m$ by 450/$d_{bg}$, but values need not be less than 1.0.

For high-strength bolts, divide values by 2.0, but values need not be less than 1.25.

For nonlinear analysis procedures, the flexural behavior of PR connections identified in Table C5.4M (Table C5.4M) shall be considered deformation-controlled, and the plastic rotation angle, $\theta_p$, predicted by analysis shall not be greater than the permissible plastic rotation angle given in Table C5.5M (Table C5.5M).
### Table C5.57
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—PR Beam-to-Column Connections Subject to Flexure

<table>
<thead>
<tr>
<th>Component</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Rotation Angle, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>( a )</td>
<td>( b )</td>
</tr>
</tbody>
</table>

**Top and Bottom Flange Angle**

- **Shear failure of rivet or bolt (Limit State 1)**
  - \( a_1 \)
  - \( a_2 \)
  - \( a_3 \)
  - \( a_4 \)

**Double Split-Tee**

- **Shear failure of rivet or bolt (Limit State 1)**
  - \( b_1 \)
  - \( b_2 \)
  - \( b_3 \)
  - \( b_4 \)

**Bolted Flange Plate**

- **Failure in net section of flange plate or shear failure of bolts or rivets**
  - \( c_1 \)
  - \( c_2 \)
  - \( c_3 \)
  - \( c_4 \)

**Bolted End-Plate**

- **Yield of end plate**
  - \( d_1 \)
  - \( d_2 \)
  - \( d_3 \)

**Composite Top with Bottom Flange Angle**

- **Failure of deck**
  - \( e_1 \)
### Table C5.57

Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—PR Beam-to-Column Connections Subject to Flexure

<table>
<thead>
<tr>
<th>Component</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Rotation Angle, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>reinforcement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b2. Local flange yielding and web crippling of column</td>
<td>0.036</td>
<td>0.042</td>
</tr>
<tr>
<td>c3. Yield of bottom flange angle</td>
<td>0.036</td>
<td>0.042</td>
</tr>
<tr>
<td>d4. Tensile yield of rivets or bolts at column flange</td>
<td>0.015</td>
<td>0.022</td>
</tr>
<tr>
<td>e5. Shear yield of beam-flange connection</td>
<td>0.022</td>
<td>0.027</td>
</tr>
</tbody>
</table>

**Shear connection—Connection with slab**

<table>
<thead>
<tr>
<th>Component</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Rotation Angle, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>b</td>
</tr>
<tr>
<td>Shear connection—Connection with slab*</td>
<td>0.029–0.00020d(_bg)</td>
<td>0.15–0.0036d(_bg\</td>
</tr>
</tbody>
</table>

*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.

**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\(_b\).**

For high-strength bolts, divide values by 2.0.
### Table C5.5M7M
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—PR Beam-to-Column Connections Subject to Flexure

<table>
<thead>
<tr>
<th>Component</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Rotation Angle, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>(a)</td>
<td>(b)</td>
</tr>
</tbody>
</table>

**Top and Bottom Flange Angle**

<table>
<thead>
<tr>
<th></th>
<th>Shear failure of rivet or bolt (Limit State 1)(^1)</th>
<th>Tension failure of horizontal leg of angle (Limit State 2)</th>
<th>Tension failure of rivet or bolt (Limit State 3)(^3)</th>
<th>Flexural failure of angle (Limit State 4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a_1)</td>
<td>0.036</td>
<td>0.048</td>
<td>0.200</td>
<td>0.008</td>
</tr>
<tr>
<td>(b_2)</td>
<td>0.012</td>
<td>0.018</td>
<td>0.800</td>
<td>0.003</td>
</tr>
<tr>
<td>(c_3)</td>
<td>0.016</td>
<td>0.025</td>
<td>1.000</td>
<td>0.005</td>
</tr>
<tr>
<td>(d_4)</td>
<td>0.042</td>
<td>0.084</td>
<td>0.200</td>
<td>0.010</td>
</tr>
</tbody>
</table>

**Double Split-Tee**

<table>
<thead>
<tr>
<th></th>
<th>Shear failure of rivet or bolt (Limit State 1) (^1)</th>
<th>Tension failure of rivet or bolt (Limit State 2) (^2)</th>
<th>Tension failure of split-tee stem (Limit State 3)</th>
<th>Flexural failure of split-tee (Limit State 4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a_1)</td>
<td>0.036</td>
<td>0.048</td>
<td>0.200</td>
<td>0.008</td>
</tr>
<tr>
<td>(b_2)</td>
<td>0.016</td>
<td>0.024</td>
<td>0.800</td>
<td>0.003</td>
</tr>
<tr>
<td>(c_3)</td>
<td>0.012</td>
<td>0.018</td>
<td>0.800</td>
<td>0.003</td>
</tr>
<tr>
<td>(d_4)</td>
<td>0.042</td>
<td>0.084</td>
<td>0.200</td>
<td>0.010</td>
</tr>
</tbody>
</table>

**Bolted Flange Plate**

<table>
<thead>
<tr>
<th></th>
<th>Failure in net section of flange plate or shear failure of bolts or rivets (^3)</th>
<th>Weld failure or tension failure on gross section of plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a_1)</td>
<td>0.030</td>
<td>0.030</td>
</tr>
<tr>
<td>(b_2)</td>
<td>0.012</td>
<td>0.018</td>
</tr>
</tbody>
</table>

**Bolted End-End-Plate**

<table>
<thead>
<tr>
<th></th>
<th>Yield of end plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a_1)</td>
<td>0.042</td>
</tr>
</tbody>
</table>
### Table C5.5M7M
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—PR Beam-to-Column Connections Subject to Flexure

<table>
<thead>
<tr>
<th>Component</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Rotation Angle, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>$a$</td>
<td>$b$</td>
</tr>
<tr>
<td>b₂ Yield of bolts</td>
<td>0.018</td>
<td>0.024</td>
</tr>
<tr>
<td>c₃ Failure of weld</td>
<td>0.012</td>
<td>0.018</td>
</tr>
<tr>
<td>Composite Top with Bottom Flange Angle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a₁ Failure of deck reinforcement</td>
<td>0.018</td>
<td>0.035</td>
</tr>
<tr>
<td>b₂ Local flange yielding and web crippling of column</td>
<td>0.036</td>
<td>0.042</td>
</tr>
<tr>
<td>c₃ Yield of bottom flange angle</td>
<td>0.036</td>
<td>0.042</td>
</tr>
<tr>
<td>d₄ Tensile yield of rivets or bolts at column flange</td>
<td>0.015</td>
<td>0.022</td>
</tr>
<tr>
<td>e₅ Shear yield of beam-flange connection</td>
<td>0.022</td>
<td>0.027</td>
</tr>
<tr>
<td>Shear Connection with Slab</td>
<td>0.029–$0.000079d_{bg}$</td>
<td>0.15–$0.00014d_{bg}$</td>
</tr>
<tr>
<td>Shear Connection without Slab</td>
<td>0.15–$0.00014d_{bg}$</td>
<td>0.15–$0.00014d_{bg}$</td>
</tr>
</tbody>
</table>

*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.
*Web plate or stiffened seat shall be considered to carry shear. Without shear connection, action shall not be classified as secondary. If beam depth, $d_b > 45.0$ in., multiply m by 450/$d_b$.
*For high-strength bolts, divide values by 2.0.

