

ارائه به روزترین منابع، کتابها و جزوات مهندسی عمران
به زبان فارسی و انگلیسی به صورت کاملاً رایگان

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ANSI/AISC 358-22
An American National Standard

Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

.....

August 18, 2022

Supersedes *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, dated May 12, 2016, including Supplements No. 1 (2018) and No. 2 (2020), and all previous versions

Approved by the Connection Prequalification Review Panel



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Stronger.
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PREFACE

(This Preface is not part of ANSI/AISC 358-22, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, but is included for informational purposes only.)

ANSI/AISC 358-22, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, was developed by the AISC Connection Prequalification Review Panel (CPRP) using a consensus process in concert with the *Specification for Structural Steel Buildings* (ANSI/AISC 360-22) and *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-22). ANSI/AISC 358-22 is incorporated by reference into the *Seismic Provisions*.

This edition includes a new connection in Chapter 15, DuraFuse Frames Moment Connection. Chapters 9 (Cast Bolted Bracket) and 14 (Slotted Web) have been updated to reflect that these connections are no longer proprietary. Chapter 5 (Reduced Beam Section) has been updated to expand the connection prequalification allowable beam size and beam flange thickness, as well as the allowable column depth. Chapter 6 (Stiffened and Unstiffened End Plates) has been updated to add guidance on the location of supplemental lateral bracing, and the bolt bearing/tearout failure equations have been updated to use a summation for all bolt locations. All connections were updated (as applicable) to reflect the following:

- A consistent approach to the gap between the column flange and the concrete structural slab.
- Clarification on shapes allowed for columns.
- Updated bolt-hole fabrication requirements.
- Net-section beam flange tensile rupture check in applicable chapters.
- Equations for determination of bolt hole diameter updated to reflect current AISC *Specification* provisions.

A nonmandatory Commentary has been prepared to provide background for the provisions of the standard, and the user is encouraged to consult it. Additionally, nonmandatory User Notes are interspersed throughout the standard to provide concise and practical guidance in the application of the provisions.

The reader is cautioned that professional judgment must be exercised when data or recommendations in this standard are applied, as described more fully in the disclaimer notice preceding the Preface.

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SYMBOLS

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

Symbol	Definition	Section
A_b	Nominal gross area of bolt, in. ² (mm ²)	6.7.1
A_b	Nominal unthreaded body area of bolt, in. ² (mm ²)	7.5
A_c	Contact areas between continuity plate and column flanges that have attached beam flanges, in. ² (mm ²)	6.4
A_c	Area of concrete in column, in. ² (mm ²)	10.7
A_n	Net area of end plate, in. ² (mm ²)	6.7.1
A_{pz}	Panel zone area, in. ² (mm ²)	10.7
A_s	Area of steel in column, in. ² (mm ²)	10.7
A_{tb}	Gross area of a tension bolt measured through its shank, in. ² (mm ²)	13.5
A_{yb}	Gross area of a shear bolt measured through its shank, in. ² (mm ²)	13.5
A_{y-link}	Yield area of reduced Yield-Link section, in. ² (mm ²)	12.9
A'_{y-link}	Estimated required Yield-Link yield area, in. ² (mm ²)	12.9
A_{\perp}	Perpendicular amplified seismic drag or chord forces transferred through the SidePlate connection, resulting from applicable building code, kips (N)	11.7
A_{\parallel}	In-plane factored lateral drag or chord axial forces transferred along the frame beam through the SidePlate connection, resulting from load case 1.0 E_Q per applicable building code, kips (N)	11.7
C_{pr}	Factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions	2.4.3
C_t	Factor used in Equation 6.7-17	6.7.2
E	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	13.5
F_{EXX}	Filler metal classification strength, ksi (MPa)	9.7
F_f	Maximum force in the T-stub and beam flange, kip (N)	13.5
F_{fu}	Factored beam flange force, kips (N)	6.7.1
F_{nt}	Nominal tensile stress of bolt from the AISC <i>Specification</i> , ksi (MPa)	6.7.1
F_{nv}	Nominal shear stress of bolt from the AISC <i>Specification</i> , ksi (MPa)	6.7.1
F_{pr}	Probable maximum force in the T-stub and beam flange, kips (N)	13.5
F_{pr}	Force in the flange plate due to M_f , kips (N)	7.5
F_{su}	Required stiffener strength, kips (N)	6.7.2
F_{su}	Required strength of continuity or stiffener plate, kips (N)	12.9
F_u	Specified minimum tensile strength of yielding element, ksi (MPa)	2.4.3
F_{ub}	Specified minimum tensile strength of beam, ksi (MPa)	7.5

Symbol	Definition	Section
F_{uf}	Specified minimum tensile strength of flange material, ksi (MPa)	9.7
F_{up}	Specified minimum tensile strength of end-plate material, ksi (MPa)	6.7.1
F_{up}	Specified minimum tensile strength of plate material, ksi (MPa)	7.5
F_{ut}	Specified minimum tensile stress of T-stub, ksi (MPa)	13.5
F_{u-link}	Specified minimum tensile strength of Yield-Link stem material, ksi (MPa)	12.9
F_w	Nominal weld design strength per the AISC <i>Specification</i> , ksi (MPa)	9.7
F_y	Specified minimum yield stress of the yielding element, ksi (MPa)	2.4.3
F_{yb}	Specified minimum yield stress of beam material, ksi (MPa)	6.7.1
F_{yb}	Specified minimum yield stress of the beam, ksi (MPa)	11.4(3)
F_{yc}	Specified minimum yield stress of column material, ksi (MPa)	6.7.2
F_{yc}	Specified minimum yield stress of the column, ksi (MPa)	11.4(3)
F_{ye}	Actual yield strength of the column at the connection. In the absence of F_{ye} , expected yield strength of the column, $R_y F_{yc}$, may be used, ksi (MPa)	11.4(3)
F_{ye}	Expected yield strength of steel beam, ksi (MPa)	14.7
F_{yf}	Specified minimum yield stress of flange material, ksi (MPa)	9.7
F_{yp}	Specified minimum yield stress of end-plate material, ksi (MPa)	6.7.1
F_{yp}	Specified minimum yield stress of plate material, ksi (MPa)	15.6
F_{ys}	Specified minimum yield stress of stiffener material, ksi (MPa)	6.7.1
F_{yt}	Specified minimum yield stress of the T-stub, ksi (MPa)	13.5
F_{y_BRP}	Specified minimum yield stress of buckling restraint plate material, ksi (MPa)	12.9
F_{y-link}	Specified minimum yield stress of Yield-Link stem material, ksi (MPa)	12.9
H	Story height, in. (mm)	11.4(3)
H_h	Distance along column height from $1/4$ of column depth above the top edge of lower-story side plates to $1/4$ of column depth below bottom edge of upper-story side plates, in. (mm)	11.4(3)
H_l	Height of story below node, in. (mm)	10.7
H_u	Height of story above node, in. (mm)	10.7
I_{beam}	Moment of inertia of the beam in plane of bending, in. ⁴ (mm ⁴)	Figure 11.16
I_{beam}	Strong-axis moment of inertia of the beam, in. ⁴ (mm ⁴)	13.5
I_{ft}	Moment of inertia of the T-flange per pair of tension bolts, in. ⁴ (mm ⁴)	13.5
I_{total}	Approximation of moment of inertia due to beam hinge location and side plate stiffness, in. ⁴ (mm ⁴)	Figure 11.16
K_1	Elastic axial stiffness contribution due to bending stiffness in Yield-Link flange, kip/in. (N/mm)	12.9
K_2	Elastic axial stiffness contribution due to nonyielding section of Yield-Link, kip/in. (N/mm)	12.9
K_3	Elastic axial stiffness contribution due to yielding section of Yield-Link, kip/in. (N/mm)	12.9
K_{comp}	Initial stiffness of a T-stub in compression, kip/in. (N/mm)	13.5

Symbol	Definition	Section
K_{eff}	Effective elastic axial stiffness of Yield-Link, kip/in. (N/mm)	12.9
K_{flange}	Initial stiffness of a T-flange, kip/in. (N/mm)	13.5
K_i	Initial stiffness of the connection, kip-in./rad (N-mm/rad)	13.5
K_{slip}	Initial stiffness of the slip mechanism between a T-stem and beam flange, kip/in. (N/mm)	13.5
K_{stem}	Initial stiffness of a T-stem, kip/in. (N/mm)	13.5
K_{ten}	Initial stiffness of a T-stub in tension, kip/in. (N/mm)	13.5
L	Distance between column centerlines, in. (mm)	11.3(5)
L_{bb}	Bolted bracket length, in. (mm)	Table 9.1
$L_{bm-side}$	Length of nonreduced Yield-Link at beam side, in. (mm)	Figure 12.2
L_c	Clear distance, in direction of force, between edge of the hole and edge of the adjacent hole or edge of material, in. (mm)	6.7.1
L_{cant}	Lever arm from start of reduced region to edge of spacer plate bolt hole, plus plate stretch from 0.05 rad of rotation, in. (mm)	Figure 12.4(a)
$L_{col-side}$	Length of nonreduced Yield-Link at column side, in. (mm)	Figure 12.2
L_{ehb}	Horizontal end distance of the beam measured from the end of the beam to the centerline of the first row of shear bolts or to the centerline of the web bolts, in. (mm)	13.5
L_{e1}	Vertical edge distance for bolts in Yield-Link flange to column flange connection, in. (mm)	Figure 12.2
L_{e2}	Horizontal edge distance for bolts in Yield-Link flange to column flange connection, in. (mm)	Figure 12.2
L_h	Distance between plastic hinge locations, in. (mm)	2.4.4
L_o	Theoretical length of the connected beam measured between the working points of the adjacent columns, in. (mm)	13.5
$L_{slot-horz}$	Shear plate horizontal bolt slot length, in. (mm)	12.9
$L_{slot-vert}$	Shear plate vertical bolt slot length, in. (mm)	12.9
L_{sp}	Length of shear connection, in. (mm)	13.5
L_{st}	Length of end-plate stiffener, in. (mm)	6.6.4
L_{vb}	Length of the shear bolt pattern in the T-stems and beam flanges, in. (mm)	13.5
L_{y-link}	Length of reduced Yield-Link section, in. (mm)	Figure 12.2
M_{bolts}	Moment at collar bolts, kip-in. (N-mm)	10.7
M_{cant}	Factored gravity moments from cantilever beams that are not in the plane of the moment frame but are connected to the exterior face of the side plates, resulting from code-applicable load combinations, kip-in. (N-mm)	11.7
M_{group}	Maximum probable demand at any connection element, kip-in. (N-mm)	11.7
M_{cf}	Column flange flexural strength, kip-in. (N-mm)	6.7.2
M_f	Probable maximum moment at face of column, kip-in. (N-mm)	2.4.5
M_f	Moment developed at face of column, kip-in. (N-mm)	13.5

Symbol	Definition	Section
M_{np}	Moment without prying action in bolts, kip-in. (N-mm)	Table 6.2
M_{pe}	Plastic moment of beam based on expected yield stress, kip-in. (N-mm)	5.7
M_{pr}	Probable maximum moment at plastic hinge, kip-in. (N-mm)	2.4.3
M_{pr}	Probable maximum moment capacity of Yield-Link pair, kip-in. (N-mm)	12.9
M_{iw}	Additional moment due to shear amplification from center of reduced beam section to centerline of column, kip-in. (N-mm).	5.4(2)
M_{iw}	Additional moment due to shear amplification from the plastic hinge, kip-in. (N-mm)	14.4
M_{pb}^e	Expected flexural strength of the beam, kip-in. (N-mm)	11.4(3)
M_{pc}^e	Nominal flexural strength of the column, kip-in. (N-mm)	11.4(3)
M_{pcl}^e	Plastic moment nominal strength of the column below the node, about the axis under consideration, considering simultaneous axial loading and loading about the transverse axis, kip-in. (N-mm).	10.7
M_{pcu}^e	Plastic moment nominal strength of the column above the node, about the axis under consideration, considering simultaneous axial loading and loading about the transverse axis, kip-in. (N-mm)	10.7
M_{u-sp}	Moment in shear plate at the column face, kip-in. (N-mm)	12.9
M_v	Additional moment due to the beam shear acting on a lever arm extending from the assumed point of plastic hinging to the centerline of the column, kip-in. (N-mm)	10.7
M_{weld}	Moment resisted by the shear plate, kip-in. (N-mm)	14.7
$M_{ye-link}$	Expected yield moment of Yield-Link pair, kip-in. (N-mm).	12.9
N_{design}	Number of contact points between reduced region of link stem and buckling restraint plate or beam flange (rounded to the nearest integer).	12.9
P_{r-weld}	Required strength of Yield-Link stem to Yield-Link flange weld, kips (N)	12.9
P_{r-link}	Probable maximum tensile strength of Yield-Link, kips (N)	12.9
P_{slip}	Expected slip load of the shear bolts between the beam flange and T-stem, kips (N)	13.5
P_t	Minimum specified tensile strength of bolt, kips (N)	Table 6.2
P_u	Axial load acting on the column at the section under consideration in accordance with the applicable load combination specified by the building code, but not considering amplified seismic load, kips (N)	10.7
P_{uc}	Column axial compressive load calculated using LRFD load combinations, kips (N)	11.4(3)
P_{u-sp}	Required axial strength of beam web-to-column flange connection, kips (N)	12.9
$P_{ye-link}$	Expected yield strength of the Yield-Link, kips (N)	12.9
P'_{y-link}	Estimated required Yield-Link yield force, kips (N).	12.9
Q	Total vertical thrust force on beam flange, kips (N)	12.9
R_n	Required force for continuity plate design, kips (N)	6.7.1
R_n	Nominal strength	7.5

Symbol	Definition	Section
R_n^{PZ}	Nominal panel zone shear strength, kips (N)	10.7
R_{pt}	Minimum bolt pretension, kips (N)	10.7
R_t	Ratio of expected tensile strength to specified minimum tensile strength for flange material	7.5
R_u^{PZ}	Required panel zone shear strength, kips (N)	10.7
R_y	Ratio of expected yield stress to specified minimum yield stress, F_y ,	2.4.3
R_{y_BRP}	Ratio of the expected yield stress to specified minimum yield stress, F_{y_BRP} , taken as 1.1 for buckling restraint plate material	12.9
S_1	Distance from face of column to nearest row of bolts, in. (mm)	7.5
S_1	Distance from the face of the column to the first row of shear bolts, in. (mm)	13.5
S_h	Distance from face of column to plastic hinge, in. (mm)	2.3.2a
T	Tension force per bolt, kips/bolt (N/bolt)	13.5
T	Beam web height as given in the AISC <i>Manual</i> , in. (mm)	14.7
T_{ux}	Vertical thrust force transferred by one restraint bolt, kips (N)	12.9
T_1	Nominal tension strength per bolt of the T-flange corresponding to a plastic mechanism in the T-flange, kip/bolt (N/bolt)	13.5
T_2	Nominal tension strength per bolt of the T-flange corresponding to a mixed-mode failure of the T-flange, kip/bolt (N/bolt)	13.5
T_3	Nominal tension strength per bolt of the T-flange corresponding to bolt fracture without T-flange yielding, kip/bolt (N/bolt)	13.5
T_{req}	Required T-stub force per tension bolt, kip/bolt (N/bolt)	13.5
V_{beam}	Shear at beam plastic hinge, kips (N)	14.4
V_{bolts}	Probable maximum shear at collar bolts, kips (N)	10.7
V_{cant}	Factored gravity shear forces from cantilever beams that are not in the plane of the moment frame but are connected to the exterior face of the side plates, resulting from code-applicable load combinations, kips (N)	11.7
V_{cf}	Probable maximum shear at face of collar flange, kips (N)	10.7
V_{col}	Column shear, kips (N)	10.7
V_f	Probable maximum shear at face of column, kips (N)	10.7
$V_{gravity}$	Beam shear force resulting from $1.2D + f_1L + 0.15S$, kips (N)	2.4.4
V_h	Beam shear force at plastic hinge location, kips (N)	2.4.4
V_{RBS}	Larger of the two values of shear force at center of reduced beam section at each end of beam, kips (N)	5.7
V_u	Required shear strength of beam, kips (N)	2.5
V_{u-bolt}	Maximum shear plate bolt shear, kips (N)	12.9
V_{ux}	Out-of-plane shear thrust force exerted on each spacer plate	12.9
V_{uy}	In-plane shear thrust force exerted on each spacer plate in the strong axis direction.	12.9
V_{weld}	Shear resisted by the shear plate, kips (N)	14.7

Symbol	Definition	Section
V_1, V_2	Factored gravity shear forces from gravity beams that are not in the plane of the moment frame but are connected to the exterior surfaces of the side plate, resulting from the load combination of $1.2D + f_1L + 0.15S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)	11.7
W_T	Width of the T-stub measured parallel to the column flange width, in. (mm)	13.5
W_{Whit}	Whitmore width of the stem of the T-stub, in. (mm)	13.5
Y_C	Yield-line parameter used to determine column-flange strength, in. (mm) . .	13.5
Y_c	Column flange yield line mechanism parameter, in. (mm)	6.7, 2
Y_m	Simplified column flange yield-line mechanism parameter	9.7
Y_p	End-plate yield line mechanism parameter, in. (mm)	6.7
Z_b	Nominal plastic section modulus of beam, in. ³ (mm ³)	11.4
Z_c	Plastic section modulus of the column about either axis, in. ³ (mm ³)	10.7
Z_e	Effective plastic modulus of section (or connection) at location of a plastic hinge, in. ³ (mm ³)	2.4, 3
Z_{ec}	Equivalent plastic section modulus of column at a distance of $\frac{1}{4}$ the column depth from top and bottom edge of side plates, projected to beam centerline, in. ³ (mm ³)	11.4(3)
Z_{RBS}	Plastic section modulus at the center of reduced beam section, in. ³ (mm ³)	5.7
Z_{web}	Plastic section modulus of the beam web, in. ³ (mm ³)	14.7
Z_{wp}	Plastic section modulus of washer plate, in. ³ (mm ³)	9.7
Z_x	Plastic section modulus about x -axis, in. ³ (mm ³)	5.7
Z_x	Plastic section modulus about the x -axis of the gross section of the beam at the location of the plastic hinge, in. ³ (mm ³)	13.5
Z_{xb}	Plastic modulus of beam about x -axis, in. ³ (mm ³)	11.7
Z_{xc}	Plastic modulus of column about x -axis, in. ³ (mm ³)	11.7
$Z_{x,net}$	Plastic section modulus of the net section of the beam at the location of the plastic hinge, in. ³ (mm ³)	7.5
a	Horizontal distance from face of column flange to start of a reduced beam section cut, in. (mm)	5.7
a	Distance from outside face of the collar to reduced beam section cut, in. (mm)	10.7
a	Horizontal distance from centerline of bolt holes in shear plate to face of column, in. (mm)	12.4(2)
a	Distance between bolt line and outside edge of T-flange, in. (mm)	13.5
a'	Distance between inside edge of bolt line and outside edge of T-flange, in. (mm)	13.5
a_c	Horizontal distance from inside tension bolts and edge of column flange, in. (mm)	13.5

Symbol	Definition	Section
b	Width of compression element as defined in the AISC <i>Specification</i> , in. (mm)	2.3.2b
b	Length of reduced beam section cut, in. (mm)	5.7
b	Vertical distance from centerline of bolt holes in Yield-Link flange to face of Yield-Link stem, in. (mm).	12.9
b	Distance between effective T-stem and bolt line in the T-flange, in. (mm) . .	13.5
b'	Distance from the bolt centerline to the beam centerline, in. (mm)	12.9
b'	Distance between effective T-stem and inside edge of bolt line in T-flange, in. (mm).	13.5
b_{bb}	Bolted bracket width, in. (mm)	Table 9.1
b_{bf}	Width of beam flange, in. (mm)	5.7
$b_{bm-side}$	Width of nonreduced Yield-Link at beam side, in. (mm)	Figure 12.2
b_c	Horizontal distance from column web to inside tension bolts, in. (mm)	13.5
b_{cf}	Width of column flange, in. (mm).	9.7
$b_{col-side}$	Width of nonreduced Yield-Link at column side, in. (mm)	Figure 12.2
b_{fc}	Flange width of the column, in. (mm)	13.5
b_f	Flange width of the beam, in. (mm)	14.7
b_{flange}	Width of Yield-Link flange at column side, in. (mm).	Figure 12.2
b_{fp}	Width of flange plate, in. (mm).	7.5
b_{ft}	Flange width of the T-stub, in. (mm)	13.5
b_n	Net width of buckling restraint plate, in. (mm).	12.9
b_p	Width of end plate, in. (mm)	Table 6.1
b_{yield}	Width of reduced Yield-Link section, in. (mm)	Figure 12.2
c	Depth of cut at center of reduced beam section, in. (mm)	5.7
d	Depth of beam, in. (mm).	5.3.1
d_b	Diameter of column flange bolts, in. (mm).	9.7
d_{b-brp}	Diameter of bolt connecting buckling restraint plate to beam flange, in. (mm).	Figure 12.3
$d_{b-flange}$	Diameter of bolt connecting Yield-Link flange to column flange, in. (mm).	12.9
d_{b-sp}	Diameter of bolts in shear plate, in. (mm).	12.9
d_{b-stem}	Diameter of bolts connecting Yield-Link stem to beam flange, in. (mm). . . .	12.9
$d_{b,req}$	Required bolt diameter, in. (mm)	6.7.1
d_c	Depth of column, in. (mm)	6.7
d_{c1}, d_{c2}	Depth of column on each side of a bay in a moment frame, in. (mm) . .	11.3.1(5)
d_e	Column bolt edge distance, in. (mm)	Table 9.2
d_{eff}	Effective beam depth, calculated as the centroidal distance between bolt groups in the upper and lower brackets, in. (mm)	9.7
d_{leg}^{CC}	Effective depth of collar corner assembly leg, in. (mm).	10.7
d_{tb}	Diameter of the tension bolts between the T-flange and the column flange, in. (mm)	13.5

Symbol	Definition	Section
d_{tht}	Diameter or width of the holes in the T-flange for the tension bolts, in. (mm)	13.5
d_{vb}	Diameter of the shear bolts between the T-stem and the beam flange, in. (mm)	13.5
d_{vht}	Diameter of the holes in the T-stem for the shear bolts, in. (mm)	13.5
d_z	Depth of web of deeper beam at connection, in. (mm).	15.6
e_x	Eccentricity of the shear plate weld, in. (mm)	14.7
f'_c	Specified compressive strength of concrete fill, ksi (MPa)	10.7
f_1	Load factor determined by the applicable building code for live loads but not less than 0.5	5.7
g	Horizontal distance (gage) between fastener lines, in. (mm)	Table 6.1
g	Bolt gage of column, in. (mm)	Table 9.1
g	Gap increase due to transverse shortening of the Yield-Link thickness, in. (mm).	12.9
g_{flange}	Vertical distance between rows of bolts in connection of Yield-Link flange to column flange, in. (mm)	Figure 12.2
g_{ic}	Gage of interior tension bolts in the column flange, in. (mm)	13.5
g_{stem}	Horizontal distance between rows of bolts in connection of Yield-Link stem to beam flange, in. (mm).	Figure 12.2
g_{tb}	Gage of the tension bolts in the T-stub, in. (mm)	13.5
g_{vb}	Gage of the shear bolts in the T-stub, in. (mm)	13.5
h	Height of shear plate, in. (mm)	14.7
h_1	Distance from the centerline of a compression flange to the tension-side inner bolt rows in four-bolt extended and four-bolt stiffened extended end-plate moment connections, in. (mm)	Table 6.2
h_{bb}	Bolted bracket height, in. (mm)	Table 9.1
h_{flange}	Height of Yield-Link flange, in. (mm)	Figure 12.2
h_i	Distance from centerline of beam compression flange to the centerline of the i th tension bolt row, in. (mm)	6.7.1
h_o	Distance from centerline of compression flange to the tension-side outer bolt row in four-bolt extended and four-bolt stiffened extended end-plate moment connections, in. (mm).	Table 6.2
h_p	Height of plate, in. (mm).	8.6(2)
h_{st}	Height of stiffener, in. (mm)	6.6.4
k_1	Distance from web centerline to flange toe of fillet, in. (mm)	3.6
k_c	Distance from outer face of a column flange to web toe of fillet (design value) or fillet weld, in. (mm)	6.7.2
k_{det}	Largest value of k_1 used in production, in. (mm)	3.6
l	Bracket overlap distance, in. (mm)	9.7
l_b	Thickness of beam flange plus 2 times the reinforcing fillet weld size plus 2 times the end-plate thickness, in. (mm)	6.7.2

Symbol	Definition	Section
l_b	Half the clear span length of the beam, in. (mm)	14.7
l_o	Effective buckling wave length	12.9
l_p	Width of shear plate, in. (mm)	14.4
l_s	Beam slot length, in. (mm)	14.7
l_w	Length of available fillet weld, in. (mm).	9.7
l_w^{CC}	Total length of available fillet weld at collar corner assembly, in. (mm)	10.7
l_w^{CWX}	Total length of available fillet weld at collar web extension, in. (mm).	10.7
n	Number of bolts.	7.5
n_b	Number of bolts at compression flange	6.7.1
n_{bb}	Number of beam bolts	Table 9.3
n_{bolt}	Number of bolts in Yield-Link stem-to-beam flange connection	12.9
$n_{bolt-sp}$	Total number of bolts in shear plate	12.9
$n_{bolt-sp-horz}$	Total number of horizontal bolts resisting axial force in the shear plate in line with the central bolt	12.9
$n_{bolt-sp-vert}$	Total number of vertical bolts resisting shear force in the shear plate	12.9
n_{BRP_bolts}	Total number of buckling restraint plate bolts.	12.9
n_{cb}	Number of column bolts	Table 9.1
n_{cf}	Number of collar bolts per collar flange	10.7
n_{rows}	Number of rows of bolts in Yield-Link stem.	12.9
n_{tb}	Number of tension bolts connecting the T-flange to the column flange	13.5
n_{yb}	Number of shear bolts connecting the T-stem to the beam flange	13.5
p	Perpendicular tributary length per bolt, in. (mm)	9.7
p	Minimum of $b_{flange}/2$ or s_{flange} , in. (mm).	12.9
p	Width of the T-stub tributary to a pair of tension bolts, in./bolt (mm/bolt) . .	13.5
p_b	Vertical distance between inner and outer row of bolts in eight-bolt stiffened extended end-plate moment connection, in. (mm)	Table 6.1
p_b	Column bolt pitch, in. (mm)	Table 9.2
p_e	Effective (tributary) length per bolt from the yield line pattern, in. (mm) . .	12.9
p_{fi}	Vertical distance from inside of a beam tension flange to nearest inside bolt row, in. (mm)	Table 6.1
p_{fo}	Vertical distance from outside of a beam tension flange to nearest outside bolt row, in. (mm)	Table 6.1
p_s	Vertical distance from continuity plate to horizontal row of tension bolts, in. (mm)	13.5
p_{si}	Distance from inside face of continuity plate to nearest inside bolt row, in. (mm)	6.7.2
p_{so}	Distance from outside face of continuity plate to nearest outside bolt row, in. (mm)	6.7.2
r_h	Bracket horizontal radius, in. (mm)	Table 9.2
r_{nt}	Nominal tensile strength of a tension bolt, kips/bolt (N/bolt)	13.5
r_{nv}	Nominal shear strength of a shear bolt, kips/bolt (N/bolt)	13.5

Symbol	Definition	Section
r_t	Required tension force per bolt in Yield-Link flange to column flange connections, kips/bolt (kN/bolt)	12.9
r_{ut}	Required column bolt tension strength, kips (N)	9.7
r_v	Radius of bracket stiffener, in. (mm)	Table 9.2
s	Distance from centerline of most inside or most outside tension bolt row to the edge of a yield line pattern, in. (mm)	Table 6.2
s	Spacing of bolt rows in a bolted flange plate moment connection, in. (mm)	7.5
s	Vertical distance defining potential yield-line pattern in column flange, in. (mm)	13.5
s_b	Distance from center of last row of bolts to beam-side end of Yield-Link, in. (mm)	Figure 12.2
s_{bolts}	Distance from center of plastic hinge to the centroid of the collar bolts, in. (mm)	10.7
s_c	Distance from the reduced section of the Yield-Link to the center of the first row of bolts, in. (mm)	Figure 12.2
s_f	Distance from center of plastic hinge to face of column, in. (mm)	10.7
s_{flange}	Spacing between bolts for Yield-Link flange-to-column-flange connection, in. (mm)	Figure 12.2
s_h	Distance from center of plastic hinge to center of column, in. (mm)	10.7
s_{stem}	Spacing between rows of bolts for Yield-Link stem-to-beam-flange connection, in. (mm)	Figure 12.2
s_{vb}	Spacing of the shear bolts in the T-stub, in. (mm)	13.5
s_{vert}	Vertical distance from center of the top (or bottom) shear plate bolt to center of center shear plate bolt, in. (mm)	12.9
t_{bf}	Thickness of beam flange, in. (mm)	5.7
t_{bf_min}	Minimum beam flange thickness to prevent yielding and BRP bolt induced prying, in. (mm)	12.9
t_{BRP_min}	Minimum thickness of buckling restraint plate to prevent yielding during compression of the link stem, in. (mm)	12.9
t_{bw}	Thickness of beam web, in. (mm)	6.7.1
t_{cf}	Column flange thickness, in. (mm)	6.7.2
t_{cf_min}	Minimum column flange thickness for flexural yielding, in. (mm)	12.9
t_{col}	Wall thickness of HSS or built-up box column, in. (mm)	10.7
t_{collar}	Distance from face of column to outside face of collar, in. (mm)	10.7
t_{cp}	Thickness of continuity plates, in. (mm)	Figure 6.3
t_{cw}	Thickness of column web, in. (mm)	6.7.2
t_f^{CC}	Fillet weld size required to join collar corner assembly to column, in. (mm)	10.7
t_f^{CWX}	Fillet weld size required to join each side of beam web to collar web extension, in. (mm)	10.7

Symbol	Definition	Section
t_{fb}	Flange thickness of the beam, in. (mm)	13.5
t_{fc}	Flange thickness of the column, in. (mm)	13.5
t_{flange}	Thickness of Yield-Link flange, in. (mm)	Figure 12.2
t_{ft}	Flange thickness of the T-stub, in. (mm)	13.5
$t_{ft,crit}$	Flange thickness of the T-stub above which prying is negligible, in. (mm)	13.5
t_{leg}^{CC}	Effective thickness of collar corner assembly leg, in. (mm)	10.7
t_p	Thickness of end plate, in. (mm)	Table 6.1
t_p	Minimum required shear plate thickness, in. (mm)	14.7
$t_{p,req}$	Required end-plate thickness, in. (mm)	6.7.1
t_s	Thickness of stiffener, in. (mm)	6.7.1
t_s	Bracket stiffener thickness, in. (mm)	Table 9.2
t_{st}	Stem thickness of the T-stub, in. (mm)	13.5
$t_{st,eff}$	Effective stem thickness of the T-stub used for prying calculations (see Figure 13.6 and Equation 13.5-51), in. (mm)	13.5
t_{stem}	Thickness of Yield-Link stem, in. (mm)	Figure 12.2
w	Minimum size of fillet weld, in. (mm)	Table 9.2
w_u	Distributed load on beam, kip/ft (N/mm), using the load combination $1.2D + f_1L + 0.15S$	10.7
w_z	Width of panel zone between column flanges, in. (mm).	15.6
x	Distance from plastic hinge location to centroid of connection element, in. (mm)	11.7
$\Delta_{0.04}$	Axial deformation in Yield-Link at a connection rotation of 0.04 rad	12.9
$\Delta_{0.07}$	Axial deformation in Yield-Link at a connection rotation of 0.07 rad	12.9
Δ_{slip}	Expected deformation at the onset of slip, 0.0076 in. (0.19 mm)	13.5
Δ_y	Axial deformation in Yield-Link at expected yield, in. (mm)	12.9
θ_y	Connection rotation at expected yield of Yield-Link, rad	12.9
α	Adjustment factor for predicting the expected slip load of the connection.	13.5
β_a	Adjustment factor to account for shear deformation in the T-flange outside of the tension bolts.	13.5
β_b	Adjustment factor to account for shear deformation in the T-flange between the tension bolts.	13.5
δ	Factor accounting for net area of T-stub flange.	13.5
ϕ_d	Resistance factor for ductile limit states	2.4.1
ϕ_n	Resistance factor for nonductile limit states	2.4.1
μ_k	Coefficient of dry kinetic friction, taken as 0.3.	12.9
θ_y	Connection rotation at expected yield of Yield-Link, rad	12.9

GLOSSARY

This Standard uses the following terms in addition to the terms defined in the *Specification for Structural Steel Buildings* (ANSI/AISC 360-22) and the *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-22).