### 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 strength of the connection for
a given action shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound component strength, \( Q_{CL} \), of the connection 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, the strength of the connection for a given action shall be evaluated using ASCE/SEI 41, Equation 7-38, with \( Q_{CL} \), of the connection determined in accordance with Section C5.3b.

Alternatively, when a force-controlled action, for the connection, is explicitly modeled with a nonlinear force-deformation behavior, it is permitted to evaluate the connection based on deformation. When nonlinear analysis procedures are used and the flexural, shear, or axial behavior of a connection is considered force-controlled for such an evaluation, the total deformation of the connection for a given action shall not exceed the yield deformation for that action defined in Figure C1.1 calculated based on principles of structural mechanics using expected material properties.

The lower-bound connection strength, \( Q_{CL} \), for a given action determined in accordance with Section C5.3b shall not be less than the maximum force for that action determined by ASCE/SEI 41, Section 2.5.3.2.3.2. FR and PR Beam-to-Column Moment Connections and Beam-to-Beam 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.

### 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.

Where required by the classification of the connection, the expected and lower-bound strengths of a connection 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.

Where required by the classification of the connection, the expected or lower-bound strengths of a column-to-base connection 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.

Where required by the classification of the connection, the expected or lower-bound strengths of the anchor rod connection between the column-to-base connection and the concrete substratum 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), using $\phi = 1.00$, 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.
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 thin-plate shear walls.

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 deformations (e.g., flexural, shear) that affect its behavior.

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 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 stiffened plate wall, $K_w$, shall be determined in accordance with Equation C6-1 unless another method based on principles of mechanics is used.

$$K_w = \frac{Gat_w}{h}$$

(C6-1)

where

$a$ = clear width of wall between vertical boundary elements, 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 mechanics or analysis.

3. Strength

The shear strength of steel plate shear walls shall be determined in accordance with this section. For determination of shear strength, the plate wall shall be modeled as the web of a plate girder.

The shear strength of the concrete encasement or backing can be included in the shear strength of the 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 Action

The expected shear strength, $V_{CE}$, of a 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 be sufficiently stiffened to prevent shear buckling. In this case, $Q_{CE} = Q_s = V_{CE}$. Stiffener strength, stiffness and spacing shall be in accordance with the requirements for plate 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 can be computed taking $h/t_w$ in Specification, Chapter G, as equal to zero.

3b. Force-Controlled Action

The lower-bound shear strength, $V_{CL}$, of a plate wall shall be determined using equations for nominal shear strength, $V_n$, given in Specification, Chapter G, except that $F_{yLB}$ 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 Chapter Section F5, except that $F_{yLB}$ 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 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 shear behavior shall be evaluated using ASCE/SEI 41, Equation 7-36, with the expected shear strength, $Q_{CE} = V_{CE}$, determined in accordance with Section C6.3a and $m$ taken from Table C6.1.

<table>
<thead>
<tr>
<th>Component</th>
<th>IO</th>
<th>LS</th>
<th>CP</th>
<th>LS</th>
<th>CP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Plate Shear Walls</td>
<td>1.5</td>
<td>8</td>
<td>12</td>
<td>12</td>
<td>14</td>
</tr>
</tbody>
</table>

Table C6.1
Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—Steel Plate Shear Walls Subject to Shear

- 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

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 may be derived from experiment testing or analysis. For steel plate shear walls, a strain-hardening slope is permitted to take $\alpha_h$ for shear action of $\leq 6\%$ of the elastic slope is permitted. Further modification of the curve is permitted if a greater strain-hardening slope value of $\alpha_h$ is justified by experiment testing or analysis.

When the shear strength of a steel plate shear wall is considered deformation-controlled, the plastic shear deformation demand, $\gamma_p$, 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, $\gamma_y$, of a steel plate shear wall shall be determined from Equation C6-2.

$$\gamma_y = \frac{V_{CE}}{K_w \cdot h}$$  \hspace{1cm} (C6-2)

where

- $K_w = $ elastic shear stiffness of stiffened plate wall, determined in accordance with Section 2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings

Draft Ballot 3 Dated September 30, 2019

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
\[ V_{CE} = \text{expected shear strength of the steel plate shear wall determined in accordance with Section C6.3a, kips (N)} \]

### TABLE C6.2

<table>
<thead>
<tr>
<th>Component</th>
<th>Modeling Parameters</th>
<th>Expected Deformation Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Shear Deformation, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td>Steel Plate Shear Walls a</td>
<td>a ( \gamma_y )</td>
<td>b ( \gamma_y )</td>
</tr>
</tbody>
</table>

*a Applicable if stiffeners, concrete encasement or backing are provided to prevent shear buckling.

### 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 shear behavior shall be evaluated using Equation 7-37 of ASCE/SEI 41 with the lower-bound shear strength, \( Q_{CL} = V_{CL} \), 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 shear behavior shall be evaluated using Equation 7-38 of ASCE/SEI 41 with the lower-bound shear strength, \( Q_{CL} = V_{CL} \), determined in accordance with Section C6.3b.

Alternatively, when a force-controlled action, for the steel plate shear wall, is explicitly modeled with a nonlinear force-deformation behavior, it is permitted to evaluate the steel plate shear wall based on deformation. When nonlinear analysis procedures are used and the shear behavior of a steel plate shear wall is considered force-controlled, the total shear deformation, \( \gamma \), of the steel plate shear wall predicted by analysis shall not exceed \( \gamma \), determined from Equation C6-2.

The lower-bound shear strength, \( Q_{CL} = V_{CL} \), determined in accordance with Section C6.3b shall
not be less than the maximum force determined by ASCE/SEI 41, Section 7.5.3.3.
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: CBF, EBF and BRBF, as discussed in Chapter E. In these elements, braced frame connections are located at the brace-beam-column and/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 complex combined stress state loading conditions resulting from (1) axial tension and/or compression of the brace(s); (2) flexural post-buckling demands for frames with buckling braces; 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

Braced-frame connection stiffness shall be based on principles of structural mechanics and as specified in the Specification unless superseded by provisions of this section.

2a. Rotation-Restrained Connections at Brace Ends

Where brace buckling is implicitly (phenomenologically) modeled using the parameters defined in Section C3.2 and described in Section E1.2b, the rotational stiffness of connections, which accommodate end rotation of the brace, is permitted to be evaluated as rigid. When brace buckling is implicitly modeled using the parameters defined in Section C3.2 and described in Section E2b.1, the rotational stiffness of connections, which accommodate end rotation of the brace, are permitted to be evaluated as rigid.

2b. Rotation-Accommodating Connections at Brace Ends

Where when brace buckling is implicitly (phenomenologically) modeled using the parameters defined in Section C3.2 and described in Section E1.2b, the rotational stiffness of connections, which accommodate end rotation of the brace, is permitted to be considered evaluated as essentially rigid, except to account for effects on adjacent members as required by Section C7.2c.

Where when brace buckling is explicitly modeled using nonlinear beam-column elements, rotational stiffness of rotation-accommodating connections, such as gusset plates or knife plates, shall be computed based on the Whitmore section effective gusset plate width, \( B_w \), Figure C7.1a, and the average unrestrained length of the gusset plate, \( L_{avg} \), using a projection angle of 30 degrees, as shown in Figure C7.1b. The average unrestrained length of gusset plate, \( L_{avg} \), shall 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 (\( L_2 \)). If the Whitmore width end intersects the adjacent member, the corresponding length shall be taken as a negative value. For knife plate connections, the average unrestrained length shall be the linear clearance in the knife plate as shown in Figure C7.2.
**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.

![Diagram](image)

**Fig. C7.1.** Whitmore section for evaluation of gusset plate axial and flexural actions.

![Diagram](image)

**Fig. C7.2.** Average unrestrained length for knife plate connections.

The elastic rotational spring stiffness, $K_\theta$, in the direction plane of brace buckling shall be determined as:

$$ K_\theta = \frac{EA_t t_p^2}{12 L_{avg}} $$

(C7-1)

where

- $A_g = B_w t_p$ = gross area of gusset plate, in.$^2$ (mm$^2$)
- $B_w$ = Whitmore width using 30° projection 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. **Increased Stiffness of Adjacent Members: Modeling of Beam-to-Column Joint**

The restraint of braces, gusset plates, and other elements connected directly to beams or columns shall be considered in analysis.

Where gusset plates join beams and columns and the gusset-plate thickness is greater than or equal to 0.75$t_w$ (where $t_w$ is the greater web thickness) of both the beam and the column, the beam-to-column joint connection shall be modeled as fully restrained unless justified otherwise by rational analysis. For gusset plates welded directly to beams or columns, rigid or very stiff elastic 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 C7.1A, unless justified otherwise by rational analysis, as shown in Figure C7.3.

**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.

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

![Fig. C7.1A](image)

**Fig. C7.1A** 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* unless superseded by provisions of this section.

3a. **Deformation-Controlled Actions**

1. **Welded Gusset-Plate Rotation**

Welds connecting rotation-accommodating gusset plates connections shall be designated as gusset-plate interface welds.
2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
The effective width of a knife plate may be determined similarly, but is often restricted by the gross width of the plate.

1. **Gross Section Yielding in Tension**

   **Yield strength**, \( T_{CL} \), of a connecting plate shall be determined using the nominal axial strength, \( P_e \), determined from Specification Equation D2-1, except that \( F_y^{LB} \) shall be substituted for \( F_y \). \( T_{CL} \) shall be evaluated in the direction of the brace, where the gross area is the minimum of the full area of the plate and the area of the Whitmore section, in accordance with Equation C7-4, and the lower-bound strength, \( Q_{CL} = T_{CL} \). The Whitmore section shall be determined as in Figure C7.1a, using a projection angle of 37 degrees.

   \[
   T_{CL} = F_y^{LB} A_t \quad \text{(C7-4)}
   \]

   where
   - \( A_t \) = gross cross-sectional area of plate, in.\(^2\) (mm\(^2\))
   - \( B_{w,37} \) = Whitmore width using 37° projection, in. (mm)
   - \( t_p \) = thickness of plate, in. (mm)

2. **Net Section Fracture in Tension**

   **Rupture strength**, \( T_{CL} \), of a brace or connecting plate shall be evaluated determined using 75% of the nominal axial strength, \( P_e \), determined from Specification Equation D2-2, except that \( F_u^{LB} \) shall be substituted for the specified minimum tensile strength, \( F_u \). \( T_{CL} \) shall be evaluated based on rupture of the net areas in shear and tension using Equation C7-5, and the lower-bound strength, \( Q_{CL} = T_{CL} \), determined using the nominal strength, \( R_{CL} \), determined from Specification Equation J4-5, except that the lower bound properties, \( F_y^{LB} \) and \( F_u^{LB} \), shall be substituted for \( F_y \) and \( F_u \), respectively.

   \[
   T_{CL} = F_y^{LB} A_{nt} + 0.6 A_{nv} \quad \text{(C7-5)}
   \]

   where
   - \( A_{nt} \) = net area subject to tension, in.\(^2\) (mm\(^2\))
   - \( A_{nv} \) = net area subject to shear, in.\(^2\) (mm\(^2\))
   - \( F_y^{LB} \) = lower-bound tensile strength, ksi (MPa)

3. **Block Shear Rupture in Tension**

   Block shear rupture strength, \( T_{CL} \), of a connecting plate shall be evaluated based on rupture of the net areas in shear and tension using Equation C7-5, and the lower-bound strength, \( Q_{CL} = T_{CL} \), determined using the nominal strength, \( R_{CL} \), determined from Specification Equation J4-5, except that the lower bound properties, \( F_y^{LB} \) and \( F_u^{LB} \), shall be substituted for \( F_y \) and \( F_u \), respectively.

4. **Flexural Buckling and Gross Yielding in Compression**

   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 C7-43, and the lower-bound strength, \( Q_{CL} = P_{CL} \). The gross area shall be the minimum of the full area of the plate and the area of the Whitmore section. The Whitmore section and length for flexural buckling shall be determined as shown in Figure C7.1b using a projection angle of 30°.

---

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
The critical stress shall be computed using Specification Section E3, where the effective length, $L_e$, is permitted to be computed as $KL_{avg}$, where $L_{avg}$ is defined in Section C7.2b. If $L_{avg}$ is less than or equal to zero, the limit state of gross yielding in compression shall govern the design. 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.

$$P_{CL} = F_{cLB}A_g \leq F_{yLB}A_g$$  \hfill (C7-62)

where

- $A_g = B_{gusset, p}$ = gross section area of the plate, in.$^2$ (mm.$^2$)
- $F_{cLB}$ = critical stress of the plate computed using $F_{yLB}$, ksi (MPa)
- $F_{yLB}$ = lower-bound yield stress, ksi (MPa)

5. Bolted Connections in Shear

Lower-bound shear strength, $V_{CL}$, of bolted connections shall be determined in accordance with Specification Section J3, and the lower-bound strength, $Q_{CL} = V_{CL}$. 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 the equations in Specification Section J3.10, where 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 which are designated as rotation-restrained shall be considered force controlled.