Air carbon arc cutting. Process of cutting steel by the heat from an electric arc applied simultaneously with an air jet.

Backing. Piece of metal or other material, placed at the weld root to facilitate placement of the root pass.

Backgouge. Process of removing by grinding or air carbon arc cutting all or a portion of the root pass of a complete-joint-penetration (CJP) groove weld, from the reverse side of a joint from which a root was originally placed.

Cascaded weld ends. Method of terminating a weld in which subsequent weld beads are stopped short of the previous bead, producing a cascade effect.

Concrete structural slab. Reinforced concrete slab or concrete fill on steel deck with a total thickness of 3 in. (75 mm) or greater and a concrete compressive strength in excess of 2,000 psi (14 MPa).

Full-length beam erection method. A method of erecting a SidePlate steel frame that employs a full-length beam assembly consisting of the beam with shop-installed cover plates (if required) and vertical shear elements (except for HSS beams) that are fillet-welded near the ends of the beam. In the field, the full-length beams are lifted up in between pre-installed side plates and are joined to the plates with fillet welds.

Horizontal shear plate (HSP). Plates that transfer a portion of the moment in the side plates to the web of a wide-flange column in a SidePlate moment connection.

Link-beam erection method. A method of erecting a SidePlate steel frame that utilizes column tree assemblies with shop-installed beam stubs, which are then connected in the field to a link beam using CJP groove welds.

Nonfusible backing. Backing material that will not fuse with the base metals during the welding process.

Plastic hinge location. Location in a column-beam assembly where inelastic energy dissipation is assumed to occur through the development of plastic flexural straining.

Probable maximum moment at the plastic hinge. Expected moment developed at a plastic hinge location along a member, considering the probable (mean) value of the material strength for the specified steel and effects of strain hardening.

Reinforcing fillet. Fillet weld applied to a groove welded T-joint to obtain a contour to reduce stress concentrations associated with joint geometry.

Root. Portion of a multi-pass weld deposited in the first pass of welding.

Thermal cutting. Group of cutting processes that severs or removes metal by localized melting, burning, or vaporizing of the workpiece.

Vertical shear elements (VSE). Structural elements that transfer shear from a wide-flange beam web to the outboard edge of the side plates in a SidePlate moment connection.

Weld tab. Piece of metal affixed to the end of a welded joint to facilitate the initiation and termination of weld passes outside the structural joint.

CHAPTER 1

GENERAL

1.1. SCOPE

This standard specifies design, detailing, fabrication, and quality criteria for connections that are prequalified in accordance with the AISC *Seismic Provisions for Structural Steel Buildings* (herein referred to as the AISC *Seismic Provisions*) for use with special moment frames (SMF) and intermediate moment frames (IMF). The connections contained in this standard are prequalified to meet the requirements in the AISC *Seismic Provisions* only when designed and constructed in accordance with the requirements of this standard. Nothing in this standard shall preclude the use of connection types contained herein outside the indicated limitations, nor the use of other connection types, when satisfactory evidence of qualification in accordance with the AISC *Seismic Provisions* is presented to the authority having jurisdiction.

User Note: Proprietary connections included in this standard are optional; the inclusion of proprietary (trademarked) products is not an endorsement. Nonproprietary options are available.

1.2. REFERENCES

The following publications form a part of this standard to the extent that they are referenced and applicable:

American Institute of Steel Construction (AISC)

ANSI/AISC 341-22 *Seismic Provisions for Structural Steel Buildings* (herein referred to as the AISC *Seismic Provisions*)

ANSI/AISC 360-22 *Specification for Structural Steel Buildings* (herein referred to as the AISC *Specification*)

AISC *Steel Construction Manual*, 15th Ed. (herein referred to as the AISC *Manual*)

American Society of Mechanical Engineers (ASME)

ASME B46.1-2019 *Surface Texture, Surface Roughness, Waviness, and Lay*

American Society for Nondestructive Testing (ASNT)

ASNT SNT-TC-1A-2016 *Personnel Qualification and Certification in Nondestructive Testing*

ASTM International (ASTM)

A36/A36M-19 *Standard Specification for Carbon Structural Steel*

A354-17e2 *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

- A370-20 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*
- A488/A488M-18e2 *Standard Practice for Steel Castings, Welding, Qualifications of Procedures and Personnel*
- A500/A500M-21a *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*
- A572/A572M-21e1 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*
- A574-21 *Standard Specification for Alloy Steel Socket-Head Cap Screws*
- A609/A609M-12(2018) *Standard Practice for Castings, Carbon, Low-Alloy, and Martensitic Stainless Steel, Ultrasonic Examination Thereof*
- A668/A668M-22 *Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use*
- A709/A709M-18 *Standard Specification for Structural Steel Bridges*
- A781/A781M-21 *Standard Specification for Castings, Steel and Alloy, Common Requirements, for General Industrial Use*
- A788/A788M-22a *Standard Specification for Steel Forgings, General Requirements*
- A802-19 *Standard Practice for Steel Castings, Surface Acceptance Standards, Visual Examination*
- A903/A903M-99(2017) *Standard Specification for Steel Castings, Surface Acceptance Standards, Magnetic Particle and Liquid Penetrant Inspection*
- A913/A913M-19 *Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)*
- A958/A958M-17 *Standard Specification for Steel Castings, Carbon and Alloy, with Tensile Requirements, Chemical Requirements Similar to Standard Wrought Grades*
- A992/A992M-20 *Standard Specification for Structural Steel Shapes*
- A1085/A1085M-15 *Standard Specification for Cold-Formed Welded Carbon Steel Hollow Structural Sections (HSS)*
- B19-20 *Standard Specification for Cartridge Brass Sheet, Strip, Plate, Bar, and Disks*
- B36/B36M-18 *Standard Specification for Brass Plate, Sheet, Strip, and Rolled Bar*
- E186-20 *Standard Reference Radiographs for Heavy Walled (2 to 4½ in. (50.8 to 114 mm)) Steel Castings*
- E446-20 *Standard Reference Radiographs for Steel Castings Up to 2 in. (50.8 mm) in Thickness*
- E709-21 *Standard Guide for Magnetic Particle Testing*
- F3125/F3125M-19e2 *Standard Specification for High Strength Structural Bolts and Assemblies, Steel and Alloy Steel, Heat Treated, Inch Dimensions 120 ksi and 150 ksi Minimum Tensile Strength, and Metric Dimensions 830 MPa and 1040 MPa Minimum Tensile Strength*
- F3148-17a *Standard Specification for High Strength Structural Bolt Assemblies, Steel and Alloy Steel, Heat Treated, 144 ksi Minimum Tensile Strength, Inch Dimensions*

American Society (AWS)

AWS C4.1:2010 *Criteria for Describing Oxygen-Cut Surfaces*

AWS D1.1/D1.1M-2020 *Structural Welding Code—Steel*

AWS D1.8/D1.8M-2016 *Structural Welding Code—Seismic Supplement*

Manufacturers Standardization Society (MSS)

MSS SP-55-2011 *Quality Standard for Steel Castings for Valves, Flanges and Fittings and Other Piping Components—Visual Method for Evaluation of Surface Irregularities*

Research Council on Structural Connections (RCSC)

Specification for Structural Joints using High-Strength Bolts, 2020 (herein referred to as the RCSC *Specification*)

1.3. GENERAL

All design, materials, and workmanship shall conform to the requirements of the AISC *Seismic Provisions* and this standard. The connections contained in this standard shall be designed according to the load and resistance factor design (LRFD) provisions. Connections designed according to this standard are permitted to be used in structures designed according to the LRFD or allowable strength design (ASD) provisions of the AISC *Seismic Provisions*.

CHAPTER 2

DESIGN REQUIREMENTS

2.1. SPECIAL AND INTERMEDIATE MOMENT FRAME CONNECTION TYPES

The connection types listed in Table 2.1 are prequalified for use in connecting beams to column flanges in special moment frames (SMF) and intermediate moment frames (IMF) within the limitations specified in this standard.

2.2. CONNECTION STIFFNESS

All connections contained in this standard shall be considered fully restrained (Type FR) for the purpose of seismic analysis.

Exception: For the Simpson Strong-Tie Strong Frame connection, a partially restrained (PR) connection, the seismic analysis must include the force-deformation characteristics of the specific connection per Section 12.9.

2.3. MEMBERS

The connections contained in this standard are prequalified in accordance with the requirements of the AISC *Seismic Provisions* when used to connect members meeting the limitations of Sections 2.3.1, 2.3.2, or 2.3.3, as applicable.

1. Rolled Wide-Flange Members

Rolled wide-flange members shall conform to the cross-section profile limitations applicable to the specific connection in this standard.

2. Built-up Members

Built-up members having a doubly symmetric, I-shaped cross section shall meet the following requirements:

- (1) Flanges and webs shall have width, depth, and thickness profiles similar to rolled wide-flange sections meeting the profile limitations for wide-flange sections applicable to the specific connection in this standard, and
- (2) Webs shall be continuously connected to flanges in accordance with the requirements of Sections 2.3.2a or 2.3.2b and the requirements of the AISC *Seismic Provisions*, as applicable.

2a. Built-up Beams

Within a zone extending from the beam end to a distance not less than one beam depth beyond the plastic hinge location, S_h , the web and flanges shall be connected using complete-joint-penetration (CJP) groove welds with a pair of reinforcing fillet

TABLE 2.1
Prequalified Moment Connections

Connection Type	Chapter	Systems
Reduced beam section (RBS)	5	SMF, IMF
Bolted unstiffened extended end plate (BUEEP)	6	SMF, IMF
Bolted stiffened extended end plate (BSEEP)	6	SMF, IMF
Bolted flange plate (BFP)	7	SMF, IMF
Welded unreinforced flange-welded web (WUF-W)	8	SMF, IMF
Cast bolted bracket (CBB)	9	SMF, IMF
ConXtech ConXL (ConXL)	10	SMF, IMF
SidePlate	11	SMF, IMF
Simpson Strong-Tie Strong Frame	12	SMF, IMF
Double-tee	13	SMF, IMF
Slotted web (SW)	14	SMF
DuraFuse Frames	15	SMF, IMF

welds unless specifically indicated otherwise in this standard. The minimum size of these fillet welds shall be the lesser of $\frac{5}{16}$ in. (8 mm) and the thickness of the beam web.

2b. Built-up Columns

Built-up columns shall conform to the provisions of subsections (1) through (4), as applicable. Built-up columns shall satisfy the requirements of the AISC *Specification*, except as modified in this section. Transfer of all internal forces and stresses between elements of the built-up column shall be through welds.

(1) I-Shaped Columns

The elements of built-up I-shaped columns shall conform to the requirements of the AISC *Seismic Provisions*.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, unless specifically indicated in this standard, the column webs and flanges shall be connected using CJP groove welds with a pair of reinforcing fillet welds. The minimum size of the fillet welds shall be the lesser of $\frac{5}{16}$ in. (8 mm) and the thickness of the column web.

Exception: For SidePlate moment connections, each column flange may be connected to the column web using a pair of continuous fillet welds. The required shear strength of the fillet welds, ϕR_n , shall equal the shear developed at the column flange-to-web connection where the shear force in the column is the smaller of

- (a) The nominal shear strength of the column per AISC *Specification* Equation G2-1.
- (b) The maximum shear force that can be developed in the column when plastic hinge(s) form in the connected beam(s).

(2) Boxed Wide-Flange Columns

The wide-flange shape of a boxed wide-flange column shall conform to the requirements of the AISC *Seismic Provisions*.

The width-to-thickness ratio, b/t , of plates used as flanges shall not exceed $0.6\sqrt{E/F_y}$, where b shall be taken as not less than the clear distance between plates.

The width-to-thickness ratio, h/t_w , of plates used only as webs shall conform to the requirements of the AISC *Seismic Provisions*.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, flange and web plates of boxed wide-flange columns shall be joined by CJP groove welds. Outside this zone, plate elements shall be continuously connected by fillet or groove welds.

(3) Built-up Box Columns

The width-to-thickness ratio, b/t , of plates used as flanges shall not exceed $0.6\sqrt{E/F_y}$, where b shall be taken as not less than the clear distance between web plates.

The width-to-thickness ratio, h/t_w , of plates used only as webs shall conform to the requirements of the AISC *Seismic Provisions*.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, flange and web plates of box columns shall be joined by CJP groove welds. Outside this zone, box column web and flange plates shall be continuously connected by fillet welds or groove welds.

Exception: For ConXL moment connections, partial-joint-penetration (PJP) groove welds conforming to the requirements of Section 10.3.2 shall be permitted within the zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange.

(4) Flanged Cruciform Columns

The elements of flanged cruciform columns, whether fabricated from rolled shapes or built up from plates, shall meet the requirements of the AISC *Seismic Provisions*.

Within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, the web of the tee-shaped sections shall be welded to the web of the continuous I-shaped section with CJP

groove welds with a pair of reinforcing fillet welds. The minimum size of fillet welds shall be the lesser of $\frac{5}{16}$ in. (8 mm) or the thickness of the column web. Continuity plates shall conform to the requirements for wide-flange columns.

Exception: For SidePlate moment connections, the web of the tee-shaped section(s) may be welded to the web of the continuous I-shaped section with a pair of continuous fillet welds. The required strength of the fillet welds, ϕR_n , shall equal the shear developed at the column web to tee-shaped section connection where the shear force in the column is the smaller of

- (a) The shear strength of the column section per AISC *Specification* Equation G2-1.
- (b) The maximum shear that can be developed in the column when plastic hinge(s) form in the connected beam(s).

3. Hollow Structural Sections (HSS)

The width-to-thickness ratio, h/t_w , of HSS members shall conform to the requirements of the AISC *Seismic Provisions* and shall conform to additional cross-section profile limitations applicable to the individual connection as specified in the applicable chapter.

4. Isolation of Concrete Slab

Where an individual connection requires isolation of the concrete slab, the column and connection elements at the face of the column shall be isolated from the concrete slab such that the rotation of the frame beam relative to the column joint is unimpeded by the concrete slab.

2.4. CONNECTION DESIGN PARAMETERS

1. Resistance Factors

Where available strengths are calculated in accordance with the AISC *Specification*, the resistance factors specified therein shall apply. When available strengths are calculated in accordance with this standard, the resistance factors ϕ_d and ϕ_n shall be used as specified in the applicable section of this standard. The values of ϕ_d and ϕ_n shall be taken as follows:

- (a) For ductile limit states

$$\phi_d = 1.00$$

- (b) For nonductile limit states

$$\phi_n = 0.90$$

2. Plastic Hinge Location

The distance of the plastic hinge from the face of the column, S_h , shall be taken in accordance with the requirements for the individual connection as specified herein.

3. Probable Maximum Moment at Plastic Hinge

The probable maximum moment at the plastic hinge shall be

$$M_{pr} = C_{pr}R_yF_yZ_e \quad (2.4-1)$$

where

C_{pr} = factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. Unless otherwise specifically indicated in this standard, the value of C_{pr} shall be

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \quad (2.4-2)$$

F_u = specified minimum tensile strength of yielding element, ksi (MPa)

F_y = specified minimum yield stress of yielding element, ksi (MPa)

R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y , as specified in the AISC *Seismic Provisions*

Z_e = effective plastic section modulus of section (or connection) at location of the plastic hinge, in.³ (mm³)

4. Shear Force at Plastic Hinge

Unless otherwise specifically indicated in this standard, the shear force at the plastic hinge, V_h , shall be

$$V_h = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (2.4-3)$$

where

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

$V_{gravity}$ = shear force resulting from $1.2D + f_1L + 0.15S$ acting on the beam between plastic hinge locations, where D is the dead load, S is the snow load, and f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5, kips (N)

In applying Equation 2.4-3, the values of $2M_{pr}/L_h$ and $V_{gravity}$ shall be taken as having the same sign.

User Note: The load combination of $1.2D + f_1L + 0.15S$ is in conformance with ASCE/SEI 7-22. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.15 for S when the roof configuration is such that it does not shed snow off of the structure.

User Note: Equation 2.4-3 can be derived from a free-body diagram of the portion of the beam between plastic hinge locations, assuming the moment at each plastic hinge is M_{pr} and with gravity load acting on the free-body diagram. The

shear at the plastic hinge will be different at the two ends of the beam because the shear force due to $2M_{pr}/L_h$ and the shear force due to gravity load will be additive at one end and will be of opposite sign at the other end. V_h is defined as the larger of these two values—that is, the value where $2M_{pr}/L_h$ and $V_{gravity}$ are additive.

Equation 2.4-3 assumes moment connections are provided at both ends of the beam. If only one end of the beam has a moment connection, the shear at the plastic hinge can be determined based on a free-body diagram between the plastic hinge at the moment connected end of the beam and the opposite end of the beam.

5. Probable Maximum Moment at Face of Column

Unless otherwise specifically indicated in this standard, the probable maximum moment at the face of the column, M_f , shall be

$$M_f = M_{pr} + V_h S_h \quad (2.4-4)$$

where

M_{pr} = probable maximum moment at plastic hinge, kip-in. (N-mm)

S_h = distance from face of column to plastic hinge, in. (mm)

V_h = shear force at plastic hinge location, kips (N)

User Note: Equation 2.4-4 can be derived from a free-body diagram of the portion of the beam between the plastic hinge location and the face of the column. This equation neglects the gravity load on the portion of the beam between the plastic hinge location and the face of the column because this gravity load typically has a negligible contribution to M_f . If desired, the gravity load on the portion of the beam between the plastic hinge location and the face of the column can be included in the calculation of M_f .

2.5. REQUIRED SHEAR STRENGTH OF BEAM

Unless otherwise specifically indicated in this standard, the required shear strength of the beam, V_u , shall be taken as the shear at the plastic hinge, V_h , plus shear due to gravity load applied to the beam between the plastic hinge and the face of the column, using the load combination in Section 2.4.4.

2.6. CONTINUITY PLATES

Unless otherwise specifically indicated in this standard, beam flange continuity plates shall be provided in accordance with the AISC *Seismic Provisions*.

2.7. PANEL ZONES

Unless otherwise specifically indicated in this standard, panel zones shall conform to the requirements of the AISC *Seismic Provisions*.

2.8. COLUMN-BEAM MOMENT RATIOS

Unless otherwise specifically indicated in this standard, column-beam moment ratios shall conform to the requirements of the AISC *Seismic Provisions*.

2.9. PROTECTED ZONE

The protected zone shall be as defined for each prequalified connection. Unless otherwise specifically indicated in this standard, the protected zone of the beam shall be defined as the area from the face of the column flange to one-half of the beam depth beyond the plastic hinge. The protected zone shall meet the requirements of the AISC *Seismic Provisions*, except as indicated in this standard.

CHAPTER 3

WELDING REQUIREMENTS

3.1. FILLER METALS

Filler metals shall conform to the requirements of the AISC *Seismic Provisions*.

3.2. WELDING PROCEDURES

Welding procedures shall be in accordance with the AISC *Seismic Provisions*.

3.3. BACKING AT BEAM-TO-COLUMN AND CONTINUITY PLATE-TO-COLUMN JOINTS

1. Steel Backing at Continuity Plates

Steel backing used at continuity plate-to-column welds need not be removed. At column flanges, steel backing left in place shall be attached to the column flange using a continuous $\frac{5}{16}$ in. (8 mm) fillet weld on the edge below the CJP groove weld.

When backing is removed, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet weld. The reinforcing fillet weld shall be continuous with a minimum size of $\frac{5}{16}$ in. (8 mm).

2. Steel Backing at Beam Bottom Flange

Where steel backing is used with CJP groove welds between the bottom beam flange and the column, the backing shall be removed.

Following the removal of steel backing, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet weld. The size of the reinforcing fillet weld leg adjacent to the column flange shall be a minimum of $\frac{5}{16}$ in. (8 mm), and the reinforcing fillet weld leg adjacent to the beam flange shall be such that the fillet weld toe is located on the beam flange base metal.

Exception: If the base metal and weld root are ground smooth after removal of the backing, the reinforcing fillet weld adjacent to the beam flange need not extend to the base metal.

3. Steel Backing at Beam Top Flange

Where steel backing is used with CJP groove welds between the top beam flange and the column, and the steel backing is not removed, the steel backing shall be attached to the column by a continuous $\frac{5}{16}$ in. (8 mm) fillet weld on the edge below the CJP groove weld.

4. Prohibited Welds at Steel Backing

Backing at beam flange-to-column flange joints shall not be welded to the underside of the beam flange, nor shall tack welds be permitted at this location. If fillet welds or tack welds are placed between the backing and the beam flange in error, they shall be repaired as follows:

- (1) The weld shall be removed such that the fillet weld or tack weld no longer attaches the backing to the beam flange.
- (2) The surface of the beam flange shall be ground flush and shall be free of defects.
- (3) Any gouges or notches shall be repaired. Repair welding shall be done with E7018 SMAW electrodes or other filler metals meeting the requirements of Section 3.1 for demand critical welds. A special welding procedure specification (WPS) is required for this repair. Following welding, the repair weld shall be ground smooth.

5. Nonfusible Backing at Beam Flange-to-Column Joints

Where nonfusible backing is used with CJP groove welds between the beam flanges and the column, the backing shall be removed and the root backgouged to sound weld metal and backwelded with a reinforcing fillet weld. The size of the reinforcing fillet weld leg adjacent to the column shall be a minimum of $\frac{5}{16}$ in. (8 mm), and the reinforcing fillet weld leg adjacent to the beam flange shall be such that the fillet weld toe is located on the beam flange base metal.

Exception: If the base metal and weld root are ground smooth after removal of the backing, the reinforcing fillet weld adjacent to the beam flange need not extend to the base metal.

3.4. WELD TABS

Where used, weld tabs shall be removed to within $\frac{1}{8}$ in. (3 mm) of the base metal surface and the end of the weld finished, except at continuity plates where removal to within $\frac{1}{4}$ in. (6 mm) of the plate edge shall be permitted. Removal shall be by air carbon arc cutting (CAC-A), grinding, chipping, or thermal cutting. The process shall be controlled to minimize errant gouging. The edges where weld tabs have been removed shall be finished to a surface roughness of 500 $\mu\text{in.}$ (12.5 μm) or better. The contour of the weld end shall provide a smooth transition to adjacent surfaces, free of notches, gouges, and sharp corners. Weld defects greater than $\frac{1}{16}$ in. (2 mm) deep shall be excavated and repaired by welding in accordance with an applicable WPS. Other weld defects shall be removed by grinding and faired to a slope not greater than 1:5.

3.5. TACK WELDS

In the protected zone, tack welds attaching backing and weld tabs shall be placed where they will be incorporated into a final weld.

3.6. CONTINUITY PLATES

Along the web, the corner clip shall be detailed so that the clip extends a distance of at least $1\frac{1}{2}$ in. (38 mm) beyond the k_{det} dimension given in the *AISC Manual* for the rolled shape. Along the flange, the plate shall be clipped to avoid interference with the fillet radius of the rolled shape and shall be detailed so that the clip does not exceed a distance of $\frac{1}{2}$ in. (13 mm) beyond the k_1 dimension given in the *AISC Manual* for the rolled shape. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. When a curved corner clip is used, it shall have a minimum radius of $\frac{1}{2}$ in. (13 mm).

At the end of the weld adjacent to the column web/flange juncture, weld tabs for continuity plates shall not be used, except when permitted by the engineer of record. Unless specified to be removed by the engineer of record, weld tabs shall not be removed when used in this location.

Where continuity plate welds are made without weld tabs near the column fillet radius, weld layers shall be permitted to be transitioned at an angle of 0° to 45° measured from the vertical plane. The effective length of the weld shall be defined as that portion of the weld having full size. Nondestructive testing (NDT) shall not be required on the tapered or transition portion of the weld not having full size.

3.7. QUALITY CONTROL AND QUALITY ASSURANCE

Quality control and quality assurance shall be in accordance with the *AISC Seismic Provisions*.

CHAPTER 4

BOLTING REQUIREMENTS

4.1. FASTENER ASSEMBLIES

Bolts shall be pretensioned high-strength bolts conforming to ASTM F3125/F3125M Grades A325, A325M, A490, A490M, F1852, or F2280, unless other fasteners are permitted by a specific connection.

4.2. INSTALLATION REQUIREMENTS

Installation requirements shall be in accordance with the AISC *Seismic Provisions* and the RCSC *Specification*, except as otherwise specifically indicated in this standard.

4.3. QUALITY CONTROL AND QUALITY ASSURANCE

Quality control and quality assurance shall be in accordance with the AISC *Seismic Provisions*.

CHAPTER 5

REDUCED BEAM SECTION (RBS) MOMENT CONNECTION

5.1. GENERAL

In a reduced beam section (RBS) moment connection (Figure 5.1), portions of the beam flanges are selectively trimmed in the region adjacent to the beam-to-column connection. Yielding and hinge formation are intended to occur primarily within the reduced section of the beam.

5.2. SYSTEMS

RBS connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions. The RBS connection is exempt from the requirements of Section 2.3.4.

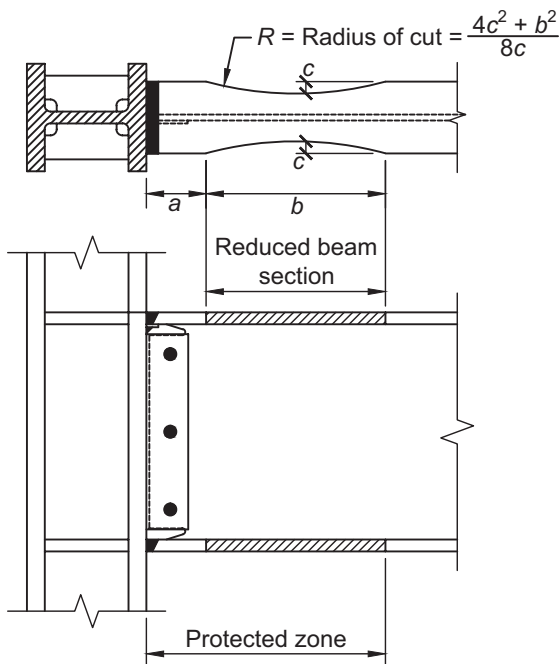


Fig. 5.1. Reduced beam section connection.

5.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.
- (2) Beam depth shall be limited to a maximum of W44 (W1100) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight shall be limited to a maximum of 408 lb/ft (610 kg/m).
- (4) Beam flange thickness shall be limited to a maximum of 2½ in. (53 mm).
- (5) The clear span-to-depth ratio of the beam shall be limited as follows:
 - (a) For SMF systems, 7 or greater.
 - (b) For IMF systems, 5 or greater.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.

When determining the width-to-thickness ratio of the flange, the value of b_f shall not be taken as less than the flange width at the ends of the center two-thirds of the reduced section provided that gravity loads do not shift the location of the plastic hinge a significant distance from the center of the reduced beam section.

- (7) Lateral bracing of beams shall be provided in conformance with the AISC *Seismic Provisions*. Supplemental lateral bracing shall be provided near the reduced section in conformance with the AISC *Seismic Provisions* for lateral bracing provided adjacent to the plastic hinges.

When supplemental lateral bracing is provided, its attachment to the beam shall be located no greater than $d/2$ beyond the end of the reduced beam section farthest from the face of the column, where d is the depth of the beam. No attachment of lateral bracing shall be made to the beam in the protected zone.

Exception: For both systems, where the beam supports a concrete structural slab that is connected between the protected zones with welded shear connectors spaced a maximum of 12 in. (300 mm) on center and where the beam depth, beam weight, and column depth are not larger than W36 (W920), 302 lb/ft (450 kg/m), and W36 (W920), respectively, supplemental top and bottom flange bracing at the reduced section is not required.

- (8) The protected zone shall consist of the portion of beam between the face of the column and the end of the RBS cut farthest from the face of the column.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled wide-flange shapes or built-up shapes permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) Rolled shape column depth shall be limited to W40 (W1000) maximum. The depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box-columns shall not have a width or depth exceeding 24 in. (600 mm). Boxed wide-flange columns shall not have a width or depth exceeding 24 in. (600 mm) if participating in orthogonal moment frames.
- (4) There is no limit on the weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Width-to-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

5.4. BEAM FLANGE-TO-COLUMN FLANGE WELD LIMITATIONS

Beam flange-to-column flange connections shall satisfy the following limitations:

- (1) Beam flanges shall be connected to column flanges using CJP groove welds. Beam flange welds shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions*.
- (2) Weld access hole geometry shall conform to the requirements of the AISC *Specification*.