4a. Deformation-Controlled Actions

1. Welded Gusset Plate Rotation

Welded gusset plate rotation capacity, $\theta_{gp}$, shall be determined based on (i) type of interface weld; (ii) demand critical compliance with the toughness requirements of the Seismic Provisions, Section A3.4a; (iii) ratio of the yield strength of the gusset plate, $f_{ygp}$, to the tensile strength of the weld group, $f_{tCE}$, with the limit on the ratio, $f_{ygp}/f_{tCE}$, defined in the following; and (iv) rotational clearance, $L_{ell}$, defined in Figure C7.4a 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 C7.4b. 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., is:

$$f_{yUD} = F_{yLP}$$  \hfill (C7-24)

(a) The rotational capacity of gusset plates with interface welds conforming to the requirements of Section F2.6c.4 of the Seismic Provisions, including the use of demand critical weld metal, need not be evaluated, and with toughness requirements of Section A3.4a of the Seismic Provisions, need not be evaluated.
The rotational capacity of gusset plates with interface welds formed with fillet welds that do not meet demand critical requirements need not be considered where both of the following conditions are satisfied:

The connection has been retrofitted with weld overlay using filler metal that meets demand-critical weld requirements such that the combined strength of the new and existing filler metal satisfies the toughness requirements of the Seismic Provisions, Section A3.4.4, shall be determined from Table C7.1 using \( f_{ucn}/f_{cc} \leq 0.75 \).

The clearance between the brace end and restraining edge satisfies \( L_{re}/f \geq 4 \).

(b) The rotational capacity of gusset plates with interface welds formed made with CJP groove welds that meet the requirements of the Specification but do not meet demand critical requirements of the toughness requirements of the Seismic Provisions, Section A3.4.4, shall be determined from Table C7.1 using \( f_{ucn}/f_{cc} = 0.75 \).

(c) The rotational capacity of gusset plates with interface welds formed made with fillet welds which do not meet demand critical requirements of the toughness requirements of the Seismic Provisions, Section A3.4.4, shall be determined from Table C7.1.

![Fig. C7.24. Gusset-plate clearance models.](image)

2.1 Linear Analysis Procedures

Where the rotational capacity of the gusset plate is a consideration, \( m \) for computing permissible performance parameters of the brace in both tension and compression in Table C3.2 shall use the modification factor for connection robustness, \( n_p \), from Equation C7-85, when \( n_p \) is less than \( m \). In Equation C7-85, \( \theta_{gp} \) is equal to \( d \) as computed from Table C7.1 and based on the requirements of Section C7.4a.1.

\[
 n_p = \frac{L_{ee} \theta_{gp}^2}{2 \Delta_c} \geq 1, \quad \frac{L_{ee} \Delta_c \theta_{gp}^2}{2 \Delta_c} \geq 1 \quad (C7-85)
\]

where

- \( L_{ee} \) = end-to-end brace length, effective length, defined in Section C3.2a.3a.1, in.
- \( \Delta_c \) = axial deformation at expected compressive buckling strength, determined using Equation C3-46
- \( \theta_{gp} \) = welded gusset plate rotation capacity, rad

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
3.2. Nonlinear Analysis Procedures

Where 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 shall be modified such that $n \Delta_y \leq L B_y^2 / 2$, where $\Delta_y$ is equal to $\Delta_F$ or $\Delta_C$ for braces in tension or compression, respectively.

Where When brace buckling is modeled using nonlinear beam-column elements capable of simulating member buckling, the modeling parameters 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 deformations shall be determined from Table C7.1.

### TABLE C7.1
Modeling Parameters and Permissible Performance Parameters for Nonlinear Analysis Procedures—Braced-Frame Connections

<table>
<thead>
<tr>
<th>Component/Action</th>
<th>Modeling Parameters</th>
<th>Permissible Performance Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded Gusset-Plate Rotation*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$d = 0.11 \left( \frac{L_e}{t_p} \right)^{0.33} \left( \frac{f_{ud}}{f_{ce}} \right)^{0.57}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$f^*$</td>
<td>IO</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>1.5$M_{ce}/K_i \leq 0.75$</td>
</tr>
</tbody>
</table>

* $CP$ = 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
* $f^*$ = resistance immediately prior to fracture (see Figure C3.1)

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 strength of the connection component for a given action shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound strength, $Q_{CL}$, of the connection determined in accordance with Section C7.3b.

For braced-frame connection actions that are considered force-controlled, the strength of the connection components shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound component strength, $Q_{CL}$, calculated according to 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 up to 1.2.
(b) For bolt groups loaded in shear which 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-capacity ratio for bolt fracture in shear is permitted to be up to 1.3.

(c) For bolt groups loaded in shear which are not the sole load-transfer mechanism from the brace and frame, the demand-capacity ratio for bearing and tearout at bolt holes in the connected material is permitted to be up to 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 strength of the connection component for a given action shall be evaluated using ASCE/SEI 41, Equation 7-38, with the lower-bound strength, \( Q_{CL} \), of the connection determined in accordance with Section C7.3b. When braced-frame connection actions are considered force-controlled, the strength of the connection shall be evaluated using ASCE/SEI 41, Section 7.5.3.2.2. The lower-bound component strength, \( Q_{CL} \), calculated according Section C7.3b, shall not be less than the maximum force determined by analysis.

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 up to 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 which 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 capacity ratio for bolt fracture in shear is permitted to be up to 1.3.

(c) For bolt groups loaded in shear which 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 up to 1.1.
CHAPTER D

STRUCTURAL STEEL MOMENT FRAMES

Steel moment frames develop their seismic resistance through bending of steel 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. Moment Frames

D1. GENERAL

Table D1.1 shall be used to identify the various connection types for which permissible performance parameters are provided in Section C5. Moment frames shall consist of beams and columns connected by one or more of the connection types defined in Table C45.1 or Table C45.2. 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 Section D2.

<table>
<thead>
<tr>
<th>Connection</th>
<th>Description</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded unreinforced flange (WUF)</td>
<td>Complete joint penetration groove welds between beam and column flanges, bolted or welded web, designed before code changes that followed the Northridge earthquake</td>
<td>FR</td>
</tr>
<tr>
<td>Bottom haunch in WUF with slab</td>
<td>Welded bottom haunch added to existing WUF connection with composite slab</td>
<td>FR</td>
</tr>
<tr>
<td>Bottom haunch in WUF without slab</td>
<td>Welded bottom haunch added to existing WUF connection without composite slab</td>
<td>FR</td>
</tr>
<tr>
<td>Welded cover plate in WUF</td>
<td>Welded cover plates added to existing WUF connection</td>
<td>FR</td>
</tr>
<tr>
<td>Improved WUF—Bolted web</td>
<td>Complete joint penetration groove welds between beam and column flanges, bolted web</td>
<td>FR</td>
</tr>
<tr>
<td>Improved WUF—Welded web</td>
<td>Complete joint penetration groove welds between beam and column flanges, welded web</td>
<td>FR</td>
</tr>
<tr>
<td>Free flange</td>
<td>Web is coped at ends of beam to separate flanges, welded</td>
<td>FR</td>
</tr>
</tbody>
</table>
### TABLE D1.1
Structural Steel Moment Frame Beam-to-Column Connection Types

<table>
<thead>
<tr>
<th>Connection</th>
<th>Description</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded flange plates</td>
<td>Flange plate with complete-joint-penetration groove weld at column and fillet welded to beam flange</td>
<td>FR</td>
</tr>
<tr>
<td>Reduced beam section</td>
<td>Connection in which net area of beam flange is reduced to force plastic hinging away from column face</td>
<td>FR</td>
</tr>
<tr>
<td>Welded bottom haunch</td>
<td>Haunched connection at bottom flange only</td>
<td>FR</td>
</tr>
<tr>
<td>Welded top and bottom haunches</td>
<td>Haunched connection at top and bottom flanges</td>
<td>FR</td>
</tr>
<tr>
<td>Welded cover - Plated flanges</td>
<td>Beam flange and cover plate are welded to column flange</td>
<td>FR</td>
</tr>
<tr>
<td>Bolted end-plate in conformance with ANSI/AISC 358</td>
<td>Stiffened or unstiffened end plate welded to beam and bolted to column flange</td>
<td>FR</td>
</tr>
<tr>
<td>Top and bottom clip angles</td>
<td>Clip angle bolted or riveted to beam flange and column flange</td>
<td>PR</td>
</tr>
<tr>
<td>Double split Tee</td>
<td>Split Tees bolted or riveted to beam flange and column flange</td>
<td>FR/PR</td>
</tr>
<tr>
<td>Composite top and clip-angle bottom</td>
<td>Clip angle bolted or riveted to column flange and beam bottom flange with composite slab</td>
<td>PR</td>
</tr>
<tr>
<td>Bolted flange plates</td>
<td>Flange plate with complete-joint-penetration groove weld at column and bolted to beam flange</td>
<td>PR*</td>
</tr>
<tr>
<td>Bolted end-plate</td>
<td>Stiffened or unstiffened end plate welded to beam and bolted to column flange</td>
<td>PR*</td>
</tr>
<tr>
<td>Shear connection with slab</td>
<td>Simple connection with single-plate connection, composite slab</td>
<td>PR</td>
</tr>
<tr>
<td>Shear connection without slab</td>
<td>Simple connection with single-plate connection, no composite slab</td>
<td>PR</td>
</tr>
</tbody>
</table>

*Where not indicated otherwise, definition applies to connections bolted or welded to the web of a member.
*Where not indicated otherwise, definition applies to connections with or without composite slab.
*Complete-joint-penetration groove welds between haunch or cover plate to the column flange shall conform to the requirements of the Seismic Provisions.
**Complete-joint-penetration groove welds shall conform to the requirements of the Seismic Provisions and connection shall meet AISC 358 WUF-W.
***For purposes of modeling, the double split tee beam-to-column connection is permitted to be considered FR if it satisfies the strength and connection deformation requirements of Section D2.1 or those in AISC 358.
D2. MOMENT FRAMES

1. General

The beam-to-column moment connection type shall be in accordance with Table D1C1.1 or C1.2FR or PR per Section C5.

Moment frames with beam-to-column connections not included in Table D1C1.1 shall be designated as frames with FR beam-to-column moment connections if the connection deformations (not including panel zone deformation) 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. If either of these conditions is not satisfied, the beam-to-column connection shall be characterized as PR.

2. Stiffness

2a. 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.

2b. Nonlinear Static Procedure

If the nonlinear static procedure is used, in addition to the criteria given in Section D2.2a, the following criteria shall apply:

(a) Elastic stiffness properties of components shall be modeled as specified in Section D2.2a. Flexural stiffness, $E I_c$, of columns with $P > 0.5P_{ye}$ shall be modified by $\tau_b$ in Specification Chapter C, where:

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

$E = \text{modulus of elasticity of steel} = 29,000 \text{ ksi (200 000 MPa)}$

$F_{ye} = \text{expected yield stress determined in accordance with Chapter A, ksi (MPa)}$

$I_c = \text{moment of inertia of column about the axis of bending, in.}^4 (\text{mm}^4)$

$P = \text{axial force in the column (compression or tension), kips (N)}$

$P_{ye} = \text{expected axial yield strength of the column} = A_g F_{ye}, \text{kips (N)}$

$\tau_b$ is determined from Equations C3-13a 15a and C3-13b15b

(b) Plasticity in components shall be represented in the mathematical analytical model by nonlinear force-deformation relationships, incorporating multi-force interaction effects where needed, derived experimentally or from analysis from testing or analysis; and

(c) Nonlinear phenomena behavior specific to a component not addressed above (e.g., panel zone shear
deformations, bolt slippage, and composite action) that can influence the stiffness of a component by
more than 5% shall be addressed explicitly or implicitly in the mathematical-analytical model.

2c. Nonlinear Dynamic Procedure

If the nonlinear dynamic procedure is used, the complete hysteretic behavior of each component shall be
determined by testing or by procedures approved by the authority having jurisdiction.

If experimental data are not available for the formulation of component strength or deformation capacity, it
is permitted to use the component parameters described in Section D2.2b for modeling the force-
deformation behavior or backbone curve. Modelling of strength, stiffness, and deformation capacity
degradation, and similar phenomena, is required only if demands imply significant inelastic action. The
hysteretic load and deformation paths shall not cross beyond the backbone curve.

3. Strength

3a. General

Component strengths shall be determined in accordance with Section B2 and Chapter C and the
requirements of this section. Classification of component actions as deformation-controlled or force-
controlled shall be in accordance with Section D2.4.

3b. Linear Analysis Procedures

There are no additional requirements beyond those specified in Chapter C.

3c. Nonlinear Static Procedure

There are no additional requirements beyond those specified in Chapter C.

3d. Nonlinear Dynamic Procedure

There are no additional requirements beyond those specified in Chapter C.

4. Permissible Performance Parameters

4a. General

Component permissible strengths and deformations shall be determined in accordance with Section B2 and
this section.

Unless required otherwise by project-specific requirements, the following criteria shall apply for
assessment of moment frames.

(a) Flexure actions in FR and PR beam-to-column moment connections listed in Tables D4C5.1 and C5.2

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
American Institute of Steel Construction
shall be considered deformation-controlled actions.

(b) Flexural actions in columns shall be taken as force controlled where $P_{G}/P_{ye}$ is greater than 0.6, where $P_{G}$ is the axial force component of the gravity load as determined by ASCE/SEI 41 Equation 7.3.

Flexural actions in columns where $P_{G}/P_{ye}$ is less than 0.6 shall be considered deformation controlled.

(c) Axial compression action is force-controlled for all components.

(d) Shear actions in panel zones and beams are considered deformation-controlled, and shear actions in beams, columns, and FR and PR beam-to-column connections are 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.

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 moment frames with FR and PR beam-to-column moment connections shall satisfy the requirements of 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 having eccentricity, \( e \). Bending caused by such eccentricities shall be considered in the modeling and evaluation of the components. In addition, connection response as well as beam and column behavior have a strong influence on seismic performance.

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 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 brace frame response or be calibrated to accurately simulate the analytical force-deformation envelope 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 Sections B, C2, C3, and C7.

**User Note:** The commentary provides modeling methods, which have been documented to provide acceptable accuracy, for the generalized force-deformation relation 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.