5.5. BEAM WEB-TO-COLUMN FLANGE CONNECTION LIMITATIONS

Beam web-to-column flange connections shall satisfy the following limitations:

- (1) For SMF systems, the beam web shall be connected to the column flange using a CJP groove weld extending between weld access holes. The single-plate shear connection shall extend between the weld access holes as shown in Figure 5.1. The single-plate shear connection shall be permitted to be used as backing for the CJP groove weld. The thickness of the plate shall be at least $\frac{3}{8}$ in. (10 mm). Weld tabs are not required at the ends of the CJP groove weld at the beam web. Bolt holes in the beam web for the purpose of erection are permitted.
- (2) For IMF systems, the beam web shall be connected to the column flange as required for SMF systems.

Exception: For IMF, it is permitted to connect the beam web to the column flange using a bolted single-plate shear connection. The bolted single-plate shear connection shall be designed as a slip-critical connection, with the design slip resistance per bolt determined according to the AISC *Specification*. For seismic loading, the nominal bearing strength at bolt holes shall not be taken

greater than the value given by Equation J3-6a of the AISC *Specification*. The design shear strength of the single-plate shear connection shall be determined based on shear yielding of the gross section and on shear rupture of the net section. The plate shall be welded to the column flange with a CJP groove weld or with fillet welds on both sides of the plate. The minimum size of the fillet weld on each side of the plate shall be $\frac{3}{8}$ of the thickness of the plate. Standard holes shall be provided in the beam web and in the plate, except that short-slotted holes (with the slot parallel to the beam flanges) may be used in either the beam web or in the plate, but not in both. Bolts are permitted to be pretensioned either before or after welding.

5.6. FABRICATION OF FLANGE CUTS

The reduced beam section shall be made using thermal cutting to produce a smooth curve. The maximum surface roughness of the thermally cut surface shall be 500 μm . (12.5 μm) in accordance with ANSI B46.1, as measured using AWS C4.1 Sample 4 or a similar visual comparator. All transitions between the reduced beam section and the unmodified beam flange shall be rounded in the direction of the flange length to minimize notch effects due to abrupt transitions. Corners between the reduced section surface and the top and bottom of the flanges shall be ground to remove sharp edges, but a minimum chamfer or radius is not required.

Thermal cutting tolerances shall be plus or minus $\frac{1}{4}$ in. (6 mm) from the theoretical cut line. The beam effective flange width at any section shall have a tolerance of plus or minus $\frac{3}{8}$ in. (10 mm).

Gouges and notches that occur in the thermally cut RBS surface may be repaired by grinding if not more than $\frac{1}{4}$ in. (6 mm) deep. The gouged or notched area shall be faired in by grinding so that a smooth transition exists, and the total length of the area ground for the transition shall be no less than five times the depth of the removed gouge on each side of the gouge. If a sharp notch exists, the area shall be inspected by magnetic particle testing (MT) after grinding to ensure that the entire depth of notch has been removed. Grinding that increases the depth of the RBS cut more than $\frac{1}{4}$ in. (6 mm) beyond the specified depth of cut is not permitted.

Gouges and notches that exceed $\frac{1}{4}$ in. (6 mm) in depth, but not exceeding $\frac{1}{2}$ in. (13 mm) in depth, and those notches and gouges where repair by grinding would increase the effective depth of the RBS cut beyond tolerance, may be repaired by welding. The notch or gouge shall be removed and ground to provide a smooth root radius of not less than $\frac{1}{4}$ in. (6 mm) in preparation for welding. The repair area shall be preheated to a minimum temperature of 150°F (66°C) or the value specified in AWS D1.1/D1.1M, whichever is greater, measured at the location of the weld repair.

Notches and gouges exceeding $\frac{1}{2}$ in. (13 mm) in depth shall be repaired only with a method approved by the engineer of record.

5.7. DESIGN PROCEDURE

Step 1. Choose trial values for the beam sections, column sections, and RBS dimensions a , b , and c (Figure 5.1) subject to the following limits:

$$0.5b_{bf} \leq a \leq 0.75b_{bf} \quad (5.7-1)$$

$$0.65d \leq b \leq 0.85d \quad (5.7-2)$$

$$0.1b_{bf} \leq c \leq 0.25b_{bf} \quad (5.7-3)$$

where

a = horizontal distance from face of column flange to start of an RBS cut, in. (mm)

b = length of RBS cut, in. (mm)

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

c = depth of cut at center of reduced beam section, in. (mm)

d = depth of beam, in. (mm)

Confirm that the beams and columns are adequate for all load combinations specified by the applicable building code, including the reduced section of the beam, and that the design story drift for the frame complies with applicable limits specified by the applicable building code. Calculation of elastic drift shall consider the effect of the reduced beam section. In lieu of more detailed calculations, effective elastic drifts may be calculated by multiplying elastic drifts based on gross beam sections by 1.1 for flange reductions up to 50% of the beam flange width. Linear interpolation may be used for lesser values of beam width reduction.

Step 2. Compute the plastic section modulus at the center of the reduced beam section, Z_{RBS} :

$$Z_{RBS} = Z_b - 2ct_{bf}(d - t_{bf}) \quad (5.7-4)$$

where

Z_b = plastic section modulus of beam, in.³ (mm³)

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

Step 3. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3, where $Z_e = Z_{RBS}$.

Step 4. Compute the distance of the plastic hinge from the face of the column, S_h :

$$S_h = a + \frac{b}{2} \quad (5.7-5)$$

Step 5. Compute the shear force at the plastic hinge, V_h , in accordance with Section 2.4.4.

Step 6. Compute the probable maximum moment at the face of the column, M_f , in accordance with Section 2.4.5.

Step 7. Compute the plastic moment of the beam based on the expected yield stress, M_{pe} :

$$M_{pe} = R_y F_y Z_b \quad (5.7-6)$$

where

F_y = specified minimum yield stress, ksi (MPa)

R_y = ratio of expected yield stress to specified minimum yield stress

Step 8. Check the flexural strength of the beam at the face of the column:

$$M_f \leq \phi_d M_{pe} \quad (5.7-7)$$

If Equation 5.7-7 is not satisfied, adjust the values of c , a , and b , or adjust the section size, and repeat Steps 2 through 8.

Step 9. Check the design shear strength of the beam in accordance with the AISC *Specification*.

The required shear strength of the beam shall be determined in accordance with Section 2.5.

Step 10. Design the beam web-to-column connection in accordance with Section 5.5.

The required shear strength of the beam web-to-column connection shall be taken equal to the required shear strength of the beam.

Step 11. Check continuity plate requirements in accordance with Section 2.6.

Step 12. Check the panel zone in accordance with Section 2.7.

Step 13. Check the column-beam moment ratio in accordance with Section 2.8.

CHAPTER 6

BOLTED UNSTIFFENED (BUEEP) AND STIFFENED EXTENDED END-PLATE (BSEEP) MOMENT CONNECTIONS

6.1. GENERAL

Bolted end-plate connections are made by welding the beam to an end plate and bolting the end plate to a column flange. The three end-plate configurations shown in Figure 6.1 are covered in this section and are prequalified under the AISC *Seismic Provisions* within the limitations of this standard.

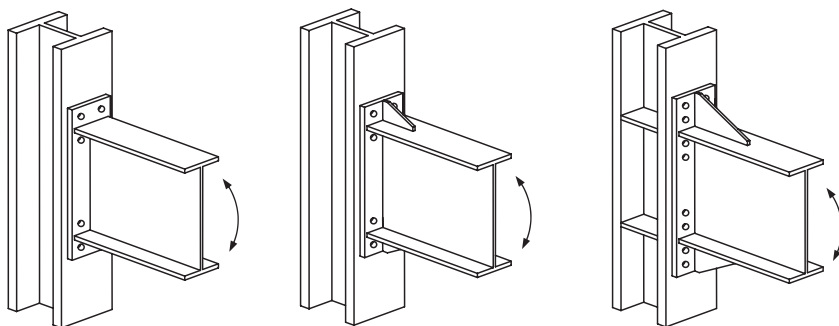
The behavior of this type of connection can be controlled by a number of different limit states, including flexural yielding of the beam section, flexural yielding of the end plates, yielding of the column panel zone, tension rupture of the end-plate bolts, shear rupture of the end-plate bolts, or rupture of various welded joints. The design criteria provide sufficient strength in the elements of the connections to ensure that the inelastic deformation of the connection is achieved by beam yielding.

6.2. SYSTEMS

Extended end-plate moment connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems.

Exception: Extended end-plate moment connections with concrete structural slabs are prequalified only if

- (1) In addition to the limitations of Section 6.3, the nominal beam depth is not less than 24 in. (600 mm);



(a) Four-bolt unstiffened, 4E (b) Four-bolt stiffened, 4ES (c) Eight-bolt stiffened, 8ES

Fig. 6.1. Extended end-plate configurations.

TABLE 6.1
Parametric Limitations on Prequalification

Parameter	Four-Bolt Unstiffened (4E)		Four-Bolt Stiffened (4ES)		Eight-Bolt Stiffened (8ES)	
	Maximum in. (mm)	Minimum in. (mm)	Maximum in. (mm)	Minimum in. (mm)	Maximum in. (mm)	Minimum in. (mm)
t_{bf}	¾ (19)	⅝ (10)	¾ (19)	⅝ (10)	1 (25)	⅞ (14)
b_{bf}	9¼ (230)	6 (150)	9 (230)	6 (150)	12¼ (310)	7½ (190)
d	24 (600)	13¾ (350)	24 (600)	13¾ (350)	36 (910)	18 (460)
t_p	2¼ (57)	½ (13)	1½ (38)	½ (13)	2½ (64)	¾ (19)
b_p	10¾ (270)	7 (180)	10¾ (270)	7 (180)	15 (380)	9 (230)
g	6 (150)	4 (100)	6 (150)	3¼ (83)	6 (150)	5 (130)
ρ_{fi}, ρ_{fo}	4½ (110)	1½ (38)	5½ (140)	1¾ (44)	2 (51)	1⅝ (41)
ρ_b	–	–	–	–	3¾ (95)	3½ (89)

b_{bf} = width of beam flange, in. (mm)
 b_p = width of end plate, in. (mm)
 d = depth of connecting beam, in. (mm)
 g = horizontal distance between bolts, in. (mm)
 ρ_b = vertical distance between the inner and outer row of bolts in an 8ES connection, in. (mm)
 ρ_{fi} = vertical distance from the inside of a beam tension flange to the nearest inside bolt row, in. (mm)
 ρ_{fo} = vertical distance from the outside of a beam tension flange to the nearest outside bolt row, in. (mm)
 t_{bf} = thickness of beam flange, in. (mm)
 t_p = thickness of end plate, in. (mm)

- (2) There are no shear connectors within 1.5 times the beam depth from the face of the connected column flange; and
- (3) The concrete slab is isolated in accordance with Section 2.3.4.

6.3. PREQUALIFICATION LIMITS

Table 6.1 is a summary of the range of parameters that have been satisfactorily tested. All connection elements shall be within the ranges shown.

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3. At moment-connected ends of welded built-up sections, within at least the depth of beam or 3 times the width of flange, whichever is less, the beam web and flanges shall be connected using either a CJP groove weld or a pair of fillet welds each having a size 75% of the beam

web thickness but not less than $\frac{1}{4}$ in. (6 mm). For the remainder of the beam, the weld size shall not be less than that required to accomplish shear transfer from the web to the flanges.

- (2) Beam depth, d , shall be limited to values shown in Table 6.1.
- (3) There is no limit on the weight per foot of beams.
- (4) Beam flange thickness shall be limited to the values shown in Table 6.1.
- (5) The clear span-to-depth ratio of the beam shall be limited as follows:
 - (a) For SMF systems, 7 or greater.
 - (b) For IMF systems, 5 or greater.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of beams shall be provided in accordance with the AISC *Seismic Provisions*. To satisfy the requirements of the AISC *Seismic Provisions* for lateral bracing at plastic hinges, supplemental lateral bracing shall be provided at the top and bottom beam flanges and shall be located between the end of the protected zone and $d/2$ from the end of the protected zone. No attachment of lateral bracing shall be made within the protected zone.

Exception: For both SMF and IMF systems where the beam supports a concrete structural slab that is connected along the beam span between protected zones with welded shear connectors spaced at a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at plastic hinges is not required.

- (8) The protected zone shall be determined as follows:
 - (a) For unstiffened extended end-plate connections: the portion of beam between the face of the column and a distance from the face of the end plate equal to the depth of the beam or 3 times the width of the beam flange from the face of the column, whichever is less.
 - (b) For stiffened extended end-plate connections: the portion of beam between the face of the column and a distance equal to the location of the end of the stiffener plus one-half the depth of the beam or 3 times the width of the beam flange, whichever is less.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled wide-flange shapes, welded built-up I-shapes, or flanged cruciform sections permitted in Section 2.3.
- (2) The end plate shall be connected to the flange of the column.
- (3) Rolled shape column depth shall be limited to W36 (W920) maximum. The depth of built-up wide-flange columns shall not exceed that for rolled shapes.

Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes.

- (4) There is no limit on the weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Width-to-thickness ratios for the flanges and web of the column shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

6.4. CONTINUITY PLATES

Continuity plates shall satisfy the following limitations:

- (1) The need for continuity plates shall be determined in accordance with Section 6.7.
- (2) When provided, continuity plates shall conform to the requirements of Section 6.7.
- (3) Continuity plates shall be attached to columns by welds in accordance with the AISC *Seismic Provisions*.

Exception: Continuity plates less than or equal to $\frac{3}{8}$ in. (10 mm) shall be permitted to be welded to column flanges using double-sided fillet welds. The required strength of the fillet welds shall not be less than $F_y A_c$, where A_c is defined as the contact areas between the continuity plate and the column flanges that have attached beam flanges and F_y is defined as the specified minimum yield stress of the continuity plate.

6.5. BOLTS

Bolts shall conform to the requirements of Chapter 4.

6.6. CONNECTION DETAILING

1. Gage

The gage, g , is as defined in Figures 6.2 through 6.4. The maximum gage dimension is limited to the width of the connected beam flange.

2. Pitch and Row Spacing

The minimum pitch distance is the bolt diameter plus $\frac{1}{2}$ in. (13 mm) for bolts up to 1 in. (25 mm) diameter and the bolt diameter plus $\frac{3}{4}$ in. (19 mm) for larger diameter bolts. The pitch distances, p_{fi} and p_{fo} , are the distances from the face of the beam flange to the centerline of the nearer bolt row, as shown in Figures 6.2 through 6.4. The pitch distances, p_{si} and p_{so} , are the distances from the face of the continuity plate to the centerline of the nearer bolt row, as shown in Figures 6.2 through 6.4.

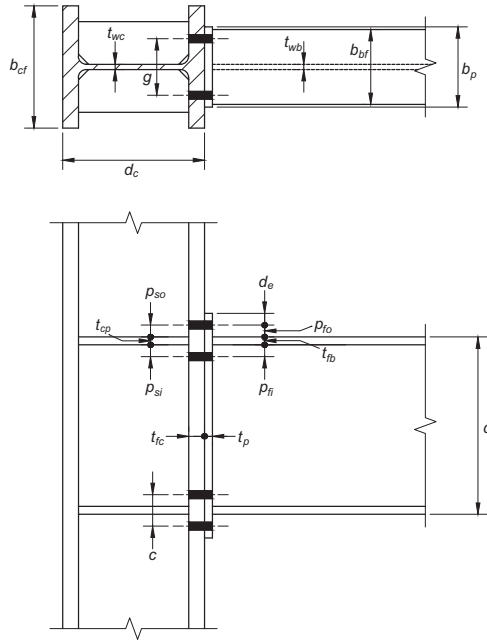


Fig. 6.2. Four-bolt unstiffened extended end-plate (4E) geometry.

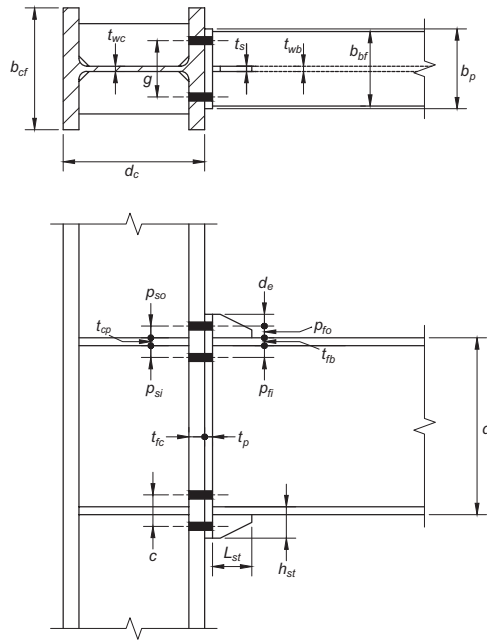


Fig. 6.3. Four-bolt stiffened extended end-plate (4ES) geometry.

The spacing, p_b , is the distance between the inner and outer row of bolts in an 8ES end-plate moment connection and is shown in Figure 6.4. The spacing of the bolt rows shall be at least $2\frac{2}{3}$ times the bolt diameter.

User Note: A distance of 3 times the bolt diameter is preferred. The distance must be sufficient to provide clearance for any welds in the region.

3. End-Plate Width

The width of the end plate shall be greater than or equal to the connected beam flange width. The effective end-plate width shall not be taken greater than the connected beam flange plus 1 in. (25 mm).

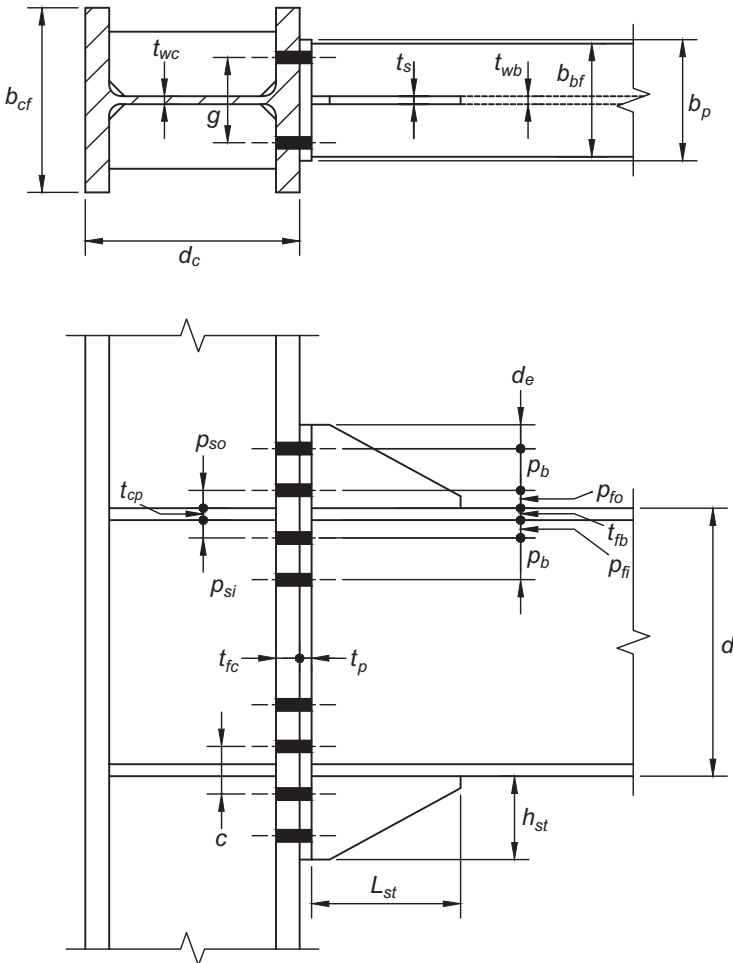


Fig. 6.4. Eight-bolt stiffened extended end-plate (8ES) geometry.

4. End-Plate Stiffener

The two extended stiffened end-plate connections, Figures 6.1(b) and (c), require a stiffener welded between the connected beam flange and the end plate. The minimum stiffener length, L_{st} , shall be

$$L_{st} = \frac{h_{st}}{\tan 30^\circ} \quad (6.6-1)$$

where h_{st} is the height of the stiffener, equal to the height of the end plate from the outside face of the beam flange to the end of the end plate as shown in Figure 6.5.

The stiffener plates shall be terminated at the beam flange and at the end of the end plate with landings not less than 1 in. (25 mm) long. The stiffener shall be clipped where it meets the beam flange and end plate to provide clearance between the stiffener and the beam flange weld.

When the beam and end-plate stiffeners have the same material strengths, the thickness of the stiffeners shall be greater than or equal to the beam web thickness. If the beam and end-plate stiffener have different material strengths, the thickness of the stiffener shall not be less than the ratio of the beam-to-stiffener plate material yield stresses times the beam web thickness.

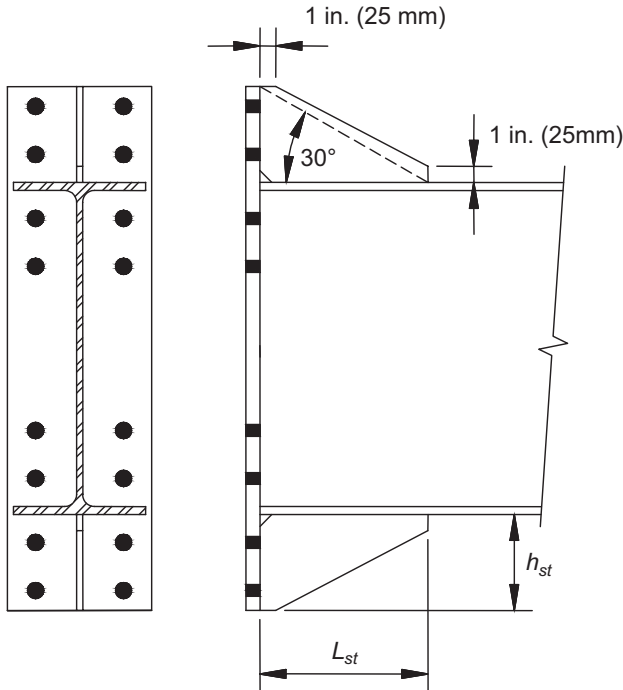


Fig. 6.5. End-plate stiffener layout and geometry for 8ES. Geometry for 4ES similar.

5. Finger Shims

The use of finger shims (illustrated in Figure 6.6) at the top and/or bottom of the connection and on either or both sides is permitted, subject to the limitations of the *RCSC Specification*.

6. Welding Details

Welding of the beam to the end plate shall conform to the following limitations:

- (1) Weld access holes shall not be used.
- (2) The beam flange to end-plate joint shall be made using a CJP groove weld without backing. The CJP groove weld shall be made such that the root of the weld is on the beam web side of the flange. The inside face of the flange shall have a $\frac{5}{16}$ in. (8 mm) fillet weld. These welds shall be demand critical.
- (3) For stiffened end-plate connections, the beam flange-to-end-plate weld shall be made before the stiffener is installed.
- (4) The beam web-to-end-plate joint shall be made using either fillet welds or CJP groove welds. When used, the fillet welds shall be sized to develop the full

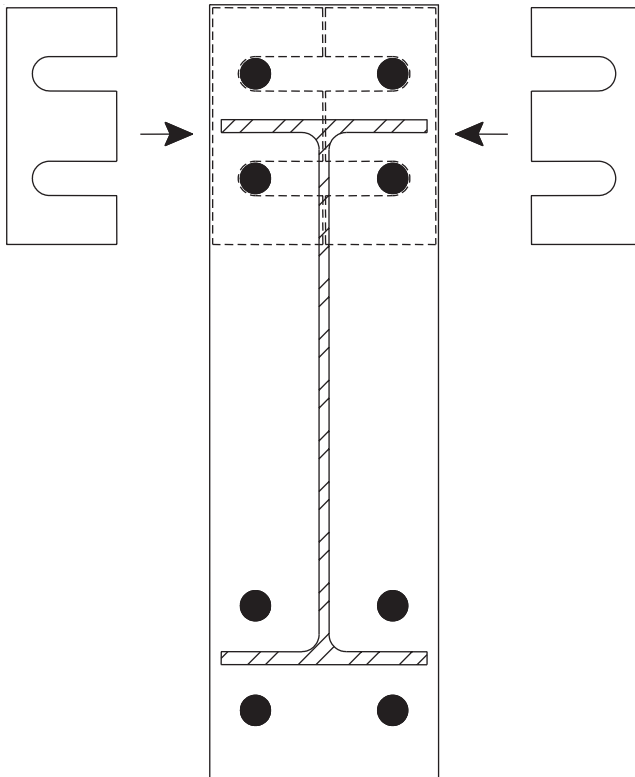


Fig. 6.6. Typical use of finger shims.

strength of the beam web in tension from the inside face of the flange to 6 in. (150 mm) beyond the bolt row farthest from the beam flange.

- (5) Backgouging of the root is not required in the flange directly above and below the beam web for a length equal to $1.5k_1$. A full-depth PJP groove weld shall be permitted at this location.
- (6) When used, all end-plate-to-stiffener joints shall be made using CJP groove welds.

Exception: When the stiffener is $\frac{3}{8}$ in. (10 mm) thick or less, it is permitted to use fillet welds that develop the strength of the stiffener.

6.7. DESIGN PROCEDURE

Connection geometry is shown in Figures 6.2, 6.3, and 6.4 for the 4E, 4ES, and 8ES connections, respectively.

1. End Plate and Bolt Design

Step 1. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3, where $Z_e = Z_b$ = the plastic section modulus of the beam.

Step 2. Determine the distance of the plastic hinge from the face of the column, S_h .

For an unstiffened connection (4E)

$$S_h = \text{the lesser of } d/2 \text{ or } 3b_f \quad (6.7-1)$$

where

b_f = width of beam flange, in. (mm)

d = depth of beam, in. (mm)

For a stiffened connection (4ES, 8ES)

$$S_h = L_{st} + t_p \quad (6.7-2)$$

where

L_{st} = length of end-plate stiffener as shown in Figure 6.5, in. (mm)

t_p = thickness of end plate, in. (mm)

Step 3. Compute the shear force at the plastic hinge, V_h , in accordance with Section 2.4.4.

Step 4. Compute the probable maximum moment at the face of the column, M_f , in accordance with Section 2.4.5.

Step 5. Select one of the three end-plate moment connection configurations and establish preliminary values for the connection geometry (g , p_{fi} , p_{fo} , p_b , g , h_i , etc.) and bolt grade.

Step 6. Determine the required bolt diameter, $d_{b,req}$, using one of the following expressions.

For four-bolt connections (4E, 4ES)

$$d_{b,req} = \sqrt{\frac{2M_f}{\pi\phi_n F_{nt} (h_1 + h_2)}} \quad (6.7-3)$$

For eight-bolt connections (8ES)

$$d_{b,req} = \sqrt{\frac{2M_f}{\pi\phi_n F_{nt} (h_1 + h_2 + h_3 + h_4)}} \quad (6.7-4)$$

where

F_{nt} = nominal tensile stress of bolt from the AISC *Specification*, ksi (MPa)

h_i = distance from centerline of the beam compression flange to the centerline of i th tension bolt line, in. (mm)

Step 7. Select a trial bolt diameter, d_b , not less than that required in Section 6.7.1, Step 6.

Step 8. Compute the required end-plate thickness, $t_{p,req}$:

$$t_{p,req} = \sqrt{\frac{1.11 M_f}{\phi_d F_{yp} Y_p}} \quad (6.7-5)$$

where

F_{yp} = specified minimum yield stress of end-plate material, ksi (Mpa)

Y_p = end-plate yield line mechanism parameter from Tables 6.2, 6.3, or 6.4, in. (mm)

Step 9. Select an end-plate thickness, t_p , not less than the required value.

Step 10. Compute the factored beam flange force, F_{fu} :

$$F_{fu} = \frac{M_f}{d - t_{bf}} \quad (6.7-6)$$

where

d = depth of beam, in. (mm)

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

Step 11. Check shear yielding of the extended portion of the four-bolt extended unstiffened end plate (4E):

$$F_{fu}/2 \leq \phi_d(0.6)F_{yp}b_p t_p \quad (6.7-7)$$

where b_p is the width of the end plate, in. (mm), to be taken as not greater than the width of the beam flange plus 1 in. (25 mm).

If Equation 6.7-7 is not satisfied, increase the end-plate thickness or increase the yield stress of the end-plate material.

TABLE 6.3
Summary of Four-Bolt Extended Stiffened
End-Plate Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ($d_e \leq s$)	Case 2 ($d_e > s$)	
Case 1 ($d_e \leq s$)	$Y_p = \frac{b_p}{2} \left[h_2 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_1 \left(\frac{1}{p_{lo}} + \frac{1}{2d_e} \right) \right] + \frac{2}{g} [h_2(p_{fi} + s) + h_1(d_e + p_{lo})]$	
Case 2 ($d_e > s$)	$Y_p = \frac{b_p}{2} \left[h_2 \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_1 \left(\frac{1}{s} + \frac{1}{p_{lo}} \right) \right] + \frac{2}{g} [h_2(p_{fi} + s) + h_1(s + p_{lo})]$	
$s = \frac{1}{2} \sqrt{b_p g}$; Note: If $p_{fi} > s$, use $p_{fi} = s$.		