The resistance of CBF shall be defined as the smaller of the brace or brace-to-gusset-plate connections combined with frame action of the beams and columns, as defined in Chapter C. The resistance of the brace shall be the expected brace resistance for braced frames if all requirements of the Seismic Provisions are satisfied. For all other frames, the resistance of the brace shall be the lesser resistance based on the characteristics of the brace as defined in Section C3 or the braced-frame connections as defined in Section C7.

For 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 used 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 light-weight 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.

Unless required otherwise by project-specific requirements, the following criteria shall apply for assessment of CBF:

(a) Axial tension and compression actions in braces shall be considered deformation-controlled.

(b) Flexure actions in beams shall be considered deformation-controlled as determined for each component in Chapter C.

(c) Axial compression action in columns shall be considered force-controlled.

(d) Flexural actions in columns shall be taken as force controlled when \( P/P_{ye} \) is greater than 0.6, where \( P \) is the axial force, \( P_{ye} = A_{g}F_{ye} \) = the expected axial yield strength, \( A_{g} \) is the gross area of the cross-section, and \( F_{ye} \) is the expected yield stress. Flexural actions in columns where \( P/P_{ye} \) is less than 0.6 shall be considered deformation controlled.

(e) 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.

(f) 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 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.

Demands for columns in CBF shall be determined from:

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

(b) An analysis in which all braces in tension are assumed to resist forces corresponding to their expected tensile strength and all braces in compression are assumed to resist 30% of the expected compressive strength, where the expected brace strengths are as defined in Section C3.

Actions in beams, beam connections, and supporting members in V-type or inverted V-type 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

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
American Institute of Steel Construction
expected yield strength of the brace in tension with 30% of the expected compressive strength of the adjacent brace in compression, where the expected brace strengths are as defined in Section C3.

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 Section E1, 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 width of the smallest member joined at the connection. The component between these points, referred to as the link beam component, 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, in addition to the criteria given in Section E2.2a, the following criteria 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.

2c. Nonlinear Dynamic Procedure

If the nonlinear dynamic procedure is used, the complete hysteretic behavior of each component shall be determined by testing or by other procedures approved by the authority having jurisdiction.

If testing data are not available for the formulation of component strength, the component force-deformation parameters described in Section E2.2b for modeling the force-deformation behavior, or backbone curve, and applying hysteretic rules for corresponding components is permitted. The hysteretic load and deformation paths shall not cross beyond the backbone curve. The characteristics of the hysteretic loops, including cyclic stiffness degradation in unloading and reloading, cyclic strength degradation, and in-cycle strength degradation, shall be represented in the modeling if exact cyclic degradation slopes vary for different components.

3. Strength

Component strengths of EBF shall be determined in accordance with Section B2 and Chapter C and this section. 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.

Unless required otherwise by project-specific requirements, 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 all actions and $m$ for shear action in Section C3.4a.3 shall reduce to unity, where $P_{UF}$ is the axial force in the member, determined in accordance with ASCE/SEI 41, Section 7.5.2.1.2. 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, $P/P_{ye}$, greater than 0.6 shall remain elastic for all actions and the permissible plastic rotation angles for shear action in Section C3.4a.3 will reduce to zero, where $P$ is axial force in the column (compression or tension), kips (N). This provision is applicable for both compression and tension axial forces.

5. Retrofit Measures

Seismic retrofit measures for EBF shall satisfy the requirements of Section E2, 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 elements with buckling-restrained braces (BRB) that are composed of a structural steel core and a casing system that restrains the core from buckling. BRBF are evaluated and designed as capacity-based elements with the BRB casing system, connections, and adjoining members designed to resist the maximum forces that the BRB core can develop.

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.

(c) Panel zones, if applicable, shall be modeled as specified in Section C4.

(d) Columns and braces shall be modeled as specified in Section C3.

(e) 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.

(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, in addition to the criteria given in Section E3.2a, 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 $\Delta$ 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, the complete hysteretic behavior of each component shall be
determined by testing or by procedures approved by the authority having jurisdiction.

If testing data are not available for the formulation of component strength, the component force-
deformation parameters described in Section E3.2b for modeling the force-deformation behavior, or
backbone curve and applying hysteretic rules for corresponding components is permitted. The hysteretic
load and deformation paths shall not cross beyond the backbone curve. The characteristics of the hysteretic
loops, including cyclic stiffness degradation in unloading and reloading, cyclic strength degradation, and
in-cycle strength degradation, shall be represented in the modeling if exact cyclic degradation slopes vary
for different components.
3. Strength

3a. General

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. The maximum force that the BRB core can develop shall include material strain-hardening effects and an adjustment to account for compression overstrength with respect to tension strength.

3b. Linear Analysis Procedures

There are no additional requirements beyond those specified in Chapter C.

3c. Nonlinear Static Procedure

There are no additional requirements beyond those specified in Chapter C.

3d. Nonlinear Dynamic Procedure

There are no additional requirements beyond those specified in Chapter C.

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.

Unless required otherwise by project-specific requirements, 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 force- or deformation-controlled as determined for each component in Chapter C.
(c) Axial compression action in columns is force-controlled.
(d) Shear actions in panel zones are considered deformation-controlled, and shear actions in beams, columns, and FR and PR beam-to-column connections are 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 shall only be permitted if testing in accordance with *Seismic Provisions*, Section K3, as a minimum, is submitted. The deformation term \( \Delta_{\text{target}} \) 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 as required 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*, taking \( \phi = 1.00 \), 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 are 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) Use of a plane stress finite element plate wall integral with beams and columns as boundary elements is permitted to analyze a structural steel plate shear wall. Steel plate used as shear wall, 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 mechanics for this type of element.

2b. **Nonlinear Static Procedure**

If the nonlinear static procedure is used, in addition to the criteria given in Section E4.2a, 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, the complete hysteretic behavior of each component shall be determined by testing or by procedures approved by the authority having jurisdiction.

If testing data are not available for the formulation of component strength, the component force-deformation parameters described in Section E4.2b for modeling the force-deformation behavior or backbone curve and applying hysteretic rules for corresponding components is permitted. The hysteretic load and deformation paths shall not cross beyond the backbone curve. The characteristics of the hysteretic loops, including cyclic stiffness degradation in unloading and reloading, cyclic strength degradation, and in-cycle strength degradation, shall be represented in the modeling if exact cyclic degradation slopes vary for different components.
3. Strength

3a. General

Component strengths of steel plate shear walls shall be determined in accordance with Section B2 and Chapter C, and the requirements of this section. Classification of component actions as deformation-controlled or force-controlled shall be in accordance with Section E4.4.

3b. Linear Analysis Procedures

There are no additional requirements beyond those specified in Chapter C.

3c. Nonlinear Static Procedure

There are no additional requirements beyond those specified in Chapter C.

3d. Nonlinear Dynamic Procedure

There are no additional requirements beyond those specified in Chapter C.

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.

Unless required otherwise by project-specific requirements, the following criteria shall apply for assessment of steel plate shear walls:

(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 as determined for each component in Chapter C.
(c) Axial compression action in columns is 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 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

Fundamental considerations for diaphragms are provided in ASCE/SEI 41, Chapter 7, includes provisions for classification of diaphragms, mathematical modeling, diaphragm chords, diaphragm collectors, and diaphragm ties, including those related to chords and collectors, and ties and out-of-plane wall anchorage, as applicable. 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 for diaphragm elements including steel truss diaphragms and archaic diaphragms are covered. Guidance is also provided on 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 Nonstructural Lightweight Insulating Concrete
G4. Horizontal Steel Bracing (Steel Truss Diaphragms)
G5. Archaic Diaphragms
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 seismic loads acting alone or in conjunction with supplementary diagonal bracing 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/or yielding, and local deformations at side-lap and structural (support) collectors.
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 side-lap or structural 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 side-lap or structural 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 strength shall be evaluated using ASCE/SEI 41, Equation 7-36, with the expected component strength, \( Q_{CE} \), determined from Section 2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings Draft Ballot 3 Dated September 30, 2019 American Institute of Steel Construction.
G1.3a and \( m \) taken from Table G1.1.

### Table G1.1

<table>
<thead>
<tr>
<th>Component or Action</th>
<th>Component or Action</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IO</td>
<td>LS</td>
<td>CP</td>
</tr>
<tr>
<td>Shear strength controlled by connectors(^a):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>support: PAF; side-lap: screw</td>
<td>1.0</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td>support: weld; side-lap: screw</td>
<td>1.0</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td>support: weld; side-lap: button punch</td>
<td>1.0</td>
<td>1.0</td>
<td>1.3</td>
</tr>
<tr>
<td>support: weld; side-lap: weld</td>
<td>1.0</td>
<td>1.3</td>
<td>1.6</td>
</tr>
<tr>
<td>Shear strength controlled by panel:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>buckling</td>
<td>1.25</td>
<td>2.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>

\( CP = \) 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 panels with spans between supports with fasteners greater than 60 in. (1500 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).

2. Nonlinear Analysis Procedures

The generalized force-deformation curve shown in Figure C1.1, with the modeling parameters \( d, e, c \) as defined in Table G1.2, shall be used for bare steel deck diaphragms, or these relationships may be derived experimentally or from analysis from testing or analysis.

When the shear strength of a bare steel deck diaphragm is considered deformation-controlled, the plastic shear deformation, \( \gamma_p \), shall be no greater than the permissible plastic shear deformation provided in Table G1.2. The yield shear deformation, \( \gamma_y \), 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.
### TABLE G1.2
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Bare steel deck diaphragms

<table>
<thead>
<tr>
<th>Component or Action</th>
<th>Modeling Parameters</th>
<th>Permissible Deformations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plastic Shear Deformation, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>e</td>
</tr>
<tr>
<td>Shear strength controlled by</td>
<td></td>
<td></td>
</tr>
<tr>
<td>connectors:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>support: PAF; side-lap: screw</td>
<td>2.8(\gamma_y)</td>
<td>4.0(\gamma_y)</td>
</tr>
<tr>
<td>support: weld; side-lap: screw</td>
<td>2.8(\gamma_y)</td>
<td>4.0(\gamma_y)</td>
</tr>
<tr>
<td>support: weld; side-lap: button punch</td>
<td>1.7(\gamma_y)</td>
<td>3.1(\gamma_y)</td>
</tr>
<tr>
<td>support: weld; side-lap: weld</td>
<td>2.3(\gamma_y)</td>
<td>3.6(\gamma_y)</td>
</tr>
<tr>
<td>Shear strength controlled by panel:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>buckling</td>
<td>3.6(\gamma_y)</td>
<td>5.6(\gamma_y)</td>
</tr>
</tbody>
</table>

\(^a\) For panels with spans between supports with fasteners greater than 60 in. (1500 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).

**Note:**
- 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

**Values are for shear walls with stiffeners to prevent shear buckling.**
**Structural connectors generally control residual strength. Value based on arc spot weld; for an arc seam weld, \(c = 0.15\).**

### 4b. Force-Controlled Actions

#### 1. Linear Analysis Procedures

When the shear strength of a bare steel deck diaphragm is considered force-controlled, the shear strength shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound shear strength, \(Q_{CL}\), determined per 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, \(\gamma\), of the diaphragm shall not exceed \(\gamma_y\) 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.

---

*2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*

Draft Ballot 3 Dated September 30, 2019

**AMERICAN INSTITUTE OF STEEL CONSTRUCTION**
5. Retrofit Measures

Seismic retrofit measures for bare steel deck diaphragms shall satisfy the requirements of Section B3 and ASCE/SEI 41.

G2. STEEL DECK DIAPHRAGMS WITH REINFORCED CONCRETE STRUCTURAL TOPPING

1. General

Steel deck diaphragms with reinforced concrete structural topping, consisting of either composite or a noncomposite construction, are permitted to resist diaphragm loads. The concrete fill shall be either normal or lightweight structural concrete, with reinforcing composed of welded wire reinforcing or reinforcing bars. If the reinforcing is only for temperature and shrinkage the provisions of Section G2 may be applied in lieu of this section. 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 frame components to which the topped steel deck diaphragms are attached shall be considered the chord and collector elements.

The criteria of the 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 flute 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 Action

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 flute 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 available nominal strengths specified in Specification Chapter I for steel headed stud anchors, with expected properties as provided in Chapter A.
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, $Q_{LB}$, 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 flue 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 available-nominal strengths specified in Specification Chapter I for steel headed stud anchors, with lower-bound properties as provided in Chapter A.

### 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 available 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 strength shall be evaluated using ASCE/SEI 41, Equation 7-36 with the expected component strength, $Q_{CE}$, determined from Section G2.3a and $m$ 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 Table 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 experimentally or from analysis.
When the shear strength of a steel deck diaphragm with reinforced concrete structural topping is considered deformation-controlled, the total shear deformation, $\gamma$, 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 strength shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound shear strength, $Q_{CL}$, 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, $\gamma$, of the diaphragm shall not exceed point B as defined in the generalized force-deformation curve of Figure G1.1 with initial stiffness defined in Section G2.2 and strength defined in 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 Section B3 and ASCE/SEI 41.

G3. STEEL DECK DIAPHRAGMS WITH UNREINFORCED STRUCTURAL CONCRETE TOPPING OR NONSTRUCTURAL TOPPING LIGHTWEIGHT INSULATING CONCRETE

1. General

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. Steel deck diaphragms with topping consisting of either structural concrete without reinforcing bars (temperature and shrinkage steel and headed stud anchors are allowed, as is plain concrete); or lightweight insulating concrete as defined in ANSI/AISI S310, are permitted to resist diaphragm loads. The provisions of this section apply where the reinforcing qualifies as temperature and shrinkage reinforcement in accordance with either Section 2.4.B.15.a.1 of ANSI/SDI C-2017 for composite steel deck-slabs or with Section 2.4.B.2 of ANSI/SDI NC-2017 for non-composite steel deck; or where headed stud anchors are used; or to plain 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 are 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.