TABLE 6.4
Summary of Eight-Bolt Extended Stiffened
End-Plate Yield Line Mechanism Parameter

End-Plate Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ($d_e \leq s$)	Case 2 ($d_e > s$)	
<p>Case 1 ($d_e \leq s$)</p>	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{2d_e} \right) + h_2 \left(\frac{1}{\rho_{io}} \right) + h_3 \left(\frac{1}{\rho_{fi}} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(d_e + \frac{3\rho_b}{4} \right) + h_2 \left(\rho_{io} + \frac{\rho_b}{4} \right) + h_3 \left(\rho_{fi} + \frac{3\rho_b}{4} \right) + h_4 \left(s + \frac{\rho_b}{4} \right) \right] + g$	
<p>Case 2 ($d_e > s$)</p>	$Y_p = \frac{b_p}{2} \left[h_1 \left(\frac{1}{s} \right) + h_2 \left(\frac{1}{\rho_{io}} \right) + h_3 \left(\frac{1}{\rho_{fi}} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(s + \frac{\rho_b}{4} \right) + h_2 \left(\rho_{io} + \frac{3\rho_b}{4} \right) + h_3 \left(\rho_{fi} + \frac{\rho_b}{4} \right) + h_4 \left(s + \frac{3\rho_b}{4} \right) \right] + g$	
<p>$s = \frac{1}{2} \sqrt{b_p g}$; Note: If $\rho_{fi} > s$, use $\rho_{fi} = s$.</p>		

Step 12. Check shear rupture of the extended portion of the end plate in the four-bolt extended unstiffened end plate (4E):

$$F_{ju}/2 \leq \phi_n(0.6)F_{up}A_n \quad (6.7-8)$$

where

A_n = net area of end plate

= $t_p[b_p - 2(d_b + 1/8)]$ when standard holes are used, in.²

= $t_p[b_p - 2(d_b + 3)]$ when standard holes are used, mm²

F_{up} = specified minimum tensile stress of end-plate material, ksi (MPa)

d_b = diameter of bolt, in. (mm)

Step 13. If using either the four-bolt extended stiffened end plate (4ES) or the eight-bolt extended stiffened end-plate (8ES) connection, select the end-plate stiffener thickness, and design the stiffener-to-beam flange and stiffener-to-end-plate welds.

$$t_s \geq t_{bw} \left(\frac{F_{yb}}{F_{ys}} \right) \quad (6.7-9)$$

where

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

F_{ys} = specified minimum yield stress of stiffener material, ksi (MPa)

t_{bw} = thickness of beam web, in. (mm)

t_s = end-plate stiffener thickness, in. (mm)

The stiffener geometry shall conform to the requirements of Section 6.6.4. In addition, to prevent local buckling of the stiffener plate, the following width-to-thickness criterion shall be satisfied:

$$\frac{h_{st}}{t_s} \leq 0.56 \sqrt{\frac{E}{F_{ys}}} \quad (6.7-10)$$

where h_{st} is the height of the stiffener, in. (mm), equal to the height of the end plate from the outside face of the beam flange to the end of the end plate.

The stiffener-to-beam-flange and stiffener-to-end-plate welds shall be designed to develop the stiffener plate in shear at the beam flange and in tension at the end plate. Either fillet or CJP groove welds are suitable for the weld of the stiffener plate to the beam flange. CJP groove welds shall be used for the stiffener-to-end-plate weld. If the stiffener is $3/8$ in. (10 mm) thick or less, double-sided fillet welds are permitted.

Step 14. Check the design shear strength of the beam in accordance with the AISC *Specification*.

The required shear strength of the beam shall be determined in accordance with Section 2.5.

Step 15. Determine the required shear strength of the beam-to-column connection. The required shear strength of the beam-to-column connection, V_u , shall be taken equal to the required shear strength of the beam.

Step 16. The bolt shear rupture strength of the connection is provided by the bolts at one (compression) flange; thus

$$V_u \leq \phi_n n_b F_{nv} A_b \quad (6.7-11)$$

where

A_b = nominal gross area of bolt, in.² (mm²)

F_{nv} = nominal shear stress of bolt from the AISC *Specification*, ksi (MPa)

n_b = number of bolts at compression flange
 = 4 for 4E and 4ES connections
 = 8 for 8ES connections

Step 17. Check bolt bearing and tearout failure of the end plate and column flange:

$$V_u \leq \phi_n \sum r_{ni} \quad (6.7-12)$$

where

$r_{ni} = 1.2l_c t F_u < 2.4d_b t F_u$ for each bolt

and

F_u = specified minimum tensile strength of end plate or column flange material, ksi (MPa)

l_c = clear distance, in direction of force, between edge of the hole and edge of the adjacent hole or edge of material, in. (mm)

t = thickness of end plate or column flange, in. (mm)

Step 18. Design the flange-to-end-plate and web-to-end-plate welds using the requirements of Section 6.6.6.

2. Column-Side Design

Step 1. Check the column flange for flexural yielding:

$$t_{cf} \geq \sqrt{\frac{1.11 M_f}{\phi_d F_{yc} Y_c}} \quad (6.7-13)$$

where

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

Y_c = unstiffened column flange yield line mechanism parameter from Table 6.5 or Table 6.6, in. (mm)

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

User Note: The yield line mechanism parameters, Y_c , from Table 6.5 and Table 6.6 assume that the distance from the top-most bolt line to the top of the column shall be greater than the dimension s .

If Equation 6.7-13 is not satisfied, increase the column size or add continuity plates.

If continuity plates are added, check Equation 6.7-13 using Y_c for the stiffened column flange from Tables 6.5 and 6.6.

TABLE 6.5
Summary of Four-Bolt Extended Column
Flange Yield Line Mechanism Parameter

Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
Unstiffened Column Flange	$Y_c = \frac{b_{cf}}{2} \left[h_2 \left(\frac{1}{s} \right) + h_1 \left(\frac{1}{s} \right) \right] + \frac{2}{g} \left[h_2 \left(s + \frac{3c}{4} \right) + h_1 \left(s + \frac{c}{4} \right) + \frac{c^2}{2} \right] + \frac{g}{2}$ $s = \frac{1}{2} \sqrt{b_{cf} g}$
Stiffened Column Flange	$Y_c = \frac{b_{cf}}{2} \left[h_2 \left(\frac{1}{s} + \frac{1}{p_{si}} \right) + h_1 \left(\frac{1}{s} + \frac{1}{p_{so}} \right) \right] + \frac{2}{g} \left[h_2 (s + p_{si}) + h_1 (s + p_{so}) \right]$ $s = \frac{1}{2} \sqrt{b_{cf} g}; \text{ Note: If } p_{fi} > s, \text{ use } p_{fi} = s.$

Step 2. If continuity plates are required for column flange flexural yielding, determine the required stiffener force.

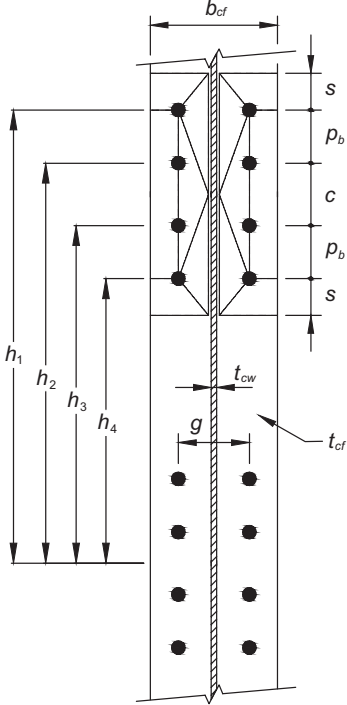
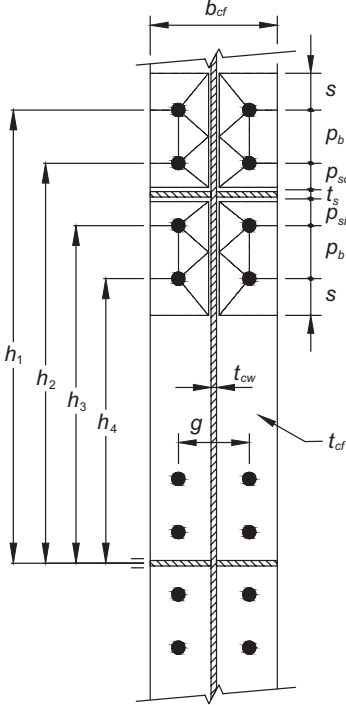
The column flange design flexural strength is

$$\phi_d M_{cf} = \phi_d F_{yc} Y_c t_{cf}^2 \quad (6.7-14)$$

where Y_c is the unstiffened column yield line mechanism parameter from Table 6.5 or Table 6.6, in. (mm). Therefore, the equivalent column flange design force is

$$\phi_d R_n = \frac{\phi_d M_{cf}}{(d - t_{bf})} \quad (6.7-15)$$

TABLE 6.6
Summary of Eight-Bolt Extended Column
Flange Yield Line Mechanism Parameter

Unstiffened Column Flange Geometry and Yield Line Pattern	Stiffened Column Flange Geometry and Yield Line Pattern
	
<p>Unstiffened Column Flange</p>	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(p_b + \frac{c}{2} + s \right) + h_2 \left(\frac{p_b}{2} + \frac{c}{4} \right) + h_3 \left(\frac{p_b}{2} + \frac{c}{2} \right) + h_4 (s) \right] + \frac{g}{2}$ $s = \frac{1}{2} \sqrt{b_{cf} g}$
<p>Stiffened Column Flange</p>	$Y_c = \frac{b_{cf}}{2} \left[h_1 \left(\frac{1}{s} \right) + h_2 \left(\frac{1}{p_{so}} \right) + h_3 \left(\frac{1}{p_{si}} \right) + h_4 \left(\frac{1}{s} \right) \right]$ $+ \frac{2}{g} \left[h_1 \left(s + \frac{p_b}{4} \right) + h_2 \left(p_{so} + \frac{3p_b}{4} \right) + h_3 \left(p_{si} + \frac{p_b}{4} \right) + h_4 \left(s + \frac{3p_b}{4} \right) + p_b^2 \right] + g$ $s = \frac{1}{2} \sqrt{b_{cf} g}; \text{ Note: If } p_{fi} > s, \text{ use } p_{fi} = s.$

Using $\phi_d R_n$, the required force for continuity plate design is determined in Section 6.7.2, Step 5.

Step 3. Check the column web local yielding strength of the unstiffened column web at the beam flanges.

Strength requirement

$$F_{fu} \leq \phi_d R_n \quad (6.7-16)$$

$$R_n = (6C_t k_c + l_b) F_{yc} t_{cw} \quad (6.7-17)$$

where

$C_t = 0.5$, if the distance from the column top to the top face of the beam flange is less than the depth of the column

$= 1.0$, otherwise

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

k_c = distance from outer face of column flange to web toe of fillet (design value) or fillet weld, in. (mm)

$l_b = t_{bf} + 2w + 2t_p$, in. (mm)

t_{cw} = thickness of column web, in. (mm)

w = leg size of double fillet weld if used in the beam flange-to-end-plate weld, in. (mm)

If the strength requirement of Equation 6.7-16 is not satisfied, column web continuity plates are required.

Step 4. Check the unstiffened column web local crippling strength at the beam compression flange.

Strength requirement

$$F_{fu} \leq \phi R_n \quad (6.7-18)$$

where $\phi = 0.75$.

- (a) When F_{fu} is applied at a distance greater than or equal to $d_c/2$ from the end of the column

$$R_n = 0.80 t_{cw}^2 \left[1 + 3 \left(\frac{l_b}{d_c} \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{E F_{yc} t_{cf}}{t_{cw}}} \quad (6.7-19)$$

- (b) When F_{fu} is applied at a distance less than $d_c/2$ from the end of the column

- (i) For $l_b/d_c \leq 0.2$

$$R_n = 0.40 t_{cw}^2 \left[1 + 3 \left(\frac{l_b}{d_c} \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{E F_{yc} t_{cf}}{t_{cw}}} \quad (6.7-20)$$

(ii) For $l_b/d_c > 0.2$

$$R_n = 0.40t_{cw}^2 \left[1 + \left(\frac{4l_b}{d_c} - 0.2 \right) \left(\frac{t_{cw}}{t_{cf}} \right)^{1.5} \right] \sqrt{\frac{EF_{yc}t_{cf}}{t_{cw}}} \quad (6.7-21)$$

where

d_c = overall depth of column, in. (mm)

$l_b = t_{bf} + 2w + 2t_p$, in. (mm)

t_p = end-plate thickness, in. (mm)

w = leg size of double fillet weld if used in the beam flange-to-end-plate weld, in. (mm)

If the strength requirement of Equation 6.7-18 is not satisfied, then column web continuity plates are required.

Step 5. If continuity plates are required for any of the column side limit states, the required strength is

$$F_{su} = F_{fu} - \min(\phi R_n) \quad (6.7-22)$$

where $\min(\phi R_n)$ is the minimum design strength value from Section 6.7.2, Step 2 (column flange local bending), Step 3 (column web local yielding), and Step 4 (column web local crippling)

The design of the continuity plates shall also conform to AISC *Seismic Provisions* Chapter E, and the welds shall be designed in accordance with Section 6.4(3).

Step 6. Check the panel zone in accordance with Section 2.7.

Step 7. Check the column-beam moment ratio in accordance with Section 2.8.

CHAPTER 7

BOLTED FLANGE PLATE (BFP) MOMENT CONNECTION

7.1. GENERAL

Bolted flange plate (BFP) moment connections utilize plates welded to column flanges and bolted to beam flanges. The top and bottom plates must be identical. Flange plates are welded to the column flange using CJP groove welds, and beam flange connections are made with high-strength bolts. The beam web is connected to the column flange using a bolted single-plate shear connection with bolts in short-slotted holes. Details for this connection type are shown in Figure 7.1. Initial yielding and plastic hinge formation are intended to occur in the beam in the region near the end of the flange plates.

7.2. SYSTEMS

Bolted flange plate connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limitations of these provisions.

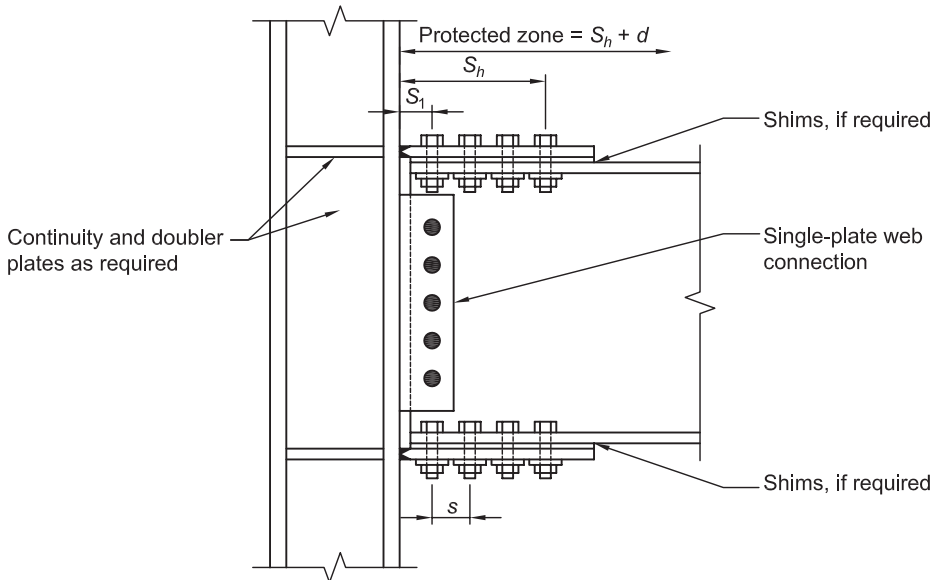


Fig. 7.1. Bolted flange plate moment connection.

Exception: Bolted flange plate connections in SMF systems with concrete structural slabs are only prequalified if the concrete slab is isolated in accordance with Section 2.3.4.

7.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements in Section 2.3.
- (2) Beam depth shall be limited to a maximum of W36 (W920) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight shall be limited to a maximum of 150 lb/ft (220 kg/m).
- (4) Beam flange thickness shall be limited to a maximum of 1 in. (25 mm).
- (5) The clear span-to-depth ratio of the beam shall be limited as follows:
 - (a) For SMF systems, 9 or greater.
 - (b) For IMF systems, 7 or greater.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of beams shall be provided as follows:

Lateral bracing of beams shall conform to the requirements of the AISC *Seismic Provisions*. To satisfy the requirements of AISC *Seismic Provisions* Chapter E for lateral bracing at plastic hinges, supplemental lateral bracing shall be provided at both the top and bottom beam flanges, and shall be located a distance of d to $1.5d$ from the bolt farthest from the face of the column. No attachment of lateral bracing shall be made within the protected zone.

Exception: For both SMF and IMF systems, where the beam supports a concrete structural slab that is connected along the beam span between protected zones with welded shear connectors spaced at a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at plastic hinges is not required.

- (8) The protected zone consists of the flange plates and the portion of the beam between the face of the column and a distance equal to one beam depth, d , beyond the bolt farthest from the face of the column.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled wide-flange shapes or built-up shapes permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.

- (3) Rolled shape column depth shall be limited to W36 (W920) maximum when a concrete structural slab is provided. In the absence of a concrete structural slab, the rolled shape column depth is limited to W14 (W360) maximum. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width or depth exceeding 24 in. (600 mm). Boxed wide-flange columns shall not have a width or depth exceeding 24 in. (600 mm), if participating in orthogonal moment frames.
- (4) There is no limit on weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Width-to-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

7.4. CONNECTION DETAILING

1. Plate Material Specifications

All connection plates shall conform to one of the following specifications: ASTM A36/A36M or A572/A572M Grade 50 (Grade 345).

2. Beam Flange Plate Welds

Flange plates shall be connected to the column flange using CJP groove welds and shall be considered demand critical. Backing, if used, shall be removed. The root pass shall be backgouged to sound weld metal and backwelded.

3. Single-Plate Shear Connection Welds

The single-plate shear connection shall be welded to the column flange. The single-plate to column-flange connection shall consist of CJP groove welds, two-sided partial-joint-penetration (PJP) groove welds, or two-sided fillet welds.

4. Bolt Requirements

Bolts shall be arranged symmetrically about the axes of the beam and shall be limited to two bolts per row in the flange plate connections. The length of the bolt group shall not exceed the depth of the beam. Standard holes shall be used in beam flanges. Holes in flange plates shall be standard or oversized holes. Bolt holes in beam flanges and in flange plates shall be made by drilling or by sub-punching and reaming.

User Note: Although standard holes are permitted in the flange plate, their use will likely result in field modifications to accommodate erection tolerances.

Bolts in the flange plates shall be ASTM F3125/F3125M Grades A490, A490M, or F2280 assemblies. Threads shall be excluded from the shear plane. The bolt diameter is limited to 1½ in. (28 mm) maximum.

5. Flange Plate Shims

Shims with a maximum overall thickness of ¼ in. (6 mm) may be used between the flange plate and beam flange as shown in Figure 7.1.

7.5. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3, where $Z_e = Z_b =$ plastic section modulus of the beam.

Step 2. Check beam flange tensile rupture. To preclude a net section rupture of the beam flange, the net section of the beam section shall satisfy the following:

$$R_t F_{ub} Z_{b,net} \geq R_y F_{yb} Z_b \quad (7.5-1)$$

where

- F_{yb} = specified minimum yield stress of beam, ksi (MPa)
- F_{ub} = specified minimum tensile strength of beam, ksi (MPa)
- R_t = ratio of expected tensile strength to specified minimum tensile strength
- R_y = ratio of expected yield stress to specified minimum yield stress
- Z_b = plastic section modulus of the beam, in.³ (mm³)
- $Z_{b,net}$ = plastic section modulus of net section of the beam at location of the plastic hinge, in.³ (mm³)

User Note: $Z_{b,net}$ of the beam may be computed by accounting for only the holes in the tension flange, or more simply, Z_b of the beam may be computed by accounting for the holes in both flanges. Note that if the former approach is employed, the plastic neutral axis will not be at the mid-depth of the beam. If the latter approach is employed, the calculations are simpler, and the requirement of Equation 7.5-1 may be met with a maximum bolt-hole diameter that is determined by

$$d_{hole} \leq \left[\frac{Z_b}{2t_f(d-t_f)} \right] \left(1 - \frac{R_y F_y}{R_t F_u} \right) - 1/16 \text{ in.} \quad (7.5-2)$$

$$d_{hole} \leq \left[\frac{Z_b}{2t_f(d-t_f)} \right] \left(1 - \frac{R_y F_y}{R_t F_u} \right) - 2 \text{ mm} \quad (7.5-2M)$$

where

- d = depth of beam, in. (mm)
- d_{hole} = nominal hole diameter for bolts between flange plate and beam flange as specified in AISC *Specification* Table J3.3 or Table J3.3M, in. (mm)
- t_f = thickness of the beam flange, in. (mm)

After the maximum bolt hole diameter is determined, the maximum bolt diameter can be determined from AISC *Specification* Table J3.3 or Table J3.3M.

Step 3. Assume a flange plate thickness, t_p . Estimate the width of the flange plate, b_{fp} , considering bolt gage, bolt edge distance requirements, and the beam flange width. Determine the controlling nominal shear strength per bolt considering bolt shear and bolt bearing:

$$r_n = \min \begin{cases} 1.0F_{nv}A_b \\ 2.4F_{ub}d_b t_f \\ 2.4F_{up}d_b t_p \end{cases} \quad (7.5-3)$$

where

A_b = nominal unthreaded body area of bolt, in.² (mm²)

F_{nv} = nominal shear stress of bolt from the AISC *Specification*, ksi (MPa)

F_{ub} = specified minimum tensile strength of beam material, ksi (MPa)

F_{up} = specified minimum tensile strength of plate material, ksi (MPa)

d_b = nominal diameter of bolt, in. (mm)

t_f = thickness of beam flange, in. (mm)

t_p = thickness of flange plate, in. (mm)

Step 4. Select a trial number of bolts.

User Note: The following equation may be used to estimate n , the trial number of bolts rounded to the next higher even number increment:

$$n \geq \frac{1.25M_{pr}}{\phi_n r_n (d + t_p)} \quad (7.5-4)$$

where

d = depth of beam, in. (mm)

Step 5. Determine the distance of the plastic hinge from the face of the column, S_h .

$$S_h = S_1 + s \left(\frac{n}{2} - 1 \right) \quad (7.5-5)$$

where

S_1 = distance from face of column to nearest row of bolts, in. (mm)

n = number of bolts

s = spacing of bolt rows, in. (mm)

The bolt spacing between rows, s , and the edge distance shall be sufficiently large to ensure that l_c , as defined in the AISC *Specification*, is greater than or equal to $2d_b$.

Step 6. Compute the shear force at the plastic hinge, V_h , in accordance with Section 2.4.4.

Step 7. Calculate the probable maximum moment at the face of the column, M_f , in accordance with Section 2.4.5.

Step 8. Compute F_{pr} , the force in the flange plate due to M_f .

$$F_{pr} = \frac{M_f}{(d + t_p)} \quad (7.5-6)$$

Step 9. Confirm that the number of bolts, n , selected in Step 4 is adequate:

$$n \geq \frac{F_{pr}}{\phi_n r_n} \quad (7.5-7)$$

Step 10. Check that the thickness of the flange plate assumed in Step 3 is adequate:

$$t_p \geq \frac{F_{pr}}{\phi_d F_y b_{fp}} \quad (7.5-8)$$

where

F_y = specified minimum yield stress of flange plate, ksi (MPa)

b_{fp} = width of flange plate, in. (mm)

Step 11. Check the flange plate for the limit state of tensile rupture:

$$F_{pr} \leq \phi_n R_n \quad (7.5-9)$$

where R_n is defined in the tensile rupture provisions of AISC *Specification* Section J4.1.

Step 12. Check the beam flange and the flange plate for the limit state of block shear rupture:

$$F_{pr} \leq \phi_n R_n \quad (7.5-10)$$

where R_n is as defined in the block shear rupture provision in AISC *Specification* Section J4.3.

Step 13. Check the flange plate for the limit state of buckling.

$$F_{pr} \leq \phi_n R_n \quad (7.5-11)$$

where R_n is defined in AISC *Specification* Section J4.4.

User Note: When checking compression buckling of the flange plate, the effective length, KL , may be taken as $0.65S_1$.

User Note: Some iteration from Steps 3 through 13 may be required to determine an acceptable flange plate size.

Step 14. Check the design shear strength of beam in accordance with the AISC *Specification*.

The required shear strength of the beam shall be determined in accordance with Section 2.5.

Step 15. Design the beam web-to-column connection.

The beam web-to-column connection shall be a single-plate shear connection meeting the requirements of the AISC *Specification*.

The required shear strength of the single-plate connection shall be taken equal to the required shear strength of the beam, and shall be assumed to be located at the face of the column.

Step 16. Check the continuity plate requirements in accordance with Section 2.6.

Step 17. Check the panel zone in accordance with Section 2.7.

When computing the required panel zone shear strength, the depth of the panel zone shall be taken as the depth of the beam plus the thickness of one flange plate.

Step 18. Check the column-beam moment ratio in accordance with Section 2.8.

CHAPTER 8

WELDED UNREINFORCED FLANGE-WELDED WEB (WUF-W) MOMENT CONNECTION

8.1. GENERAL

In the welded unreinforced flange-welded web (WUF-W) moment connection, inelastic rotation is developed primarily by yielding of the beam in the region adjacent to the face of the column. Connection rupture is controlled through special detailing requirements associated with the welds joining the beam flanges to the column flange, the welds joining the beam web to the column flange, and the shape and finish of the weld access holes. An overall view of the connection is shown in Figure 8.1.

8.2. SYSTEMS

WUF-W moment connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions. The WUF-W connection is exempt from the requirements of Section 2.3.4.

8.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

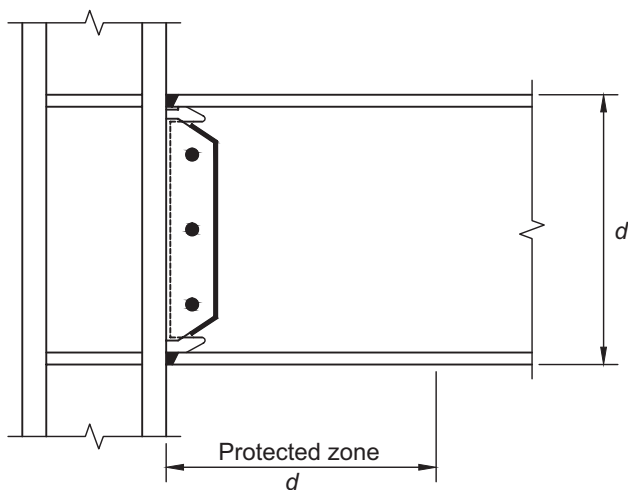


Fig. 8.1. WUF-W moment connection.

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.
- (2) Beam depth is limited to a maximum of W36 (W920) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight is limited to a maximum of 150 lb/ft (220 kg/m).
- (4) Beam flange thickness is limited to a maximum of 1 in. (25 mm).
- (5) The clear span-to-depth ratio of the beam is limited as follows:
 - (a) For SMF systems, 7 or greater.
 - (b) For IMF systems, 5 or greater.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of beams shall be provided as follows:

Lateral bracing of beams shall conform to the requirements of the AISC *Seismic Provisions*. To satisfy the requirements of the AISC *Seismic Provisions* for lateral bracing at plastic hinges, supplemental lateral bracing shall be provided at both the top and bottom beam flanges and shall be located at a distance of d to $1.5d$ from the face of the column, where d is the depth of the beam. No attachment of lateral bracing shall be made to the beam in the region extending from the face of the column to a distance d from the face of the column.

Exception: For both SMF and IMF systems, where the beam supports a concrete structural slab that is connected along the beam span between protected zones with welded shear connectors spaced at a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at plastic hinges is not required.

- (8) The protected zone consists of the portion of beam between the face of the column and a distance one beam depth, d , from the face of the column.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled wide-flange shapes or built-up shapes permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) Rolled shape column depth shall be limited to a maximum of W36 (W920). The depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width or depth exceeding 24 in. (600 mm). Boxed wide-flange columns shall not have a width or depth exceeding 24 in. (600 mm) if participating in orthogonal moment frames.

- (4) There is no limit on the weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Width-to-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

8.4. BEAM FLANGE-TO-COLUMN FLANGE WELDS

Beam flange-to-column flange connections shall satisfy the following limitations:

- (1) Beam flanges shall be connected to column flanges using CJP groove welds. Beam flange welds shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions*.
- (2) Weld access hole geometry shall conform to the requirements of AWS D1.8/D1.8M, clause 6.11.1.2. Weld access hole quality requirements shall conform to the requirements of AWS D1.8/D1.8M.

8.5. BEAM WEB-TO-COLUMN CONNECTION LIMITATIONS

The overall details of the beam web-to-column flange connection are shown in Figure 8.2. Single-plate shear connections shall conform to the requirements shown in Figure 8.2. Beam web-to-column flange connections shall satisfy the following limitations:

- (1) A single-plate shear connection shall be provided with a thickness equal to at least that of the beam web. The height of the single plate shall allow a $\frac{1}{4}$ -in.- (6-mm) minimum and $\frac{1}{2}$ -in.- (13-mm) maximum overlap with the weld access hole at the top and bottom as shown in Figure 8.3. The width shall extend 2 in. (50 mm) minimum beyond the end of the weld access hole.
- (2) The single-plate shear connection shall be welded to the column flange. The design shear strength of the welds shall be at least $0.6R_yF_{yp}h_p t_p$, where h_p is defined as the length of the plate, as shown in Figure 8.2, and t_p is the thickness of the plate.
- (3) The single-plate shear connection shall be connected to the beam web with fillet welds, as shown in Figures 8.2 and 8.3. The size of the fillet weld shall equal the thickness of the single plate minus $\frac{1}{16}$ in. (2 mm). The fillet welds shall extend along the sloped top and bottom portions of the single plate and along the vertical single plate length, as shown in Figures 8.2 and 8.3. The fillet welds on the sloped top and bottom portions of the single plate shall be terminated at least $\frac{1}{2}$ in. (13 mm) but not more than 1 in. (25 mm) from the edge of the weld access hole, as shown in Figure 8.3.
- (4) Erection bolts in standard holes or horizontal short slots are permitted as needed.
- (5) A CJP groove weld shall be provided between the beam web and the column flange. This weld shall be provided over the full length of the web between

weld access holes and shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions* and AWS D1.8/D1.8M. Weld tabs are not required. Weld tabs, if used, must be removed after welding in accordance with the requirements of Section 3.4. When weld tabs are not used, the use of cascaded weld ends within the weld groove shall be permitted at a maximum angle of 45° . Nondestructive testing (NDT) of cascaded weld ends need not be performed.

8.6. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3, where $Z_e = Z_b =$ plastic section modulus of the beam. The value of C_{pr} shall be taken equal to 1.4.

User Note: The C_{pr} value of 1.4 for WUF-W moment connections is based on experimental data that shows a high degree of strain hardening.

Step 2. The plastic hinge location shall be taken to be at the face of the column; that is, $S_h = 0$.

Step 3. Compute the shear force at the plastic hinge, V_h , in accordance with Section 2.4.4.

Step 4. Compute the probable maximum moment at the face of the column, M_f , in accordance with Section 2.4.5.

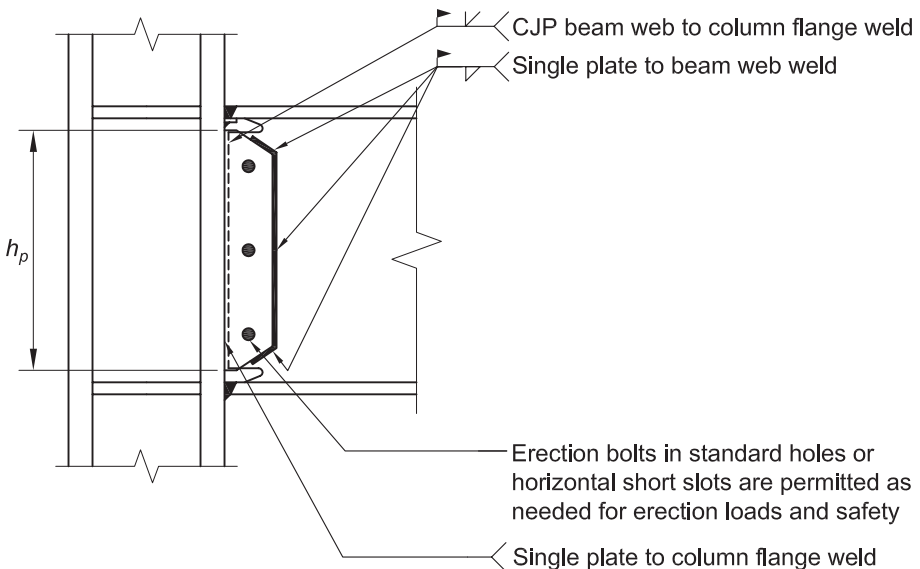


Fig. 8.2. General details of beam web-to-column flange connection.

User Note: $M_f = M_{pr}$ because $S_h = 0$. M_f is needed to check continuity plate requirements (Step 6), and panel zone requirements (Step 7).

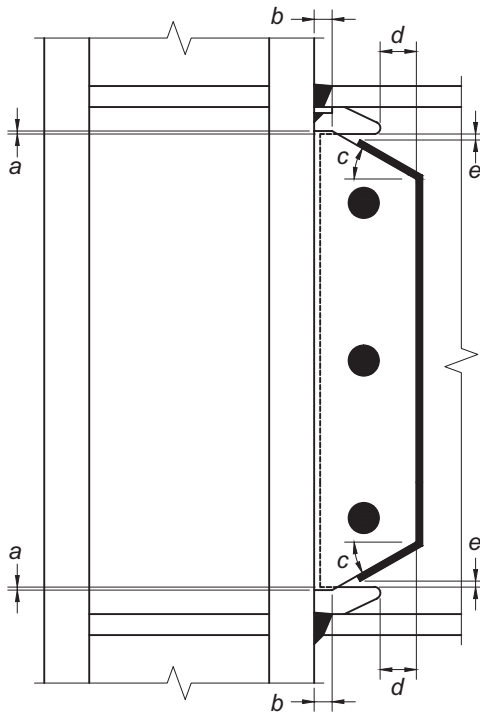
Step 5. Check design shear strength of the beam according to the AISC *Specification*.

The required shear strength of the beam shall be determined in accordance with Section 2.5.

Step 6. Check continuity plate requirements in accordance with Section 2.6.

Step 7. Check the panel zone in accordance with Section 2.7.

Step 8. Check the column-beam moment ratio in accordance with Section 2.8.



Notes

- $a = \frac{1}{4}$ in. (6 mm) minimum, $\frac{1}{2}$ in. (13 mm) maximum
- $b = 1$ in. (25 mm) minimum
- $c = 30^\circ (\pm 10^\circ)$
- $d = 2$ in. (50 mm) minimum
- $e = \frac{1}{2}$ in. (13 mm) minimum distance, 1 in. (25 mm) maximum distance from end of fillet weld to edge of access hole

Fig. 8.3. Details at top and bottom of single-plate shear connection.

CHAPTER 9

CAST BOLTED BRACKET (CBB) MOMENT CONNECTION

9.1. GENERAL

In a cast bolted bracket (CBB) moment connection, a cast high-strength steel bracket is fastened to each beam flange and bolted to the column flange as shown in Figure 9.1. The bracket attachment to the beam flange is permitted to be either welded [Figure 9.1(a)] or bolted [Figure 9.1(b)]. When welded to the beam flange, the five W-series bracket configurations permitted are shown in Figure 9.2. When bolted to the beam flange, the two B-series bracket configurations permitted are shown in Figure 9.3. The bracket configuration is proportioned to develop the probable maximum moment of the connected beam. Yielding and plastic hinge formation are intended to occur primarily in the beam at the end of the bracket away from the column face.

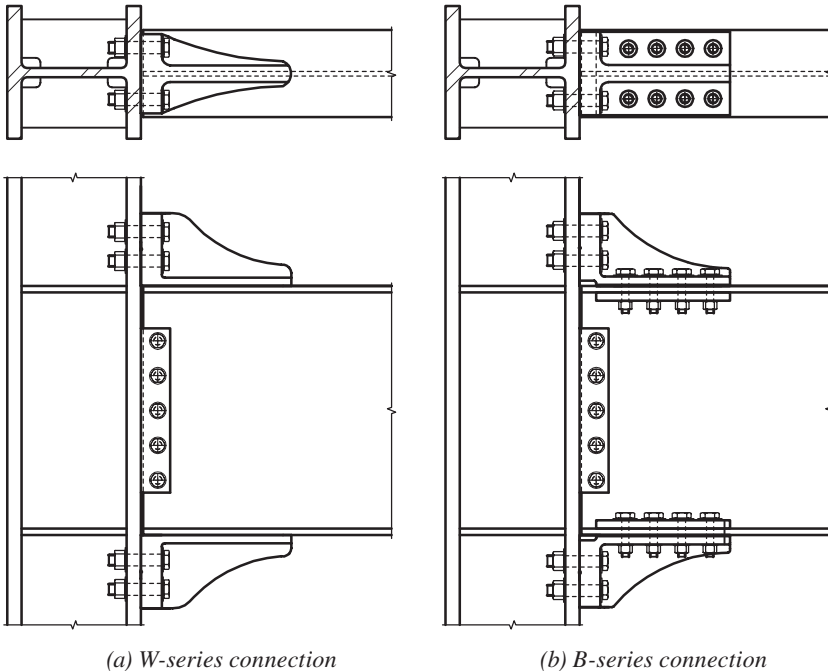


Fig. 9.1. Cast bolted bracket connection.

9.2. SYSTEMS

CBB connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions.

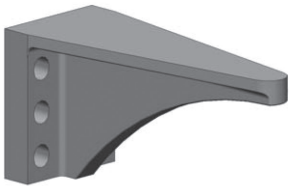
Exception: CBB connections in SMF systems with concrete structural slabs are only prequalified if the concrete slab is isolated in accordance with Section 2.3.4.

9.3. PREQUALIFICATION LIMITS

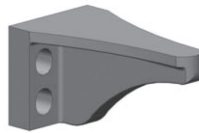
1. Beam Limitations

Beams shall satisfy the following limitations:

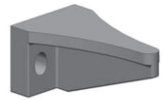
- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.
- (2) Beam depth is limited to a maximum of W33 (W840) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight is limited to a maximum of 130 lb/ft (190 kg/m).
- (4) Beam flange thickness is limited to a maximum of 1 in. (25 mm).
- (5) Beam flange width shall be at least 6 in. (150 mm) for W-series brackets and at least 10 in. (250 mm) for B-series brackets.



(a) Six column bolts, W1.0

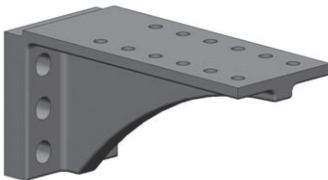


(b) Four column bolts,
W2.0 and W2.1

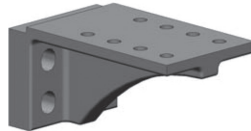


(c) Two column bolts,
W3.0 and W3.1

Fig. 9.2. Cast bolted bracket W-series configurations.



(a) Six column bolts, B1.0



(b) Four column bolts, B2.1

Fig. 9.3. Cast bolted bracket B-series configurations.

- (6) The clear span-to-depth ratio of the beam shall be limited to 9 or greater for both SMF and IMF systems.
- (7) Width-to-thickness ratios for the flanges and web of the beam shall conform to the requirements of the AISC *Seismic Provisions*.
- (8) Lateral bracing of beams shall be provided as follows:
 - (a) For SMF systems, in conformance with the AISC *Seismic Provisions*. Supplemental lateral bracing shall be provided at the expected plastic hinge in conformance with the AISC *Seismic Provisions*.

When supplemental lateral bracing is provided, attachment of supplemental lateral bracing to the beam shall be located at a distance d to $1.5d$ from the end of the bracket farthest from the face of the column, where d is the depth of the beam. No attachment of lateral bracing shall be made to the beam in the region extending from the face of the column to a distance d beyond the end of the bracket.
 - (b) For IMF systems, in conformance with the AISC *Seismic Provisions*.

Exception: For both systems, where the beam supports a concrete structural slab that is connected between the protected zones with welded shear connectors spaced at maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at the expected hinge is not required.
- (9) The protected zone consists of the portion of beam between the face of the column and one beam depth, d , beyond the end of the bracket farthest from the face of the column.

2. Column Limitations

The columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled wide-flange shapes or built-up shapes permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) The column flange width shall be at least 12 in. (300 mm).
- (4) Rolled shape column depth shall be limited to W36 (W920) maximum when a concrete structural slab is provided. In the absence of a concrete structural slab, rolled shape column depth is limited to W14 (W360) maximum. The depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width or depth exceeding 16 in. (400 mm). Boxed wide-flange columns shall not have a width or depth exceeding 16 in. (400 mm) if participating in orthogonal moment frames.
- (5) There is no limit on the weight per foot of columns.
- (6) There are no additional requirements for flange thickness.

- (7) Width-to-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (8) Lateral bracing of the columns shall conform to the requirements of the AISC *Seismic Provisions*.

3. Bracket Limitations

The high strength cast-steel brackets shall satisfy the following limitations:

- (1) Bracket castings shall conform to the requirements of Appendix A.
- (2) Bracket configuration and proportions shall conform to Section 9.6.
- (3) Holes in the bracket for the column bolts shall be vertical short-slotted holes. Holes for the beam bolts shall be standard holes.
- (4) Material thickness, edge distance, and end distance shall have a tolerance of $\pm 1/16$ in. (2 mm). Hole location shall have a tolerance of $\pm 1/16$ in. (2 mm). The overall dimensions of the bracket shall have a tolerance of $\pm 1/8$ in. (3 mm).

9.4. BRACKET-TO-COLUMN FLANGE CONNECTION LIMITATIONS

Bracket-to-column flange connections shall satisfy the following limitations:

- (1) Column flange fasteners shall be pretensioned ASTM F3125/F3125M Grade A490 or A490M, or ASTM A354 Grade BD bolts or Grade BD threaded rods, and shall conform to the requirements of Chapter 4.
- (2) Column flange bolt holes shall be $1/8$ in. (3 mm) larger than the nominal bolt diameter. Bolt holes shall be drilled or subpunched and reamed.
- (3) The use of finger shims on either or both sides at the top and/or bottom of the bracket connection is permitted, subject to the limitations of the RCSC *Specification*.
- (4) When bolted to a box column, a steel washer plate shall be inserted between the box column and the bracket on both faces of the column. The washer plate shall be ASTM A572/A572M Grade 50 (Grade 345) or better and shall be designed to transfer the bolt forces to the outside edges of the column. Where required, the vertical plate depth may extend beyond the contact surface area by up to 4 in. (100 mm). The plate thickness shall not exceed 3 in. (75 mm). The fasteners shall pass through the interior of the box column and be anchored on the opposite face. The opposite face shall also have a steel washer plate.
- (5) When connecting to the orthogonal face of a box column concurrent with a connection on the primary column face, a $1\frac{1}{4}$ in. (45 mm) steel spacer plate shall be inserted between the beam flanges and the brackets of the orthogonal connection. The spacer plate shall be made of any of the structural steel materials included in the AISC *Specification* and shall be the approximate width and length matching that of the bracket contact surface area.

9.5. BRACKET-TO-BEAM FLANGE CONNECTION LIMITATIONS

Bracket-to-beam flange connections shall satisfy the following limitations:

- (1) When welded to the beam flange, the bracket shall be connected using fillet welds. Bracket welds shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions* and AWS D1.8/D1.8M, and to the requirements of AWS D1.1/D1.1M. The weld procedure specification (WPS) for the fillet weld joining the bracket to the beam flange shall be qualified with the casting material. Welds shall not be started or stopped within 2 in. (50 mm) of the bracket tip and shall be continuous around the tip.
- (2) When bolted to the beam flange, fasteners shall be pretensioned ASTM F3125/F3125M Grade A490 or A490M bolts with threads excluded from the shear plane and shall conform to the requirements of Chapter 4.
- (3) Beam flange bolt holes shall be $1\frac{5}{32}$ in. (29 mm) and shall be drilled using the bracket as a template.
- (4) When bolted to the beam flange, a $\frac{1}{8}$ -in.- (3-mm-) thick brass washer plate with an approximate width and length matching that of the bracket contact surface area shall be placed between the beam flange and the bracket. The brass shall be a half-hard tempered ASTM B19 or ASTM B36/B36M sheet.
- (5) When bolted to the beam flange, a 1-in.- (25-mm-) thick by 4-in.- (100-mm-) wide ASTM A572/A572M Grade 50 (Grade 345) plate washer shall be used on the opposite side of the connected beam flange.

9.6. CONNECTION DETAILING

The connection detailing for the W-series bracket configurations is shown in Figure 9.4 when welded to the beam flange. Figure 9.5 shows the connection detailing for the B-series bracket configurations when bolted to the beam flange. Table 9.1 summarizes the CBB proportions and column bolt parameters. Table 9.2 summarizes the design proportions for the W-series bracket configuration. Table 9.3 summarizes the design proportions for the B-series bracket configurations.

9.7. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the location of the plastic hinge, M_{pr} , in accordance with Section 2.4.3, where $Z_e = Z_b =$ plastic section modulus of the beam.

Step 2. Select a trial bracket from Table 9.1.

Step 3. Determine the distance of the plastic hinge from the face of column, S_h .

$$S_h = L_{bb} \text{ per Table 9.1, in. (mm)}$$

Step 4. Compute the shear force at the plastic hinge, V_h , in accordance with Section 2.4.4.

Step 5. Compute the probable maximum moment at the face of the column, M_f in accordance with Section 2.4.5.

Step 6. Compute the required column bolt tensile strength, r_{ut} :

$$r_{ut} = \frac{M_f}{d_{eff} n_{cb}} \quad (9.7-1)$$

where

d_{eff} = effective beam depth, calculated as the centroidal distance between bolt groups in the upper and lower brackets, in. (mm)

n_{cb} = number of column bolts per Table 9.1

Step 7. Check the design tensile strength of the column bolts:

$$r_{ut} \leq \phi_n F_{nt} A_b \quad (9.7-2)$$

where

A_b = bolt nominal cross-sectional area, in.² (mm²)

F_{nt} = nominal tensile strength of bolt from the AISC *Specification*, ksi (MPa)

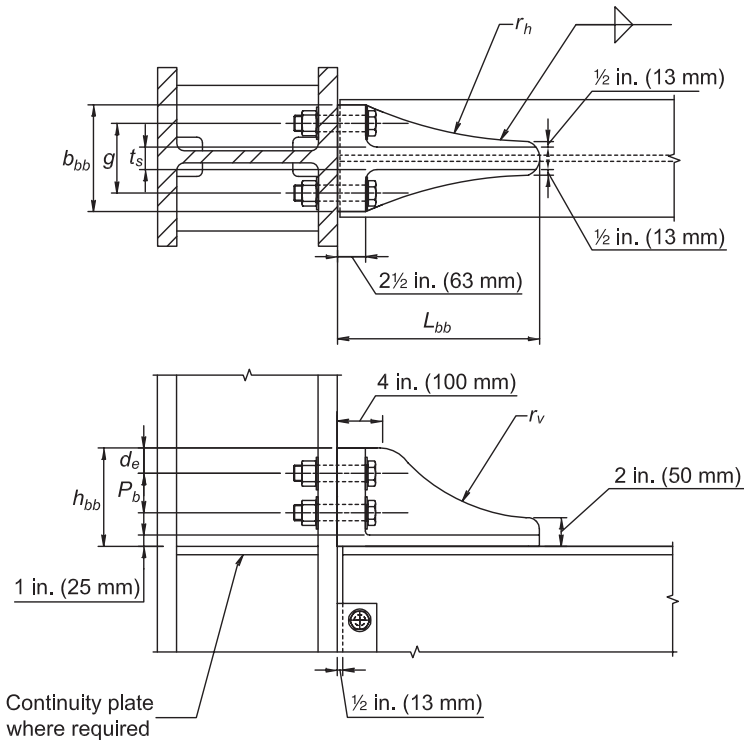


Fig. 9.4. W-series connection detailing.

TABLE 9.1
Cast Bolted Bracket Proportions

Bracket Designation	Bracket Length, L_{bb} in. (mm)	Bracket Height, h_{bb} in. (mm)	Bracket Width, b_{bb} in. (mm)	Number of Column Bolts, n_{cb}	Column Bolt Gage, g in. (mm)	Column Bolt Diameter, in. (mm)
W3.0	16 (400)	5½ (140)	9 (230)	2	5½ (140)	1⅝ (35)
W3.1	16 (400)	5½ (140)	9 (230)	2	5½ (140)	1½ (38)
W2.0	16 (400)	8¾ (220)	9½ (240)	4	6 (150)	1⅝ (35)
W2.1	18 (450)	8¾ (220)	9½ (240)	4	6½ (160)	1½ (38)
W1.0	25½ (640)	12 (300)	9½ (240)	6	6½ (160)	1½ (38)
B2.1	18 (450)	8¾ (220)	10 (250)	4	6½ (160)	1½ (38)
B1.0	25½ (640)	12 (300)	10 (250)	6	6½ (160)	1½ (38)

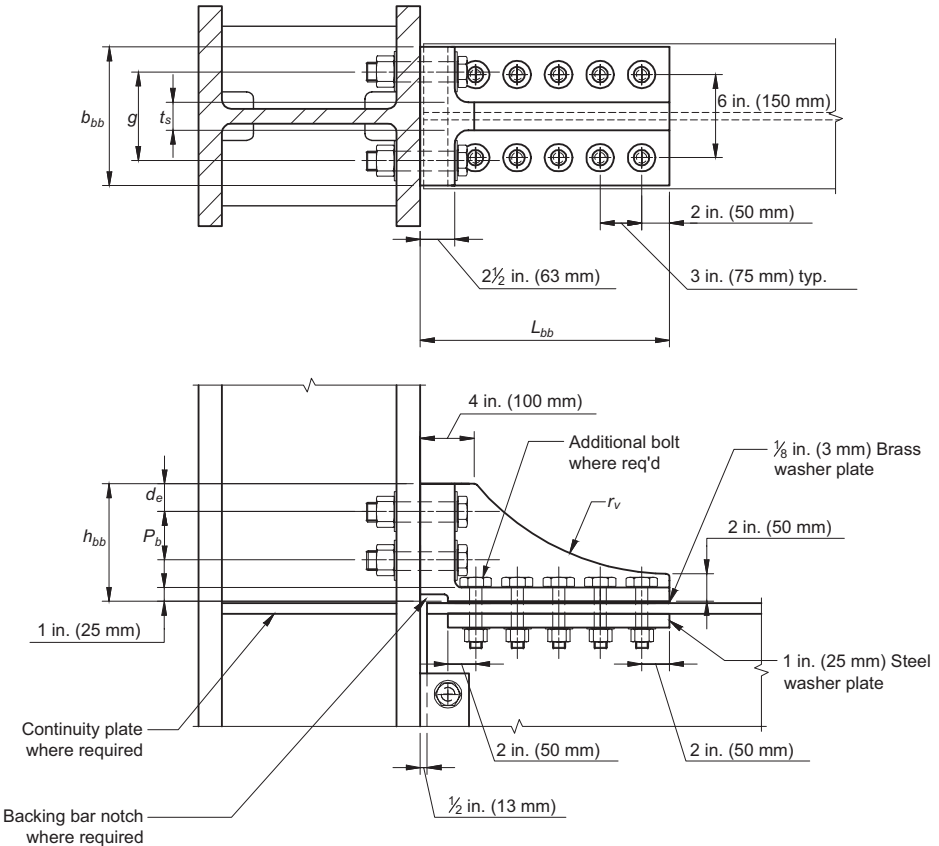


Fig. 9.5. B-series connection detailing.

TABLE 9.2
W-Series Bracket Design Proportions

Bracket Designation	Column Bolt Edge Distance, d_e in. (mm)	Column Bolt Pitch, p_b in. (mm)	Bracket Stiffener Thickness, t_s in. (mm)	Bracket Stiffener Radius, r_v in. (mm)	Bracket Horizontal Radius, r_h in. (mm)	Minimum Fillet Weld Size, w in. (mm)
W3.0	2½ (63)	n.a.	1 (25)	n.a.	28 (700)	½ (13)
W3.1	2½ (63)	n.a.	1 (25)	n.a.	28 (700)	⅝ (16)
W2.0	2¼ (56)	3½ (88)	2 (50)	12 (300)	28 (700)	¾ (19)
W2.1	2¼ (56)	3½ (88)	2 (50)	16 (400)	38 (950)	⅞ (22)
W1.0	2 (50)	3½ (88)	2 (50)	28 (700)	n.a.	⅞ (22)

TABLE 9.3
B-Series Bracket Design Proportions

Bracket Designation	Column Bolt Edge Distance, d_e in. (mm)	Column Bolt Pitch, p_b in. (mm)	Bracket Stiffener Thickness, t_s in. (mm)	Bracket Stiffener Radius, r_v in. (mm)	Number of Beam Bolts, n_{bb}	Beam Bolt Diameter, in. (mm)
B2.1	2 (50)	3½ (88)	2 (50)	16 (400)	8 or 10	1⅝ (28)
B1.0	2 (50)	3½ (88)	2 (50)	28 (700)	12	1⅝ (28)

Step 8. Determine the minimum column flange width to prevent flange tensile rupture:

$$b_{cf} \geq \frac{2(d_b + 1/8 \text{ in.})}{\left(1 - \frac{R_y F_{yf}}{R_t F_{uf}}\right)} \quad (9.7-3)$$

$$b_{cf} \geq \frac{2(d_b + 3 \text{ mm})}{\left(1 - \frac{R_y F_{yf}}{R_t F_{uf}}\right)} \quad (9.7-3M)$$

where

F_{uf} = specified minimum tensile strength of flange material, ksi (MPa)

F_{yf} = specified minimum yield stress of flange material, ksi (MPa)

R_t = ratio of expected tensile strength to specified minimum tensile strength for flange material

R_y = ratio of expected yield stress to specified minimum yield stress for flange material

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

d_b = diameter of column flange bolts, in. (mm)

Step 9. Check the column flange for flexural yielding:

$$t_{cf} \geq \sqrt{\frac{1.11M_f}{\phi_d F_{yf} d_{eff} Y_m}} \quad (9.7-4)$$

where

Y_m = simplified column flange yield line mechanism parameter

= 5.9 for W3.0 and W3.1

= 6.5 for W2.0, W2.1, and B2.1

= 7.5 for W1.0 and B1.0

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

User Note: The simplified column flange yield line mechanism parameters, Y_m , assume there are no continuity plates. If continuity plates are provided, the minimum column flange thickness from Equation 9.7-4 is conservative. Alternatively, when continuity plates are provided, new values of Y_m can be derived to justify a smaller column flange thickness.

The simplified column flange yield line mechanism parameters, Y_m , also assume that the distance from the top-most bolt line to the top of the column is at least equal to $0.5\sqrt{b_{cf}g}$ where b_{cf} = column flange width, in. (mm) and g = column bolt gage, in. (mm) as specified in Table 9.1.

Step 10. If the bracket is welded to the beam flange, proceed to Step 14; otherwise, check beam flange tensile rupture. To preclude a net section rupture of the beam flange, the net section of the beam section shall satisfy the following:

$$R_t F_{ub} Z_{b,net} \geq R_y F_{yb} Z_b \quad (9.7-5)$$

where

F_{ub} = specified minimum tensile strength of beam, ksi (MPa)

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

R_t = ratio of expected tensile strength to specified minimum tensile strength

R_y = ratio of expected yield stress to specified minimum yield stress

Z_b = plastic section modulus of beam, in.³ (mm³)

$Z_{b,net}$ = plastic section modulus of net section of the beam at location of the plastic hinge, in.³ (mm³)

User Note: $Z_{b,net}$ of the beam may be computed by accounting for only the holes in the tension flange, or more simply, $Z_{b,net}$ of the beam may be computed by accounting for the holes in both flanges. Note that if the former approach is employed, the plastic neutral axis will not be at the mid-depth of the beam. If the latter approach is employed, $Z_{b,net}$ can be computed as follows:

$$Z_{b,net} = Z_b - 2(1\frac{7}{32} \text{ in.})t_f(d - t_f) \quad (9.7-6)$$

$$Z_{b,net} = Z_b - 2(31 \text{ mm})t_f(d - t_f) \quad (9.7-6M)$$

where

d = depth of beam, in. (mm)

t_f = flange thickness of the beam, in. (mm)

Step 11. Compute the beam flange force, F_{uf} :

$$F_{uf} = \frac{M_f}{d} \quad (9.7-7)$$

Step 12. Check beam bolt strength:

$$F_{uf} \leq \sum \phi_n r_n \quad (9.7-8)$$

$$r_n = \min \left\{ \begin{array}{l} F_{nv} A_b \\ 2.4 F_{ub} d_b t_f \\ 1.2 l_c F_{ub} t_f \end{array} \right\} \text{ for each bolt in beam flange, kips (N)} \quad (9.7-9)$$

where

F_{nv} = nominal shear stress of bolt from the AISC *Specification*, ksi (MPa)

l_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of beam flange, in. (mm)

t_f = thickness of beam flange, in. (mm)

Step 13. Check the beam flange for block shear rupture:

$$F_{uf} \leq \phi_n R_n \quad (9.7-10)$$

where R_n is as defined in the block shear rupture provisions of AISC *Specification* Section J4.3.

Step 14. If the bracket is welded to the beam flange, determine the required size of the fillet weld.

$$F_{uf} \leq \phi_n F_w l_w (0.707w) \quad (9.7-11)$$

where

F_{EXX} = filler metal classification strength, ksi (MPa)

$F_w = 0.60F_{EXX}$

l_w = length of available fillet weld, in. (mm)

$= 2(L_{bb} - 2.5 \text{ in.} - l)$

(9.7-12)

$= 2(L_{bb} - 64 \text{ mm} - l)$

(9.7-12M)

where

L_{bb} = bracket length per Table 9.3, in. (mm)

l = bracket overlap distance, in. (mm)

$= 0 \text{ in. (0 mm)}$ if $b_{bf} \geq b_{bb}$

$= 5 \text{ in. (125 mm)}$ if $b_{bf} < b_{bb}$

w = fillet weld size (not less than minimum fillet weld size specified in Table 9.2), in. (mm)

Step 15. Check design shear strength of beam in accordance with the AISC *Specification*.

The required shear strength of the beam shall be determined in accordance with Section 2.5.

Step 16. Design the beam web-to-column connection.

The required shear strength of the beam web-to-column connection shall be taken equal to the required shear strength of the beam.

The beam web-to-column connection shall be a single-plate shear connection meeting the requirements of the AISC *Specification*.

The single-plate shear connection shall be connected to the column flange using a two-sided fillet weld, two-sided PJP groove weld, or CJP groove weld.

Step 17. Check the continuity plate requirements in accordance with Section 2.6 using the flange force computed in Step 11.

User Note: When checking column flange continuity plate requirements, only the limit states of web local yielding and web local crippling need be considered. The limit state of flange local bending need not be considered because satisfying Equation 9.7-4 precludes flange local bending failure.

Step 18. Check the column panel zone in accordance with Section 2.7.

When computing the required panel-zone shear strength, the depth of the panel zone shall be taken as d_{eff} , as defined in Step 6.

Step 19. Check the column-beam moment ratio in accordance with Section 2.8.

Step 20. If the column is a box configuration, determine the size of the steel washer plate between the column flange and the bracket such that

$$Z_{wp} \geq \frac{M_f(b_{cf} - t_{cw} - g)}{4\phi_d F_y d_{eff}} \quad (9.7-13)$$

where

F_y = specified minimum yield stress of washer material, ksi (MPa)

Z_{wp} = plastic section modulus of washer plate, in.³ (mm³)

g = bolt gage of column, in. (mm)

t_{cw} = thickness of column web, in. (mm)

CHAPTER 10

CONXTECH CONXL MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by patent rights. By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license, and the statement may be obtained from the standards developer.*

10.1. GENERAL

The ConXtech® ConXL™ moment connection permits full-strength, fully restrained connection of wide-flange beams to concrete-filled 16 in. (400 mm) square hollow structural section (HSS) or built-up box columns using a high-strength, field-bolted collar assembly. Beams are shop welded to forged flange and web fittings (collar flange assembly) and are field-bolted together through either forged or cast steel column fittings (collar corner assembly) that are shop-welded to the columns. Beams may include reduced beam section (RBS) cutouts if necessary to meet strong-column/weak-beam criteria. ConXL connections may be used to provide moment connections to columns in orthogonal frames. All moment beams connecting to a ConXL node (intersection of moment beams and column) must be of the same nominal depth.

Figure 10.1 shows the connection geometry and major connection components. Each ConXL collar assembly is made up of either forged or cast collar corners and forged collar flanges. At each ConXL node there are four collar corner assemblies (Figure 10.2)—one at each corner of the square built-up or HSS column. Each ConXL node also contains four collar flange assemblies (Figure 10.3)—one for each face of the square column. Each collar flange assembly can contain the end of a moment beam that is shop welded to the collar flange assembly. The combination of collar corner assemblies, collar flange assemblies, and a square concrete-filled column create the ConXL node.

Figure 10.2 shows the collar corner assemblies. The collar corner assembly is made up of a collar corner top (CCT) piece; a collar corner bottom (CCB) piece; and for beam depths greater than 18 in. (450 mm), a collar corner middle (CCM) piece. The CCT, CCB, and CCM are partial-joint-penetration (PJP) groove welded together to create the collar corner assembly; they are then shop fillet-welded to the corners of the square column.

*The connectors and structures illustrated are covered by one or more of the following U.S. and foreign patents: U.S. Pat. Nos. 7,941,985; 7,051,917; Australia Pat. No. 2004319371; Canada Pat. No. 2,564,195; China Pat. No. ZL 2004 8 0042862.5; Japan Pat. No. 4427080; Mexico Pat. Nos. 275284; Hong Kong Pat. No. 1102268. Other U.S. and foreign patent protection pending.