2020 Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings
Draft Ballot 3 Dated September 30, 2019
AMERICAN INSTITUTE OF STEEL CONSTRUCTION
2. Stiffness

Steel deck diaphragms with unreinforced structural concrete topping or nonstructural topping 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 nonstructural topping lightweight insulating concrete shall be determined in accordance with this section.

3a. Deformation-Controlled Action

The expected component strength, $Q_{CE}$, for steel deck diaphragms with unreinforced structural concrete topping or nonstructural topping 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 nonstructural topping 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 nonstructural topping is considered deformation-controlled, the strength shall be evaluated using ASCE/SEI 41, Equation 7-36, with the expected component strength, $Q_{CE}$, determined from Section G3.3a and $m$ taken from Table G3.1.
Table G3.1
Component Capacity Modification Factor, $m$, for Linear Analysis Procedures—Steel Deck Diaphragm with Unreinforced Structural Concrete Topping or Nonstructural Topping Lightweight Insulating Concrete

<table>
<thead>
<tr>
<th>Component or Action</th>
<th>Primary Component</th>
<th>Secondary Component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IO</td>
<td>LS</td>
</tr>
<tr>
<td>Shear strength of deck with lightweight insulating concrete nonstructural topping</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- deck welded to support (arc spot or arc seam)</td>
<td>1.5</td>
<td>4.0</td>
</tr>
<tr>
<td>- headed shear studs welded through deck to support</td>
<td>1.5</td>
<td>3.0</td>
</tr>
</tbody>
</table>

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

2. Nonlinear Static and Dynamic Procedures

The generalized force-deformation curve shown in Figure C1.1, with the parameters $d$, $e_2$, and $c$ as defined in Table G3.2 shall be used for steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete nonstructural topping, or these relationships may be derived experimentally or from analysis.

When the shear strength of steel deck diaphragms with unreinforced structural concrete topping or lightweight insulating concrete nonstructural topping is considered deformation-controlled, the total shear deformation, $\gamma$, shall be no greater than the permissible shear deformations provided in Table G3.2. The initial shear deformation, $\gamma_i$, of a steel deck diaphragm with unreinforced structural concrete topping or lightweight insulating concrete nonstructural topping shall be calculated as the expected strength, $Q_{CE}$, divided by the initial stiffness as determined in Section G3.2.
### TABLE G3.2
Modeling Parameters and Permissible Deformations for Nonlinear Analysis Procedures—Steel Deck Diaphragm with Unreinforced Structural Concrete Topping or Nonstructural Topping Lightweight Insulating Concrete

<table>
<thead>
<tr>
<th>Component or Action</th>
<th>Modeling Parameters</th>
<th>Permissible Deformations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear Deformation, rad</td>
<td>Residual Strength Ratio</td>
</tr>
<tr>
<td></td>
<td>$d$</td>
<td>$e$</td>
</tr>
<tr>
<td>Shear strength of deck with lightweight insulating concrete nonstructural topping</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- deck welded to support (arc spot or arc seam)</td>
<td>$8.0\gamma_i$</td>
<td>$10.0\gamma_i$</td>
</tr>
<tr>
<td>- headed shear studs welded through deck to support</td>
<td>$8.0\gamma_i$</td>
<td>$10.0\gamma_i$</td>
</tr>
</tbody>
</table>

*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*

2667 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 nonstructural topping is considered force-controlled, the shear strength shall be evaluated using ASCE/SEI 41, Equation 7-37, with the lower-bound shear strength, $Q_{CL}$, determined per 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 nonstructural topping is considered force-controlled, the total shear deformation, $\gamma$, of the diaphragm shall not exceed $\gamma_i$ 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 nonstructural topping.
lightweight insulating concrete nonstructural topping shall satisfy the requirements of Section B3 and ASCE/SEI 41.

G4. HORIZONTAL STEEL BRACING (STEEL TRUSS DIAPHRAGMS)

1. General

Horizontal steel bracing (steel truss diaphragms) is permitted to act as a diaphragm independently or in conjunction with bare steel deck roofs. 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 braced structural steel 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 braced steel 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 available 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 design 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 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 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 available 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 Section B3 and ASCE/SEI 41.

6. **CHORD AND COLLECTOR ELEMENTS**

1. **General**

Structural steel framing that supports the diaphragm and transfers lateral loads is referred to as diaphragm chord or collector components. 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 available strength of structural steel chords and collectors shall be as specified in Section C3 for FR beam-to-column connections in Section C5.3, members subjected to combined axial force and flexure, and using the appropriate expected or lower-bound properties as provided in Chapter A. Connections between chords and collectors shall be considered force-controlled. The available 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 FR 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, $Q_{UD}$, the deformation-controlled action caused by gravity loads and earthquake forces determined in accordance with ASCE/SEI 41, Section 7.5.2.1.2, 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 bending in accordance with Chapter D.

5. Retrofit Measures

Seismic retrofit measures for chord and collector elements shall satisfy the requirements of 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 conform to 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 bending moments on the pile shall be as specified for a structural steel column in Section D2.4b for linear methods and in Section D2.4c for nonlinear methods, 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 Section B3 and ASCE/SEI 41.
CHAPTER I

CAST AND WROUGHT IRON

I. 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. Existing cast iron components of structural framing are permitted to be assessed and designed to resist seismic forces or deformations as primary structural steel components.

I2. STIFFNESS OF CAST AND WROUGHT IRON

The stiffness of cast and wrought iron components shall be calculated using elastic section properties and a modulus of elasticity, $E$, of 1520,000 ksi (100 130 000 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 OF CAST AND WROUGHT IRON

Component strengths shall be determined in accordance with Section B2 and the requirements of this section.

1. Cast Iron

The use of cast iron to resist tensile stresses is not permitted.

The lower-bound compressive strength, $Q_{CL} = P_{CL}$, of a cast iron column shall be determined from Equation I3-1.

$$P_{CL} = A_g F_{cr}$$  \hspace{1cm} (I3-1)

where

- $A_g$ = gross area of the cross section, in.$^2$ (mm$^2$)
- $F_{cr}$ = critical stress, ksi (MPa)

The critical stress, $F_{cr}$, is determined as follows:

For cast iron

(a) When $\frac{L}{r} \leq 108$

$$F_{cr} = 17 \text{ ksi}$$  \hspace{1cm} (I3-2)

(b) When $\frac{L}{r} > 108$

$$F_{cr} = 120,117 \text{ MPa}$$  \hspace{1cm} (I3-2M)
2. **Wrought Iron**

Lower-bound strength of a wrought iron component is permitted to be determined using 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 and lower-bound tensile strength shall be taken from Table A5.2, unless determined by testing in accordance with Section A5, and the modulus of elasticity shall be taken as 25,000 ksi. (2) For wrought iron, lower-bound material properties for wrought iron shall be based on Table A5.2, where the lower-bound yield stress, \( F_y_{LB} \), is taken as \( F_{cr} \).

### I4. PERMISSIBLE PERFORMANCE PARAMETERS FOR CAST AND WROUGHT IRON

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 develop 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 the provisions of ASCE/SEI 41.