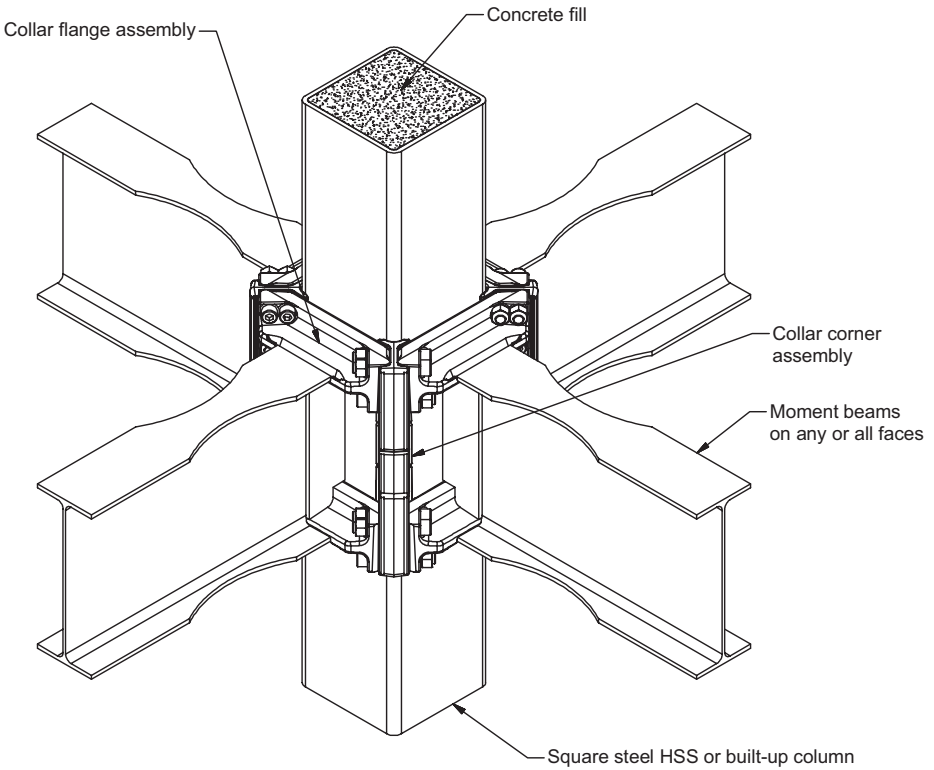


Fig. 10.1. Assembled ConXL moment connection.

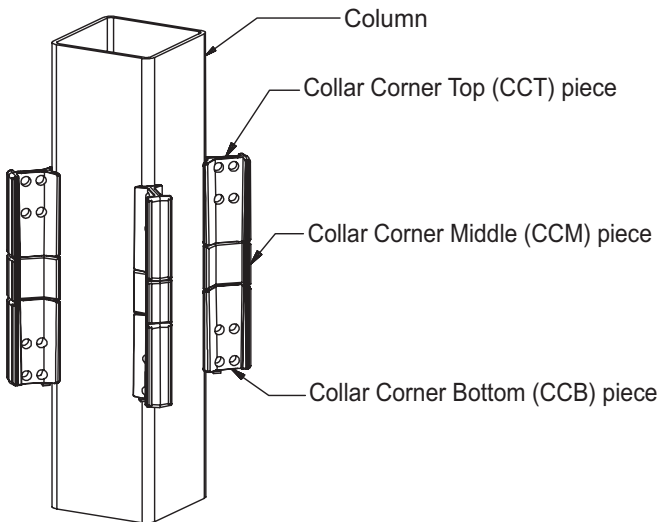


Fig. 10.2. Column with attached collar corner assemblies.

Figure 10.3 shows the collar flange assembly. Each collar flange assembly is made up of a collar flange top (CFT), collar flange bottom (CFB), and a collar web extension (CWX).

If a beam at the node requires a moment connection, the CFT (or CFB) is aligned with and shop welded to the top (or bottom) flange of the beam.

Moment-connected beam webs are also shop welded to the CWX. If a beam at the node does not require a moment connection, the size of the CWX remains unchanged and a single-plate shear connection is shop-welded to the CWX to accommodate a non-moment-connected beam that does not need to match the nominal depth of the moment-connected beam(s).

If no beams exist on a node at a particular column face, the CFT and CFB are aligned at the nominal depth of the moment beam and the CWX shall be permitted to be optionally omitted.

Section 10.8 contains drawings indicating the dimensions of individual pieces.

Columns are delivered to the job site with the collar corner assemblies shop welded to the column at the proper floor framing locations. Beams are delivered to the job site with the collar flange assemblies shop welded to the ends of the beams. During frame erection the collar flange assemblies with or without beams are lowered into the column collar corner assemblies. When all four faces of the column are filled with collar flanges, the collar bolts are inserted and pretensioned, effectively clamping and

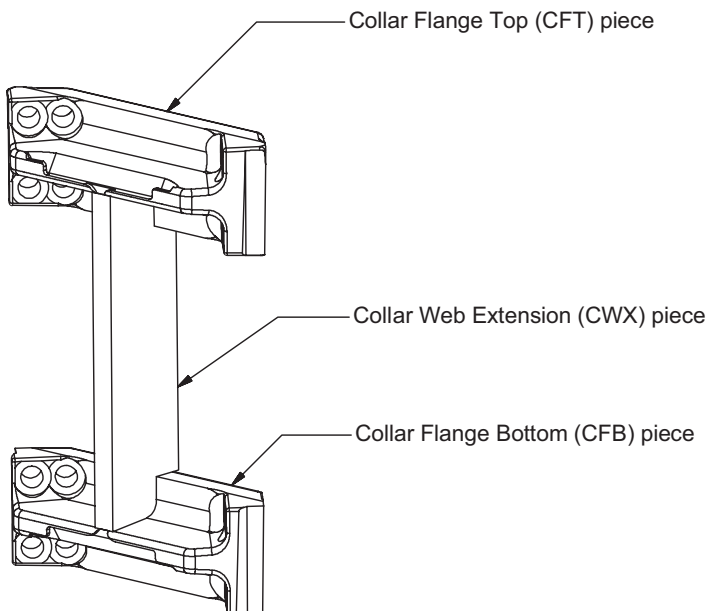


Fig. 10.3. Collar flange assembly.

compressing the collar flange assemblies around the collar corner assemblies and square column.

Beam flange flexural forces in moment beams are transferred to the collar flange assemblies via CJP groove welds. Collar flanges transfer compressive beam flange forces to the collar corners through flexure of the collar flange and direct bearing onto the collar corners. The collar flange transfers beam flange tensile forces in flexure to the pretensioned collar bolts. The collar bolts transfer these forces in tension through the orthogonal collar flanges, which then transfer the forces through the rear collar bolts attached to the collar flange on the opposite face of the column. These combined forces are then transferred to the column walls through a combination of bearing and the fillet welds attaching the collar corners to the column. Finally, a portion of these forces are transferred to the concrete fill, which is in direct contact with the column walls.

The behavior of this connection is controlled by flexural hinging of the beams adjacent to the collar assembly. When RBS cutouts are provided, yielding and plastic hinge formation primarily occur within the reduced beam section.

10.2. SYSTEMS

The ConXL moment connection is prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions. The ConXL moment connection is prequalified for use in planar moment-resisting frames or in orthogonal intersecting moment-resisting frames.

ConXL connections in SMF systems with concrete structural slabs are only prequalified if the concrete slab is isolated in accordance with Section 2.3.4. This requirement is satisfied if a vertical flexible joint at least 1 in. (25 mm) thick is placed in the concrete slab around the collar assembly and column, similar to that shown in Figure 10.4.

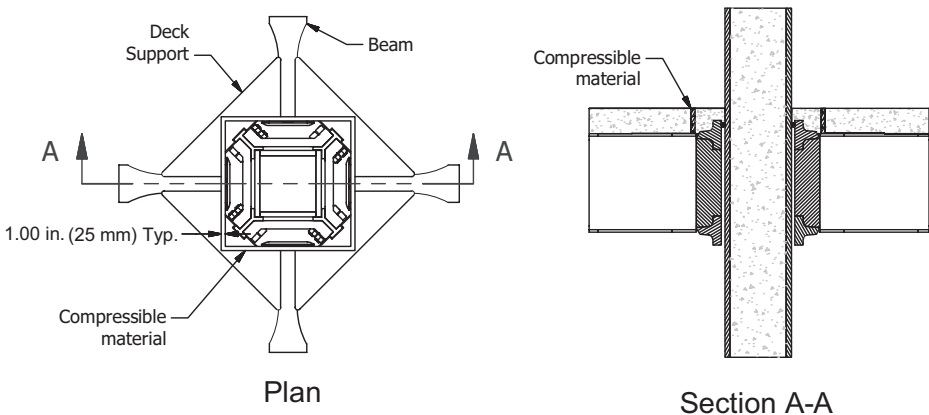


Fig.10.4. Use of compressible material to isolate structural slab from connection.

10.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.
- (2) Beam depths shall be limited to the following beam shapes or their built-up equivalents: W30 (W760), W27 (W690), W24 (W610), W21 (W530), and W18 (W460).
- (3) Beam flange thickness shall be limited to a maximum of 1 in. (25 mm).
- (4) Beam flange width shall be limited to a maximum of 12 in. (300 mm).
- (5) The clear span-to-depth ratio of the beam shall be limited as follows:
 - (a) For SMF systems, 7 or greater.
 - (b) For IMF systems, 5 or greater.
- (6) Width-to-thickness ratios for beam flanges and webs shall conform to the limits of the AISC *Seismic Provisions*. The value of b_f used to determine the width-to-thickness ratio of beams with RBS cutouts shall not be less than the flange width at the center two-thirds of the reduced section, provided that gravity loads do not shift the location of the plastic hinge a significant distance from the center of the reduced beam section.
- (7) Lateral bracing of beams shall conform to the applicable limits of the AISC *Seismic Provisions*.

Exception: For SMF and IMF systems, where the beam supports a concrete structural slab that is connected between the protected zones with welded shear connectors spaced at a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at the expected hinge is not required.

- (8) For RBS connections, the protected zone consists of the portion of the connection assembly and beam between the column face and the farthest end of the reduced beam section. For beams without reduced beam sections, the protected zone consists of the portion of the connection assembly and beam extending from the column face to a distance d from the outside face of the collar flange, where d is the depth of the beam.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be square 16 in. (400 mm) HSS or square 16 in. (400 mm) built-up box sections permitted in Section 2.3.
- (2) There is no limit on column weight per foot.
- (3) Column wall thickness shall not be less than $\frac{3}{8}$ in. (10 mm). Column wall thickness for HSS columns shall not be less than $\frac{3}{8}$ in. (10 mm) nominal.

- (4) Width-to-thickness ratios for columns shall conform to the applicable limits for filled composite columns in the AISC *Seismic Provisions*.
- (5) Lateral bracing of columns shall conform to the applicable limits in the AISC *Seismic Provisions*.
- (6) Columns shall be completely filled with structural concrete having unit weight not less than 110 lb/ft³ (1 800 kg/m³). Concrete shall have 28-day compressive strength not less than 3,000 psi (21 MPa).
- (7) Flanges and webs of built-up box columns shall be connected using PJP groove welds with a groove weld size not less than $\frac{3}{4}$ of the thickness of the connected plates as shown in Figure 10.5.

3. Collar Limitations

Collars shall satisfy the following limitations:

- (1) Collar forgings shall conform to the requirements of Appendix B, Forging Requirements. Forged parts shall conform to the material requirements of ASTM A572/A572M Grade 50 (Grade 345).
- (2) Cast collar parts shall conform to the requirements of Appendix A, Casting Requirements. Cast parts shall conform to the requirements of ASTM A958/A958M Grade SC8620, class 80/50.
- (3) Collar configuration and proportions shall conform to Section 10.8.
- (4) Collar flange bolt holes shall be $\frac{1}{8}$ in. (3 mm) larger than the nominal bolt diameter. Bolt holes shall be drilled.
- (5) Collar corner bolt holes shall be $\frac{1}{8}$ in. (3 mm) larger than the nominal bolt diameter. Bolt holes shall be drilled.
- (6) Material thickness, edge distance, end distance, and overall dimension shall have a tolerance of $\pm\frac{1}{16}$ in. (2 mm).
- (7) Faying surfaces shall be machined and meet the requirements for Class A slip-critical surfaces as defined in the AISC *Specification*.

10.4. COLLAR CONNECTION LIMITATIONS

Collar connections shall satisfy the following limitations:

- (1) Collar bolts shall be pretensioned 1¼-in.- (32-mm-) diameter high-strength bolts conforming to ASTM A574 with threads excluded from the shear plane and shall conform to the requirements of Sections 4.2 and 4.3.
- (2) The collar bolts shall be pretensioned to the requirements for ASTM F3125/F3125M Grade A490 or A490M bolts in the AISC *Specification*.
- (3) Welding of CCT, CCM, and CCB pieces to form collar corner assemblies shall consist of PJP groove welds per Figure 10.6.
- (4) Welding of collar corner assemblies to columns shall consist of flare bevel groove welds with $\frac{3}{8}$ in. (10 mm) fillet reinforcing per Figure 10.7.

- (5) Collar flanges shall be welded to CWX pieces with $\frac{5}{16}$ in. (8 mm) fillet welds, each side per Figure 10.8.
- (6) Beams shall be welded to collar flange assemblies with CJP groove welds per Figure 10.9.

10.5. BEAM WEB-TO-COLLAR CONNECTION LIMITATIONS

Beam web-to-collar connections shall satisfy the following limitations:

- (1) The required shear strength of the beam web connection shall be determined according to Section 10.7.
- (2) The beam web is welded to the collar web extension (CWX) with a two-sided fillet weld as shown in Figure 10.9. The fillet welds shall be sized to develop the required shear strength of the connection.

10.6. BEAM FLANGE-TO-COLLAR FLANGE WELDING LIMITATIONS

Welding of the beam to the collar flange shall conform to the following limitations:

- (1) Weld access holes are not permitted. Welding access to top and bottom flanges shall be made available by rotating the beam to allow a CJP weld in the flat position (position 1G per AWS D1.1/D1.1M).
- (2) The beam flange-to-collar flange weld shall be made with a CJP groove weld within the weld prep area of the collar flange. Reinforcing $\frac{5}{16}$ in. (8 mm) fillet welds shall be placed on the back side of the CJP groove welds. The CJP flange weld shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions* and AWS D1.8/D1.8M, and to the requirements of AWS D1.1/D1.1M.

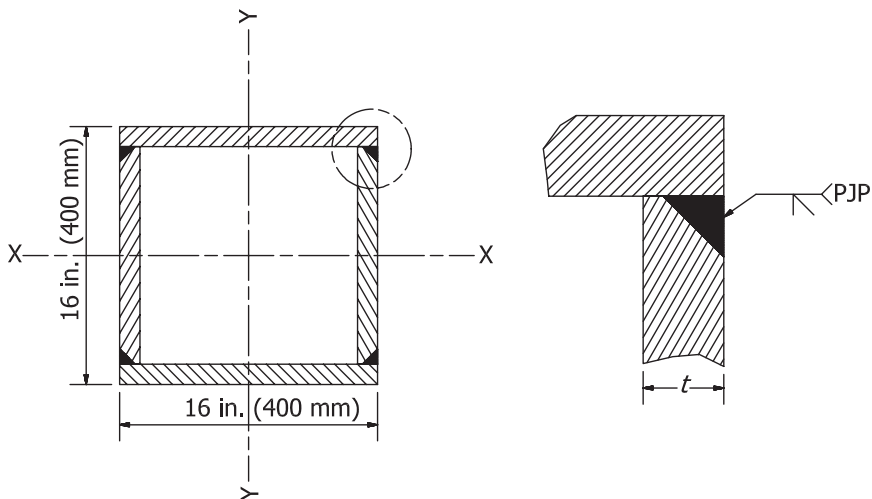


Fig. 10.5. Built-up box column flange-to-web connection detail.

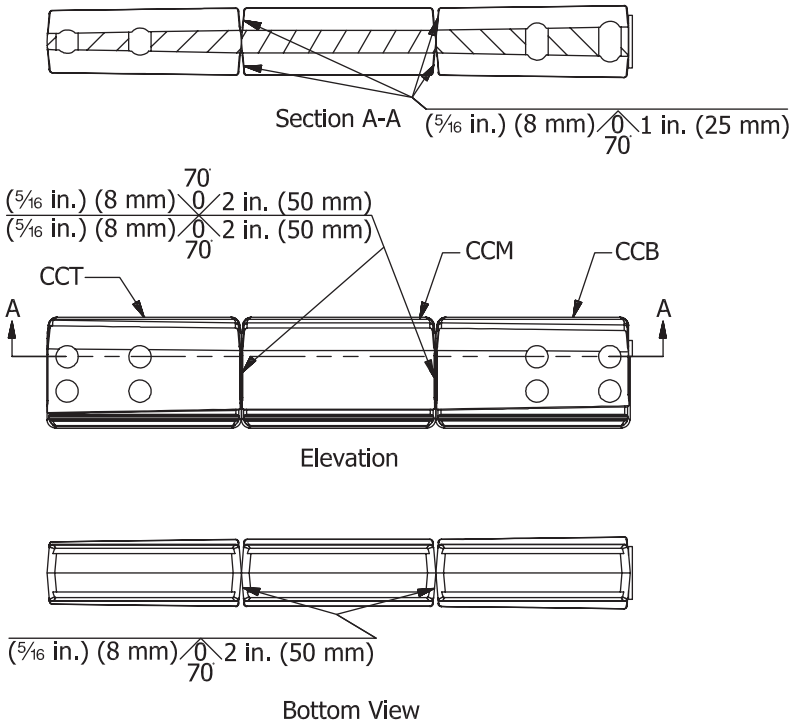


Fig. 10.6. Collar corner assembly welding.

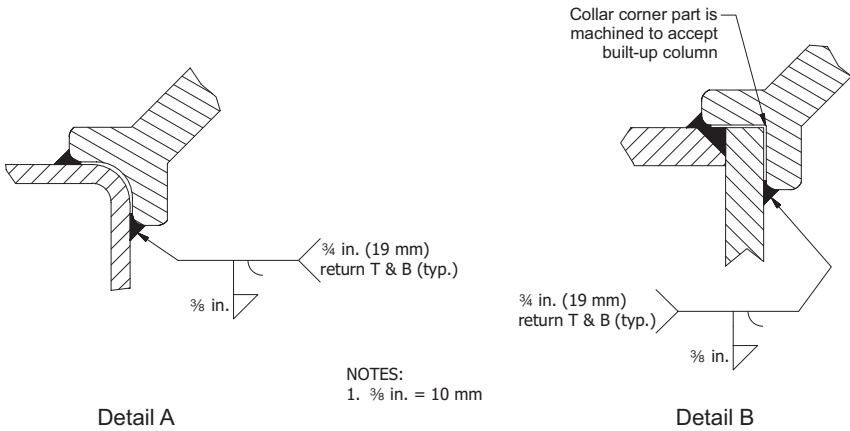
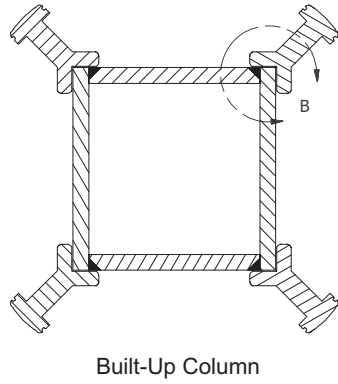
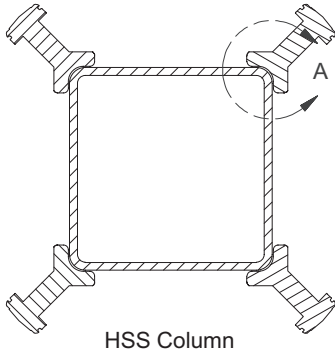


Fig. 10.7. Collar corner assembly-to-column weld, plan view.

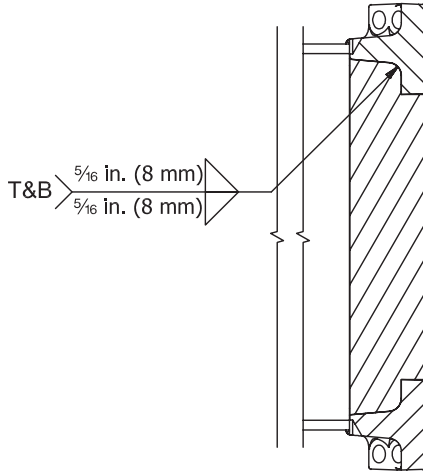


Fig. 10.8. Collar web extension-to-collar flange welds, elevation.

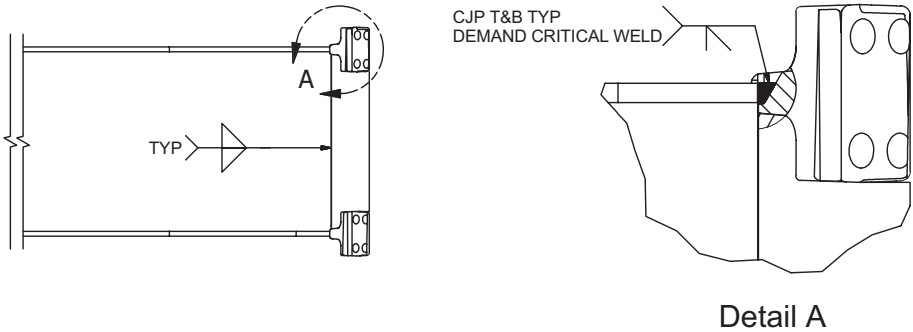


Fig. 10.9. Collar flange assembly-to-beam welds, elevation.

10.7. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3.

For beams with an RBS cutout, C_{pr} is determined using Equation 2.4-2 and $Z_e = Z_{RBS}$ defined in Equation 5.7-4.

For beams without an RBS cutout, $C_{pr} = 1.1$ and $Z_e = Z_b =$ plastic section modulus of the beam.

Step 2. Determine the location of the plastic hinge (see Figure 10.10).

For beams with an RBS cutout, the plastic hinge shall be assumed to occur at the center of the reduced section of beam flange.

For beams without an RBS cutout, the plastic hinge shall be assumed to occur at a distance $d/2$ from the outside face of the collar, where d is the beam depth.

Step 3. Compute the shear force at the plastic hinge, V_h , in accordance with Section 2.4.4.

Step 4. Compute the moment at the collar bolts for each beam:

$$M_{bolts} = M_{pr} + V_h S_{bolts} \quad (10.7-1)$$

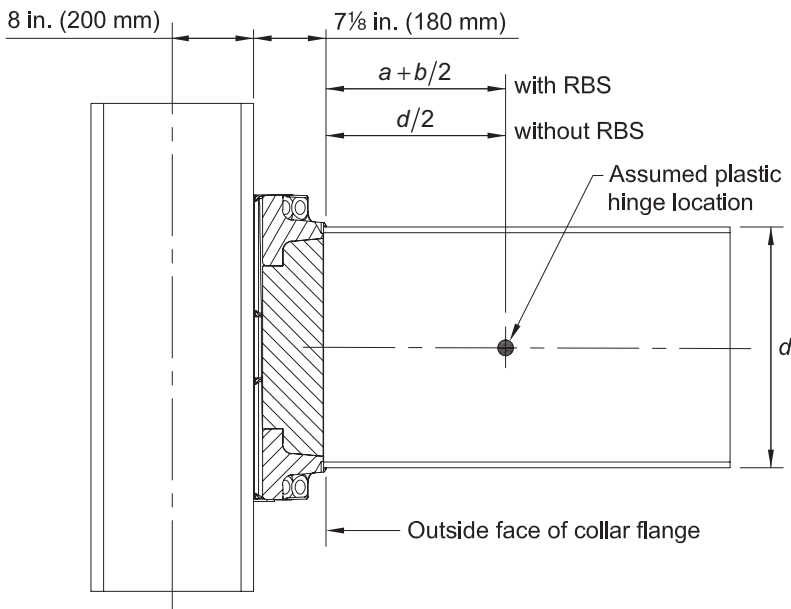


Fig. 10.10. Assumed plastic hinge location.

where

$$M_{bolts} = \text{moment at collar bolts, kip-in. (N-mm)}$$

$$s_{bolts} = \text{distance from plastic hinge to centroid of collar bolts, in. (mm)}$$

$$= \frac{t_{collar}}{2} + a + \frac{b}{2} \quad (\text{for RBS beams}) \quad (10.7-2)$$

$$= \frac{t_{collar}}{2} + \frac{d}{2} \quad (\text{for non-RBS beams}) \quad (10.7-3)$$

where

a = distance from outside face of collar to RBS cut, in. (mm)

b = length of RBS cut, in. (mm)

t_{collar} = distance from face of column to outside face of collar, taken as $7\frac{1}{8}$ in. (180 mm) as illustrated in Figure 10.10

Step 5. The following relationship shall be satisfied for the collar bolt tensile strength:

$$\frac{r_{ut}}{\phi_d R_{pt}} = \frac{r_{ut}}{102} \leq 1.0 \quad (10.7-4)$$

$$\frac{r_{ut}}{\phi_d R_{pt}} = \frac{r_{ut}}{454,000} \leq 1.0 \quad (10.7-4M)$$

where

$R_{pt} = T_b$ = minimum bolt pretension, kips (N), in accordance with the AISC *Specification*

n_{cf} = number of collar bolts per collar flange
= 8

r_{ut} = required collar bolt tensile strength, kips (N)

$$= \frac{M_{bolts}}{n_{cf} d \sin 45^\circ} = 0.177 \frac{M_{bolts}}{d} \quad (10.7-5)$$

User Note: For 1¼-in.- (31-mm-) diameter ASTM A574 bolts, the value of T_b is the same as for 1¼-in.- (31-mm-) diameter ASTM F3125/F3125M Grade A490 or A490M bolts and has a value of 102 kips (450 000 N).

Step 6. Compute V_{bolts} , the probable maximum shear at the collar bolts taken equal to V_h calculated in accordance with Section 2.4.4, plus any additional gravity loads between the plastic hinge and center of the collar flange, using the load combination in Section 2.4.4. Confirm that V_{bolts} is less than the slip-critical, Class A, bolt design strength in accordance with the AISC *Specification* and using a resistance factor, ϕ , of 1.00.

Step 7. Compute V_{cf} , the probable maximum shear at the face of collar flange taken equal to V_h calculated in accordance with Section 2.4.4, plus any additional gravity loads between the plastic hinge and outside face of the collar flange, using the load combination in Section 2.4.4.

The required shear strength of the beam shall be taken equal to V_{cf} . Check the design shear strength of the beam in accordance with the AISC *Specification*.

Step 8. Determine the required size of the fillet weld connecting the beam web to the collar web extension (CWX) using the following relationship:

$$t_f^{CWX} \geq \frac{\sqrt{2}V_{cf}}{\phi_n F_w t_w^{CWX}} \quad (10.7-6)$$

where

$$F_w = 0.60F_{EXX}, \text{ ksi (MPa)}$$

t_w^{CWX} = total length of available fillet weld at CWX, in. (mm), taken as 54 in.

(1400 mm) for W30 (W760) sections; 48 in. (1200 mm) for W27 (W690) sections; 42 in. (1100 mm) for W24 (W610) sections; 36 in. (900 mm) for W21 (W530) sections; and 30 in. (750 mm) for W18 (W460) sections

t_f^{CWX} = fillet weld size required to join each side of beam web to CWX, in. (mm)

Step 9. Compute V_f , the probable maximum shear at the face of column, taken equal to V_h calculated in accordance with Section 2.4.4, plus shear due to gravity load between the plastic hinge and the face of the column, using the load combination in Section 2.4.4.

Determine the size of fillet weld connecting the collar corner assemblies to the column using the following relationship:

$$t_f^{CC} \geq \frac{\sqrt{2}V_f}{\phi_n F_w t_w^{CC}} \quad (10.7-7)$$

where

t_w^{CC} = total length of available fillet weld at collar corner assembly, in. (mm),

taken as 72 in. (1800 mm) for W30 (W760) sections; 66 in. (1700 mm)

for W27 (W690) sections; 60 in. (1500 mm) for W24 (W610) sections;

54 in. (1400 mm) for W21 (W530) sections; and 48 in. (1200 mm) for W18 (W460) sections

t_f^{CC} = fillet weld size required to join collar corner assembly to column, in. (mm)

Step 10. Check the design shear strength of the beam in accordance with the AISC *Specification*.

The required shear strength of the beam shall be taken as V_f .

Step 11: Check the panel zone in accordance with Section 2.7.

When computing the required panel-zone shear strength, the depth of the panel zone shall be taken as the depth of the beam.

The design panel-zone shear strength, ϕR_n^{pz} , shall be computed as

$$\phi R_n^{pz} = \phi_d 0.6F_y A_{pz} \quad (10.7-8)$$

where

$$A_{pz} = 2d_c t_{col} + 4(d_{leg}^{CC} t_{leg}^{CC}) \quad (10.7-9)$$

F_y = specified minimum yield stress, ksi (MPa)

d_c = depth of column, in. (mm)

d_{leg}^{CC} = effective depth of collar corner assembly leg, taken as 3½ in. (89 mm)

t_{col} = wall thickness of HSS or built-up box column, in. (mm)

t_{leg}^{CC} = effective thickness of collar corner assembly leg, taken as ½ in. (13 mm)

User Note: If the required strength exceeds the design strength, the designer may increase the column section and/or decrease the beam section strength, assuring that all other design criteria are met.

Step 12. Check the column-beam moment ratio as follows:

- (a) For SMF, check the column-beam moment ratio in accordance with Section 2.8 considering simultaneous development of the probable maximum moments in the moment-connected beams framing into all sides of the ConXL node.

For the purpose of satisfying this requirement, it is permitted to take the yield strength of the column material as the specified F_y and to consider the full composite behavior of the column for axial load and flexural action.

User Note: The specified value of F_y may be different than the specified minimum value associated with the grade of steel if project specifications require a higher minimum yield strength.

The value of $\sum M_{pc}^*$ about each axis shall be taken as

$$\sum M_{pc}^* = M_{pcu}^* + M_{pcl}^* + \frac{M_{pb}^*}{(H_u + H_l)} d \quad (10.7-10)$$

where

H_l = height of story below node, in. (mm)

H_u = height of story above node, in. (mm)

M_{pcu}^* = plastic moment of column above node, about the axis under consideration considering simultaneous axial loading and loading about the transverse axis, kip-in. (N-mm)

M_{pcl}^* = plastic moment of column below node, about the axis under consideration considering simultaneous axial loading and loading about the transverse axis, kip-in. (N-mm)

For sections with equal properties about both axes, it is permitted to take M_{pcu}^* and M_{pcl}^* as:

$$M_{pcu}^* = M_{pcl}^* = 0.67 Z_c F_y \left(1 - \frac{P_u}{A_s F_y + 0.85 A_c f'_c} \right) \quad (10.7-11)$$

where

A_c = area of concrete in column, in.² (mm²)

A_s = area of steel in column, in.² (mm²)

P_u = axial load acting on column at section under consideration in accordance with the applicable load combination specified by the building code, but not considering the amplified seismic load, kips (N)

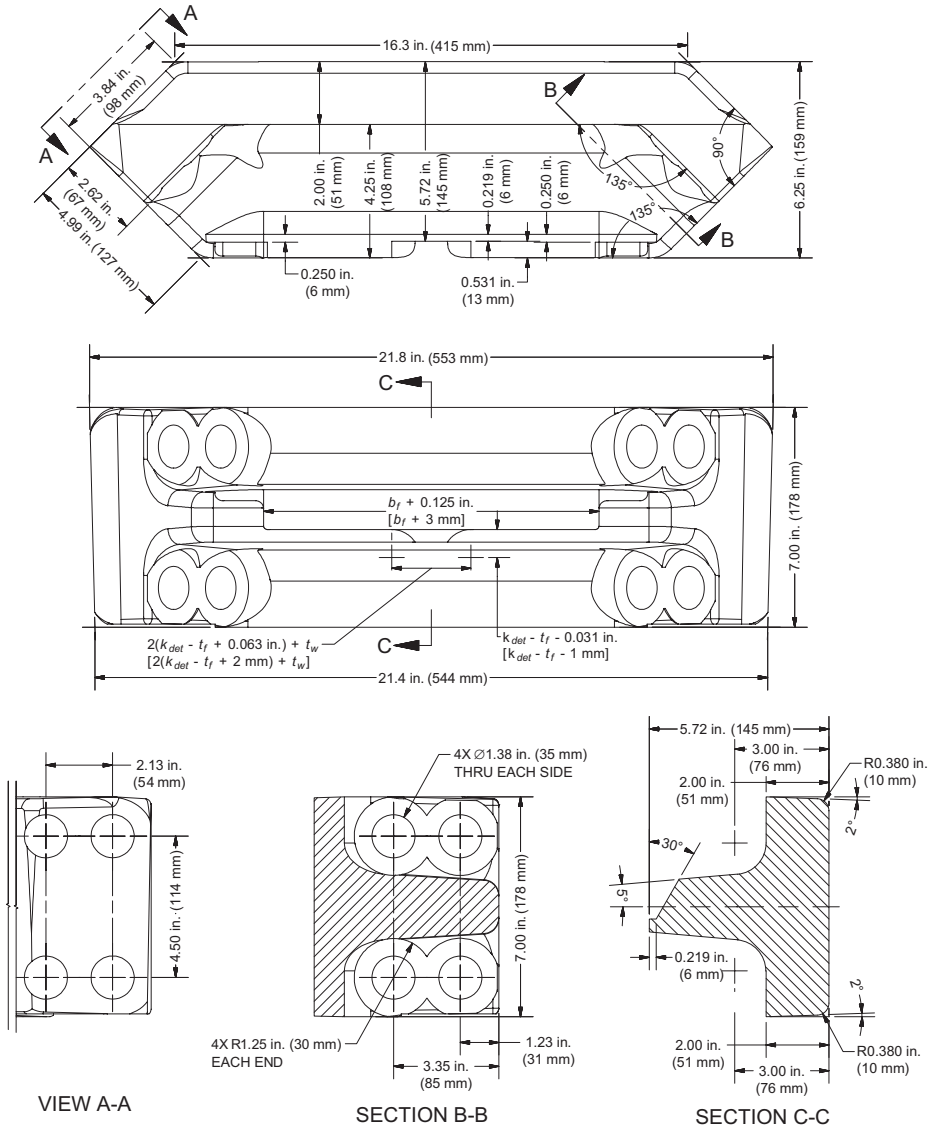
Z_c = plastic section modulus of the column about either axis, in.³ (mm³)

f'_c = specified compressive strength of concrete fill, ksi (MPa)

(b) For IMF, check column-beam moment ratio in accordance with Section 2.8.

10.8. PART DRAWINGS

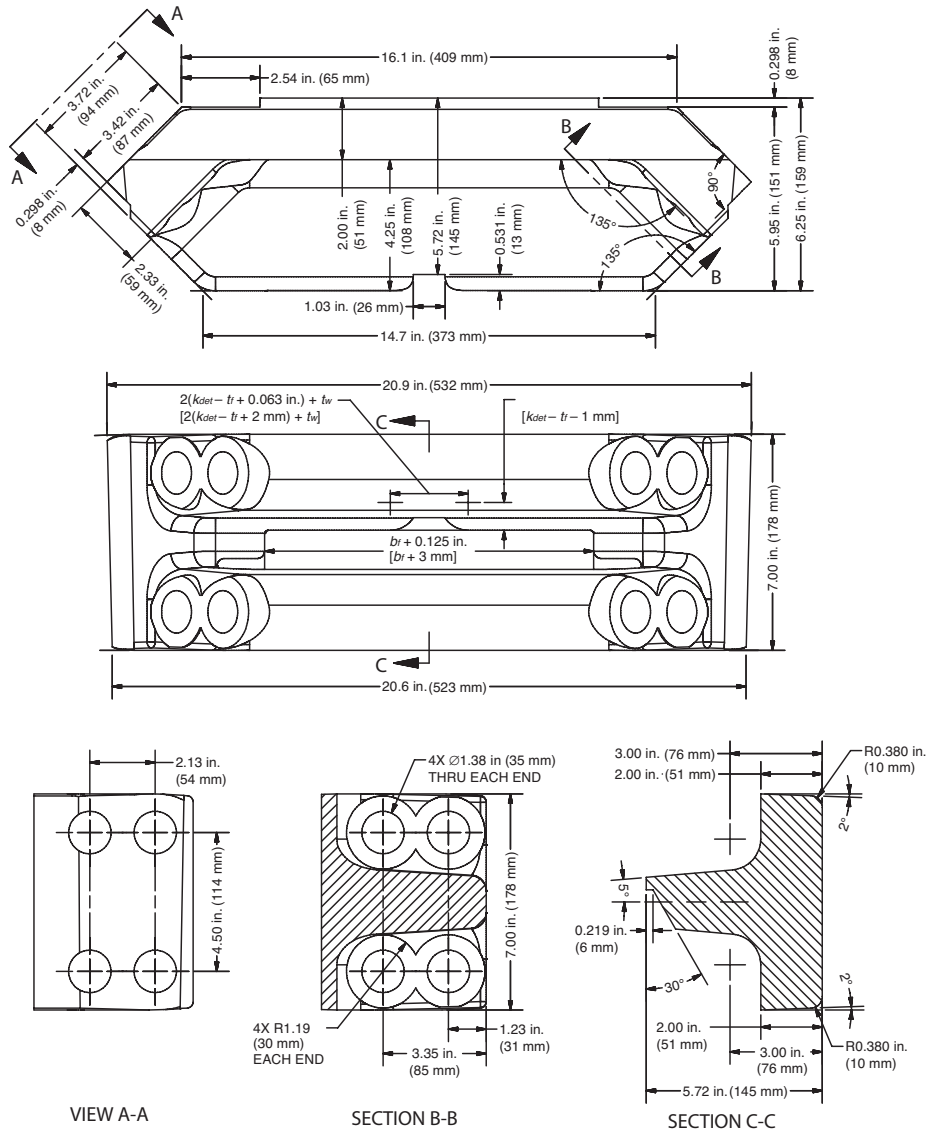
Figures 10.11 through 10.19 provide the dimensions of the various components of the ConXtech ConXL moment connection.



ConXL COLLAR FLANGE TOP

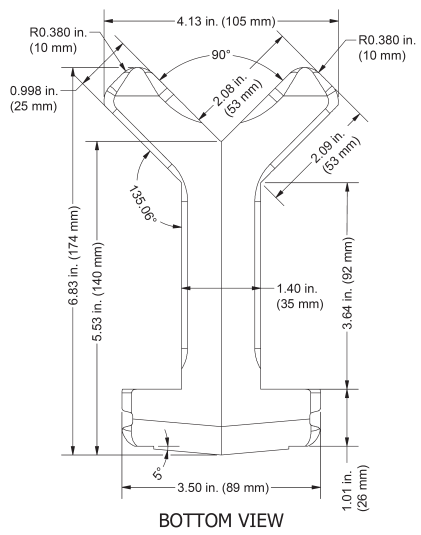
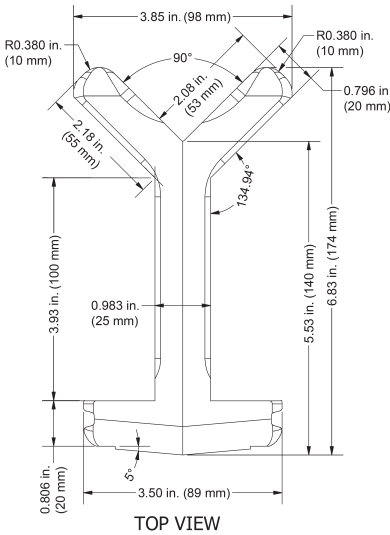
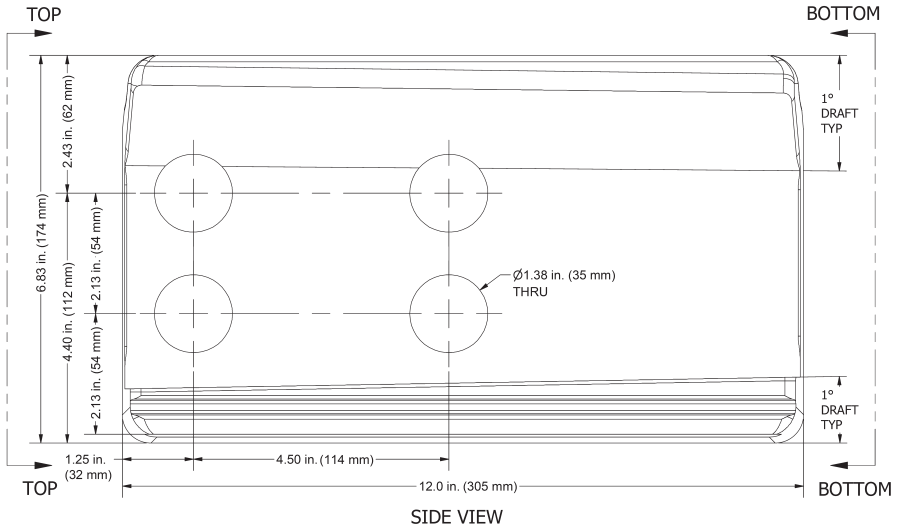
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Fig. 10.11. Forged collar flange top (CFT).



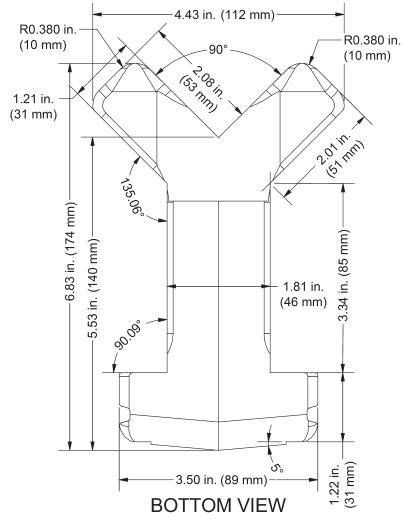
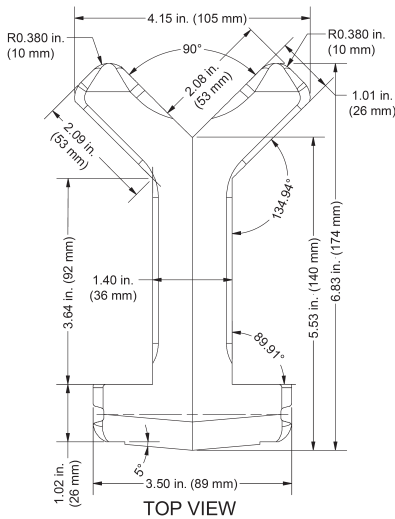
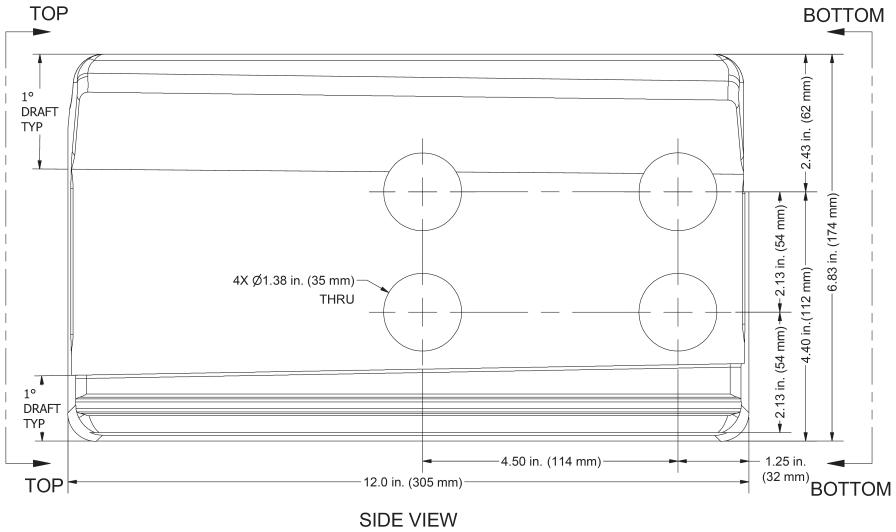
ConXL COLLAR FLANGE BOTTOM
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Fig. 10.12. Forged collar flange bottom (CFB).



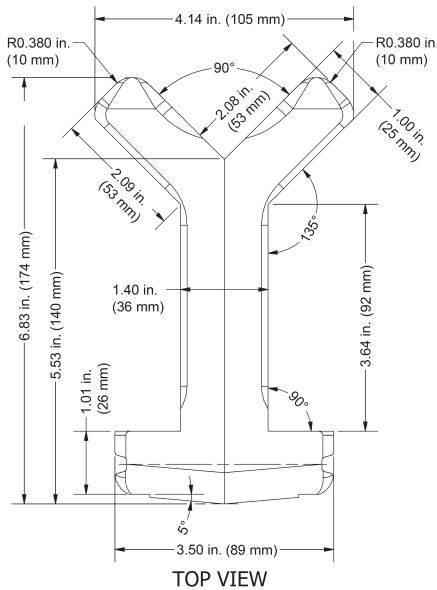
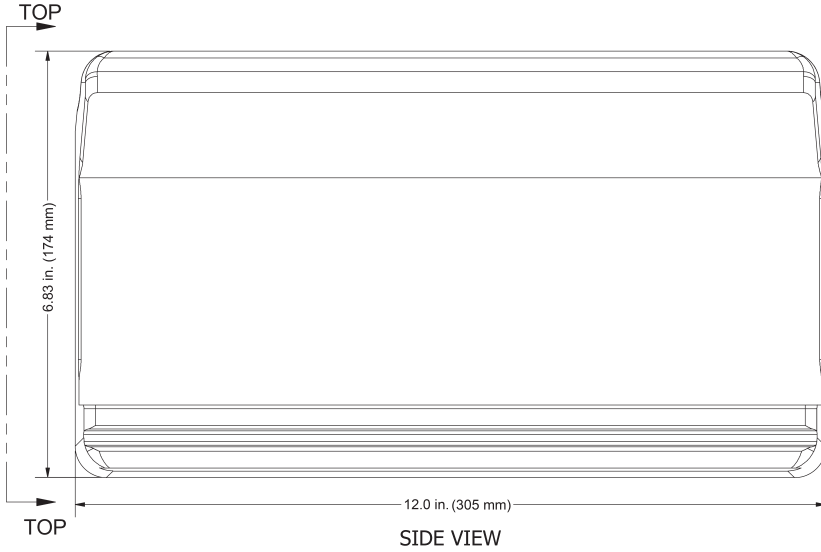
ConXL COLLAR CORNER TOP		NOTES	
		Description	Tolerance Value
UNITS:	IN / MM	FORGING TOLERANCE	+/- 0.3%
SIZE:	A	DIE WEAR TOLERANCE	+ 0.05% / - 0.0
		MILLING TOLERANCE	+/- 0.020"

Fig. 10.13. Forged collar corner top (CCT).



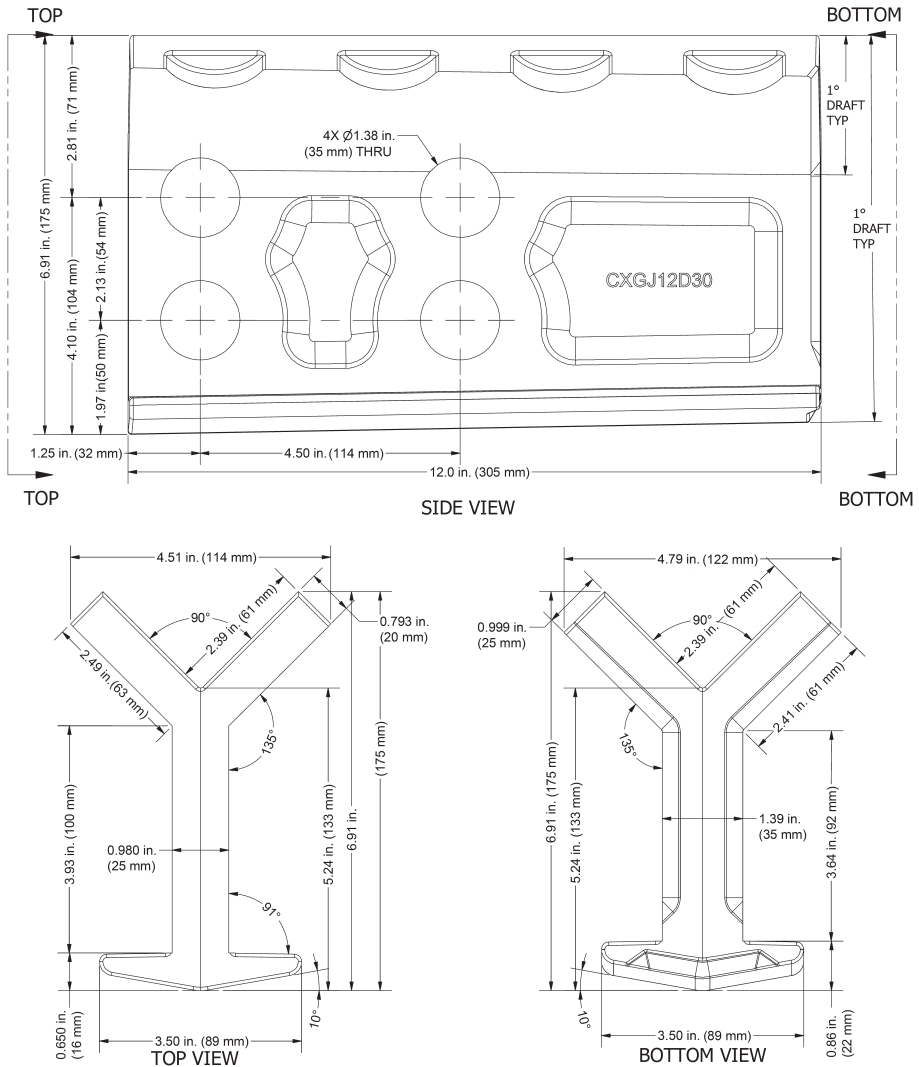
ConXL COLLAR CORNER BOTTOM		NOTES	
		Description	Tolerance Value
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		DIE WEAR TOLERANCE	+ 0.5% / - 0.0
		MILLING TOLERANCE	+/- 0.029"

Fig. 10.14. Forged collar corner bottom (CCB).



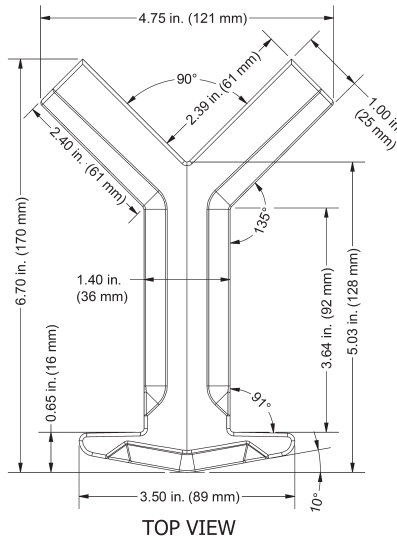
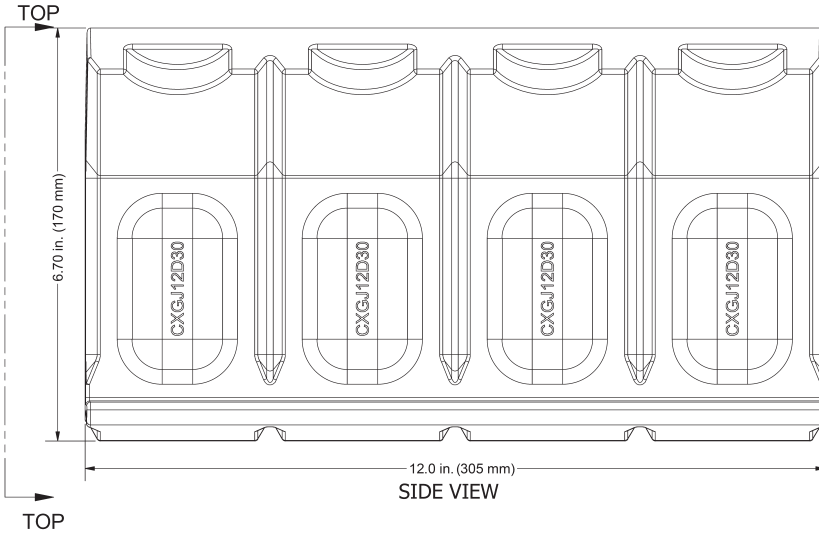
ConXL COLLAR CORNER MIDDLE			NOTES		
			Description	Tolerance Value	
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				DIE WEAR TOLERANCE	+ 0.5% / - 0.0
				MILLING TOLERANCE	± 0.020"

Fig. 10.15. Forged collar corner middle (CCM).



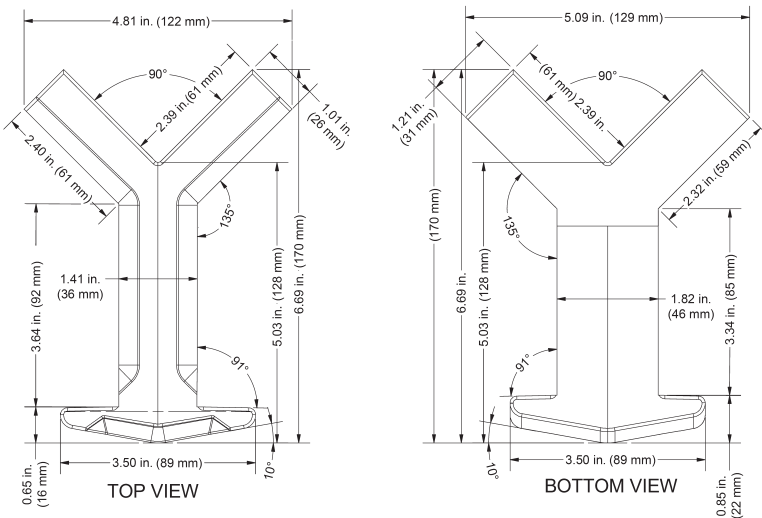
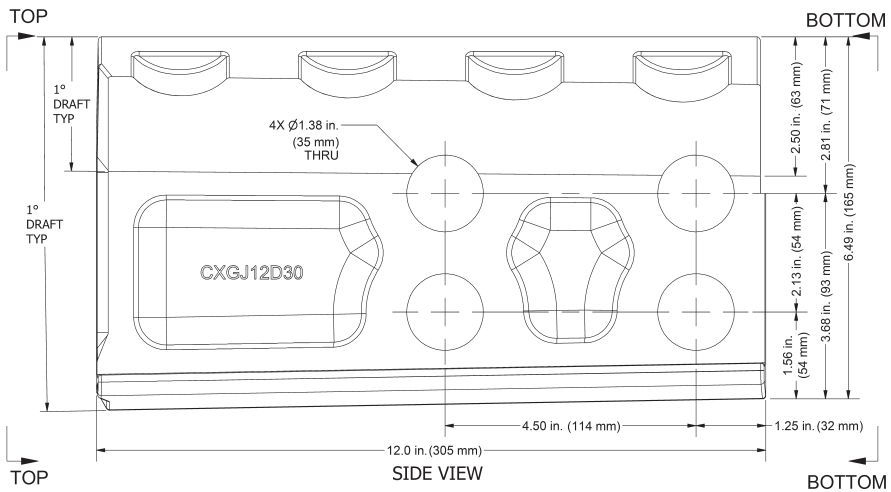
ConXL COLLAR CORNER TOP			NOTES	
			Description	Tolerance Value
			FORGING TOLERANCE	± 0.3%
			DIE WEAR TOLERANCE	+ 0.5% / - 0.0
			MILLING TOLERANCE	± 0.020"
UNITS:	IN / MM	SIZE:	A	

Fig. 10.16. Cast collar corner top (CCT).



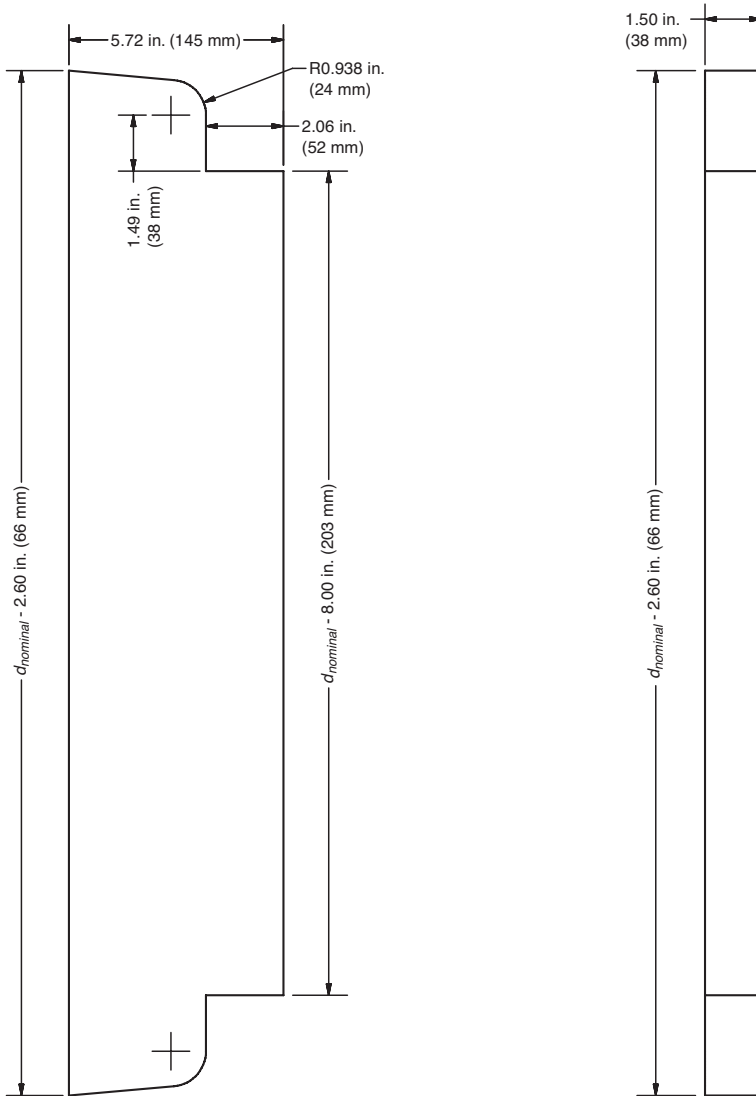
ConXL COLLAR CORNER MIDDLE		NOTES	
		Description	Tolerance Value
UNITS: IN / MM	SIZE: A	FORGING TOLERANCE	+/- 0.3%
		DIE WEAR TOLERANCE	+ 0.5% / - 0.0
		MILLING TOLERANCE	+/- 0.020"

Fig. 10.17. Cast collar corner middle (CCM).



ConXL COLLAR CORNER BOTTOM		
NOTES		
Description	Tolerance Value	
FORGING TOLERANCE	± 0.3%	
DIE WEAR TOLERANCE	+ 0.5% / - 0.0	
MILLING TOLERANCE	± 0.020"	
UNITS: IN / MM	SIZE: A	

Fig. 10.18. Cast collar corner bottom (CCB).



ConXL COLLAR WEB EXTENSION		
SCALE: ¼	UNITS: IN / MM	SIZE: A

Fig. 10.19. Collar web extension (CWX).

CHAPTER 11

SIDEPLATE MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by multiple U.S. and foreign patent rights. By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license, and the statement may be obtained from the standard's developer.*

11.1. GENERAL

The SidePlate[®] moment connection utilizes interconnecting plates to connect beams to columns. The connection features a physical separation, or gap, between the face of the column flange and the end of the beam(s). Both field-welded and field-bolted options are available. Beams may be either rolled or built-up wide-flange sections or hollow structural sections (HSS). Columns may be either rolled or built-up wide-flange, built-up box, boxed I-shaped, or HSS sections. Built-up flanged cruciform sections consisting of rolled shapes or built up from plates may also be used as columns for biaxial configurations. Figures 11.1, 11.2, and 11.3 show the various field-welded and field-bolted uniaxial connection configurations. The field-bolted option is available in three configurations, referred to as configuration A (standard), configuration B (narrow), and configuration C (tuck) as shown in Figure 11.3.

In the field-welded connection, top and bottom beam flange cover plates (rectangular or U-shaped) are used at the end(s) of the beam, as applicable, which also serve to bridge any difference between flange widths of the beam(s) and of the column. The connection of the beam to the column is accomplished with parallel full-depth side plates that sandwich and connect the beam(s) and the column together. In the field-bolted connection, beam flanges are connected to the side plates with either a cover plate or a pair of angles and high-strength pretensioned bolts as shown in Figures 11.2 and 11.3. Column horizontal shear plates and beam vertical shear elements (as applicable) are attached to the wide-flange column and beam webs, respectively.

*The SidePlate[®] connection configurations and structures illustrated herein, including their described fabrication and erection methodologies, are protected by one or more of the following U.S. and foreign patents: U.S. Pat. Nos. 5,660,017; 6,138,427; 6,516,583; 6,591,573; 7,178,296; 8,122,671; 8,122,672; 8,146,322; 8,176,706; 8,205,408; 9,091,065; Mexico Pat. No. 208,750; New Zealand Pat. No. 300,351; British Pat. No. 2497635; all held by MiTek Holdings LLC. Other U.S. and foreign patent protection are pending.

Figure 11.4 shows the connection geometry and major connection components for uniaxial field-welded configurations.

Figure 11.5 shows the connection geometry and major connection components for biaxial field-welded configurations, which permits connecting up to four beams to a column. Figure 11.6 shows the analogous field-bolted biaxial connection with built-up box or HSS columns. Built-up box and HSS columns may be filled with concrete. All field-bolted beam configurations as shown in Figure 11.3 and all field-welded configurations as shown in Figure 11.4 are permitted in biaxial applications.

Figure 11.7 shows the SidePlate built-up box/HSS biaxial configuration. Figure 11.7(b) shows the assembly of the two intersecting side plates that are attached to the column to receive the beams as shown in Figure 11.7(c). Each side plate is slotted to accommodate the other orthogonal side plate as shown in Figure 11.7(a). The configuration shown in Figure 11.7(b) will be referred to as the side plate interlock assembly herein.

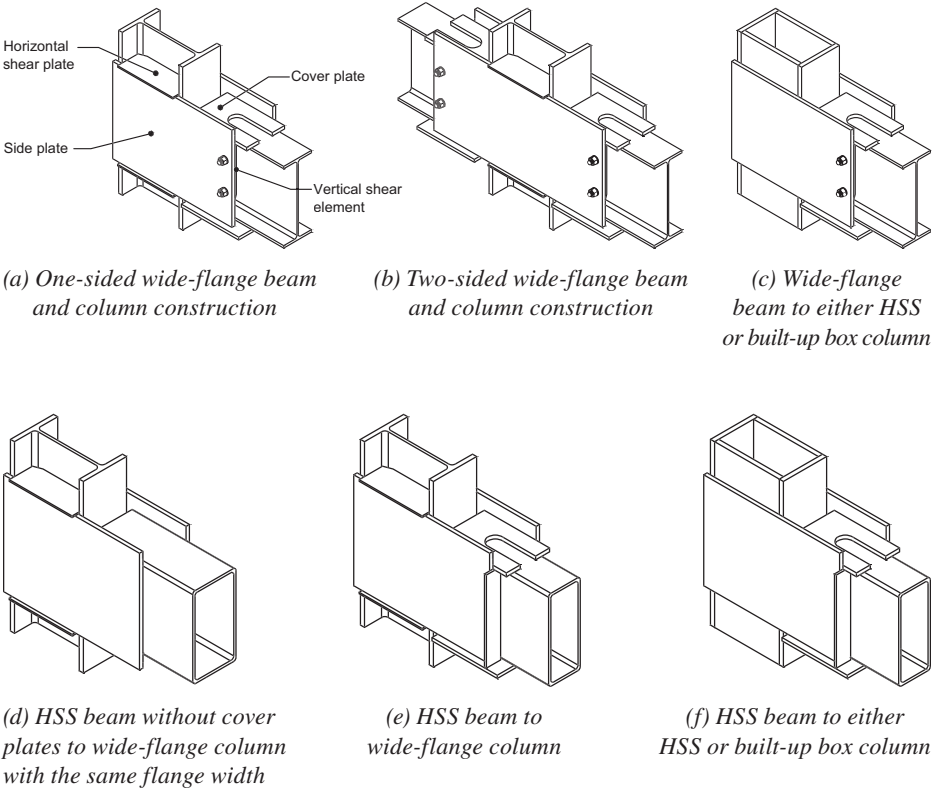


Fig. 11.1. Assembled SidePlate uniaxial field-welded configurations.

Two different details may be used for constructing the side plate interlock assembly depending on the type of the column. Figure 11.8(a) shows the plan view of a typical SidePlate biaxial configuration with an HSS column where the side plates are connected with four fillet welds as shown in Figure 11.8(b). Figure 11.8(c) shows a typical SidePlate biaxial configuration with a built-up box column where the side plates are connected with a combination of fillet and PJP groove welds as shown in Figure 11.8(d). The latter detail with PJP groove welds may also be used with HSS columns.

User Note: The side plate interlock assembly may be preassembled prior to attachment to the HSS or built-up box columns. The side plates of the interlock assembly with PJP groove welds shown in Figure 11.8(d) may alternatively be attached to the column one at a time while turning the column around its longitudinal axis.

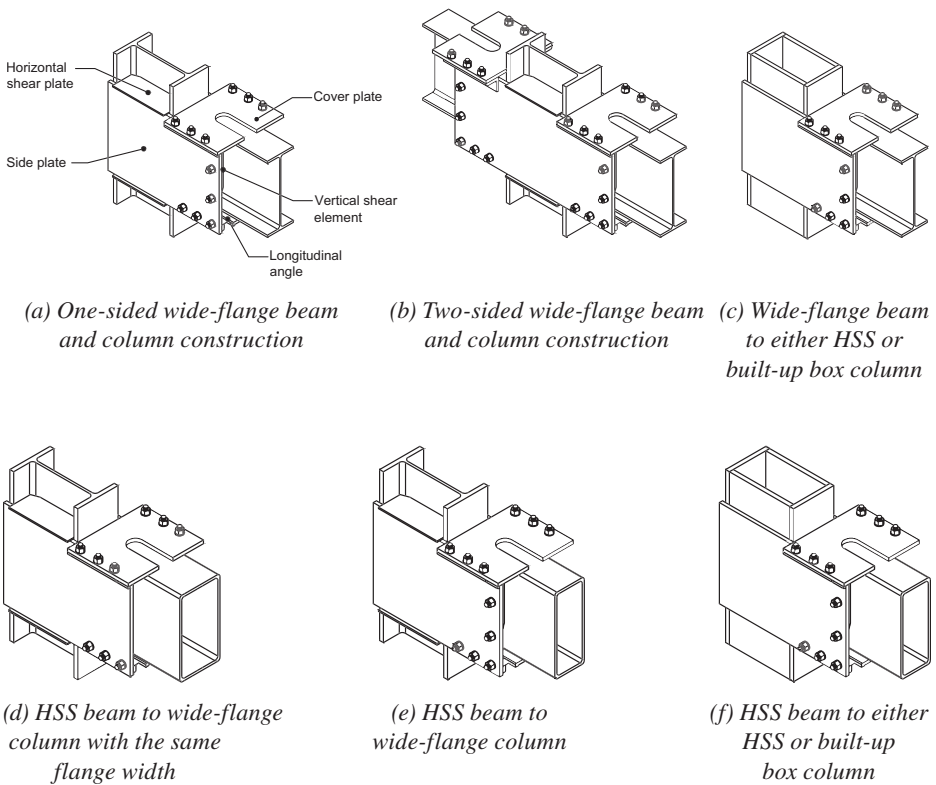


Fig. 11.2. Assembled SidePlate uniaxial field-bolted standard configurations (configuration A).

The SidePlate moment connection is proportioned to develop the probable maximum moment capacity of the connected beam. Plastic hinge formation is intended to occur primarily in the beam beyond the end of the side plates away from the column face, with limited yielding occurring in some of the connection elements.

User Note: Moment frames that utilize the SidePlate connection can be constructed using one of three methods: the full-length beam erection method (SidePlate FRAME configuration), the link-beam erection method (SidePlate Original configuration), and the fully shop prefabricated method. These methods are described in the Commentary.

11.2. SYSTEMS

The SidePlate moment connection is prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions. The SidePlate moment connections are prequalified for use in

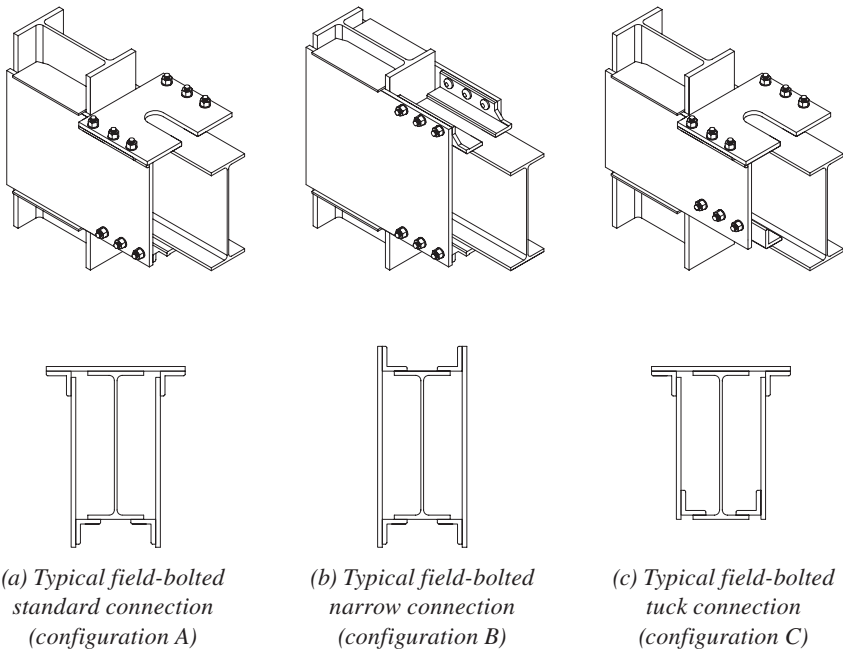
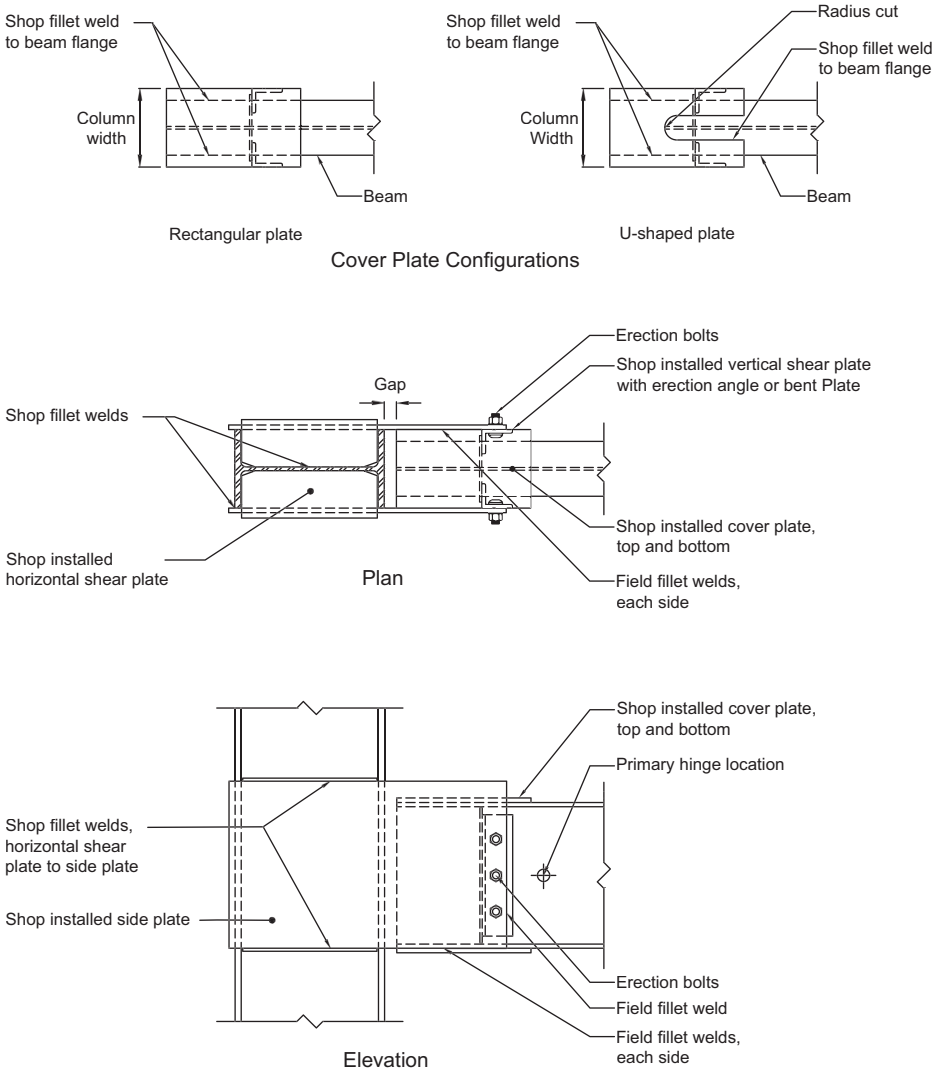
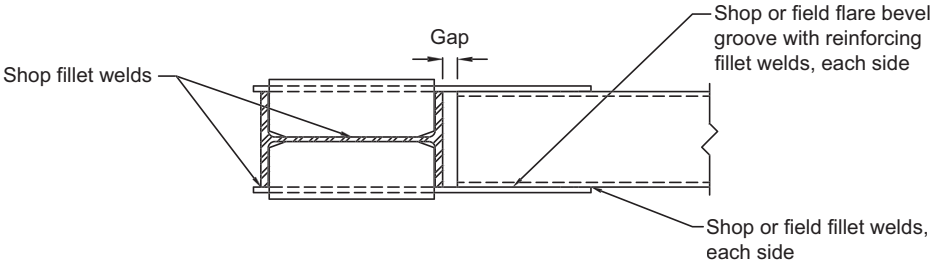


Fig. 11.3. SidePlate field-bolted connection configurations.

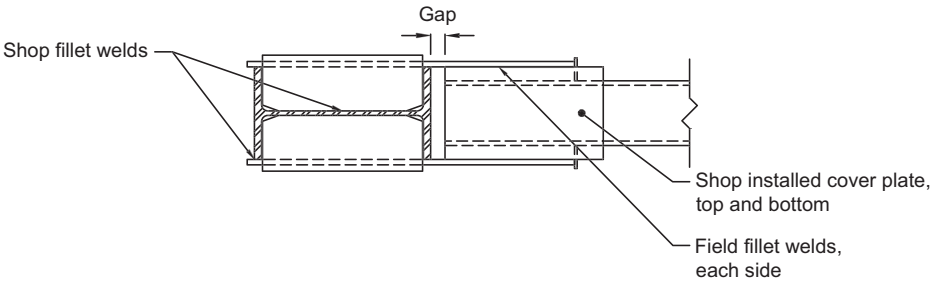


(a) Typical wide-flange beam to wide-flange column—detail, plan, and elevation views

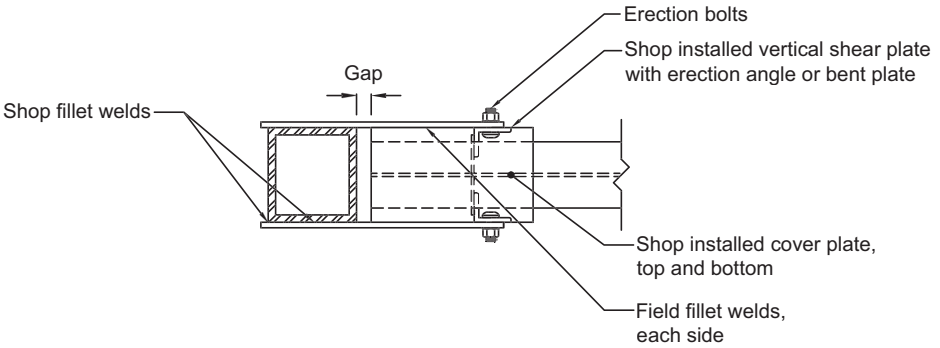
Fig. 11.4. (a) SidePlate uniaxial field-welded configuration geometry and major components.



(b) HSS beam without cover plates to wide-flange column with the same flange width—plan view



(c) HSS beam to wide-flange column—plan view



(d) Wide-flange beam to built-up box column—plan view

Fig. 11.4. (b–d) SidePlate uniaxial field-welded configuration geometry and major components.

planar moment-resisting frames and orthogonal intersecting moment-resisting frames (biaxial configurations, capable of connecting up to four beams at a column as illustrated in Figures 11.5 and 11.6). The SidePlate connection is exempt from the requirements of Section 2.3.4.

11.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange, HSS, or built-up I-shaped beams conforming to the requirements of Section 2.3. Beam flange thickness shall be limited to a maximum of 3.25 in. (81 mm).

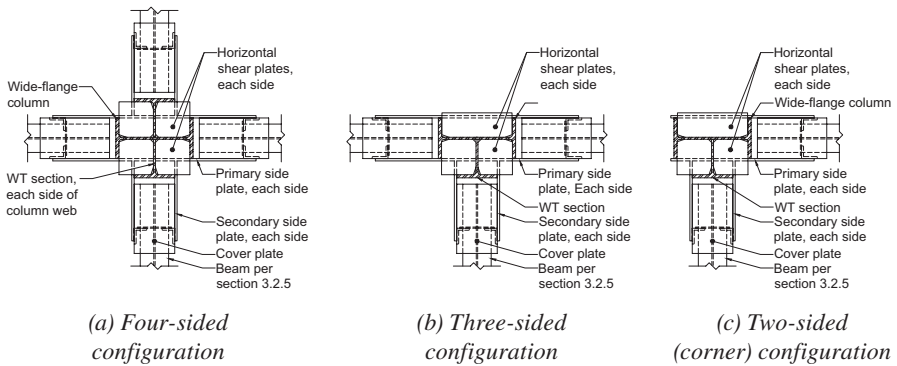


Fig. 11.5. SidePlate biaxial field-welded connection with built-up flanged cruciform column plan views.

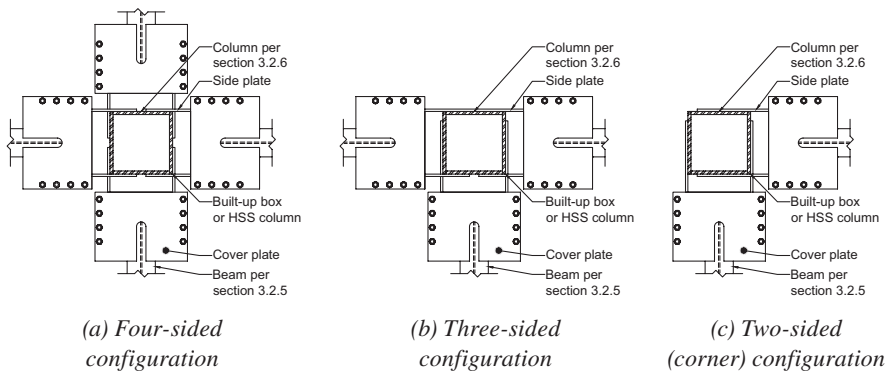


Fig. 11.6. SidePlate biaxial field-bolted connection with built-up box/HSS columns plan views.

- (2) Rolled wide-flange beam depths shall be limited to W40 (W1000) and W44 (W1100) for the field-welded and field-bolted connections, respectively. The depth of built-up wide-flange beams shall not exceed the depth permitted for rolled wide-flange beams.
- (3) Beam depths shall be limited as follows for HSS shapes:
 - (a) For SMF systems, HSS14 (HSS 356) or smaller.
 - (b) For IMF systems, HSS16 (HSS 406) or smaller.
- (4) Rolled and built-up wide-flange beam weight shall be limited to 302 lb/ft (450 kg/m) and 529 lb/ft (790 kg/m) for the field-welded and field-bolted connections, respectively. Beam flange area of the field-bolted connection shall be limited to a maximum of 48 in.² (30 000 mm²).
- (5) The ratio of the hinge-to-hinge span of the beam, L_h , to beam depth, d , shall be limited as follows:
 - (a) For SMF systems, L_h/d is limited to the following:
 - (i) 6 or greater with rectangular shaped cover plates.
 - (ii) 4.5 or greater with U-shaped cover plates for field-welded connections.
 - (iii) 4.0 or greater with U-shaped cover plates for field-bolted connections.

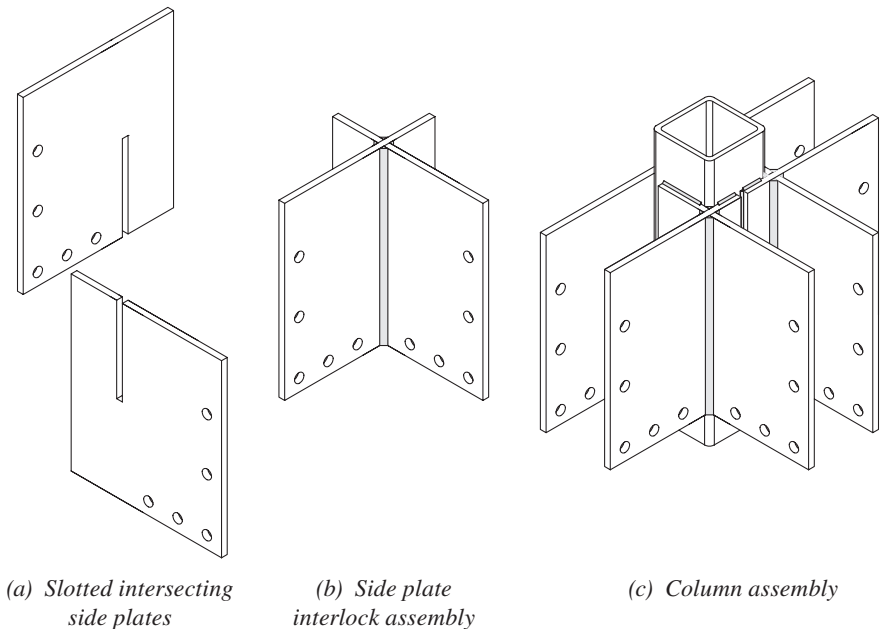


Fig. 11.7. SidePlate built-up box/HSS biaxial configuration.

(b) For IMF systems, L_h/d is limited to 3 or greater.

The hinge-to-hinge span of the beam, L_h , is the distance between the locations of plastic hinge formation at each moment-connected end of that beam. The location of the plastic hinge shall be taken as one-third of the beam depth, $d/3$, for the field-welded connection and one-sixth of the beam depth, $d/6$, for the field-bolted connection, away from the end of the side-plate extension, as shown in Figure 11.9. Thus,

$$L_h = L - \frac{1}{2}(d_{c1} + d_{c2}) - 2(0.33)d - 2A \quad (\text{field-welded}) \quad (11.3-1a)$$

$$L_h = L - \frac{1}{2}(d_{c1} + d_{c2}) - 2(0.165)d - 2A \quad (\text{field-bolted}) \quad (11.3-1b)$$

where

L = distance between column centerlines, in. (mm)

d_{c1}, d_{c2} = depth of column on each side of a bay in a moment frame, in. (mm)

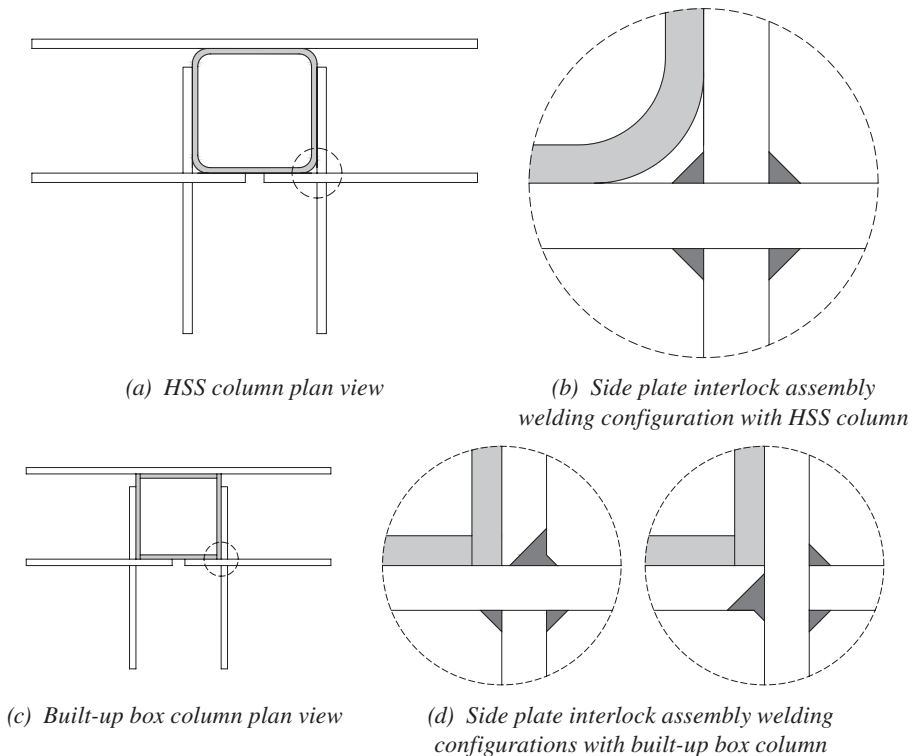


Fig. 11.8. Biaxial side plate interlock assembly welding options.

User Note: The $0.33d$ and $0.165d$ constants represent the distance of the plastic hinge from the end of the side plate extension. A represents the typical extension of the side plates from the face of column flange.

- (6) Width-to-thickness ratios for beam flanges and webs shall conform to the limits of the AISC *Seismic Provisions*.
- (7) Lateral bracing of wide-flange beams shall be provided in conformance with the AISC *Seismic Provisions*. Lateral bracing of HSS beams shall be provided in conformance with AISC *Specification* Appendix 1, Section 1.3.2c, taking $M'_1/M_2 = -1$ in AISC *Specification* Equation A-1-7. For either wide-flange or HSS beams, the segment of the beam connected to the side plates shall be considered to be braced. Supplemental top and bottom beam flange bracing at the expected hinge is not required.
- (8) The protected zone in the beam for the field-welded and field-bolted connections shall consist of the portion of the beam as shown in Figures 11.10 and 11.11, respectively.

2. Column Limitations

Columns shall satisfy the following limitations:

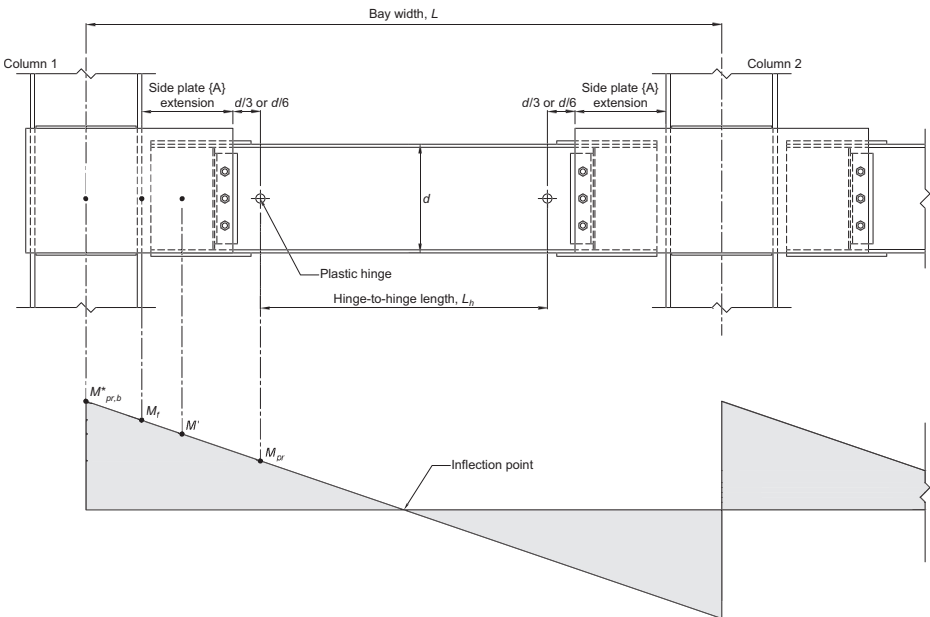


Fig. 11.9. Plastic hinge location and hinge-to-hinge length.

- (1) Columns shall be any of the shapes permitted in Section 2.3. Flange and web plates of built-up box columns shall continuously be connected by fillet welds or PJP groove welds along the length of the column.
- (2) HSS column shapes shall conform to ASTM A500/A500M or A1085/A1085M.
- (3) The beam shall be connected to the side plates that are connected to the flange tips of the wide-flange column or corners/sides of HSS or box columns.
- (4) Rolled shape column depth shall be limited to W44 (W1100). The depth of built-up wide-flange columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width exceeding 33 in. (840 mm).
- (5) There is no limit on column weight per foot.
- (6) There are no additional requirements for column flange thickness.
- (7) Width-to-thickness ratios for the flanges and webs of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (8) Lateral bracing of columns in accordance with AISC *Seismic Provisions* Section E3.4c1 is not a requirement if the beam is braced at the top beam flange (e.g., with a deck or slab), otherwise, lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

3. Connection Limitations

The connection shall satisfy the following limitations.

- (1) Material used for plate elements shall comply with the following:
 - (a) For the side plate interlock assembly shown in Figure 11.7 that utilizes plates greater than $2\frac{1}{4}$ in. (57 mm) in thickness, the intersecting plates shall be fabricated from structural steel that complies with ASTM A709/A709M Grade 50T zone 2 (Grade 345T) or normalized ASTM A572/A572M Grade 50 (Grade 345).
 - (b) All other connection plates, which consist of side plates, cover plates, horizontal shear plates, and vertical shear elements (if applicable) shall be fabricated from structural steel that complies with ASTM A572/A572M Grade 50 (Grade 345).

Exception: The vertical shear element as defined in Section 11.6 may be fabricated using ASTM A36/A36M material.
- (2) The extension of the side plates beyond the face of the column shall be within the range of $0.65d$ to $1.0d$ for the field-welded connection and $0.65d$ to $1.7d$ for the field-bolted connection, where d is the nominal depth of the beam.

- (3) The protected zone of the connection in the side plates shall consist of a portion of each side plate that is 6 in. (150 mm) high and starts at the inside face of the flange of a wide-flange or HSS column and ends either at the end of the gap (field-welded connection) or the edge of the first bolt hole (field-bolted connection) as shown in Figures 11.10 and 11.11.

11.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations.

- (1) Beam flange width and thickness for rolled, built-up, and HSS shapes shall satisfy the following equations for geometric compatibility (see Figure 11.12):

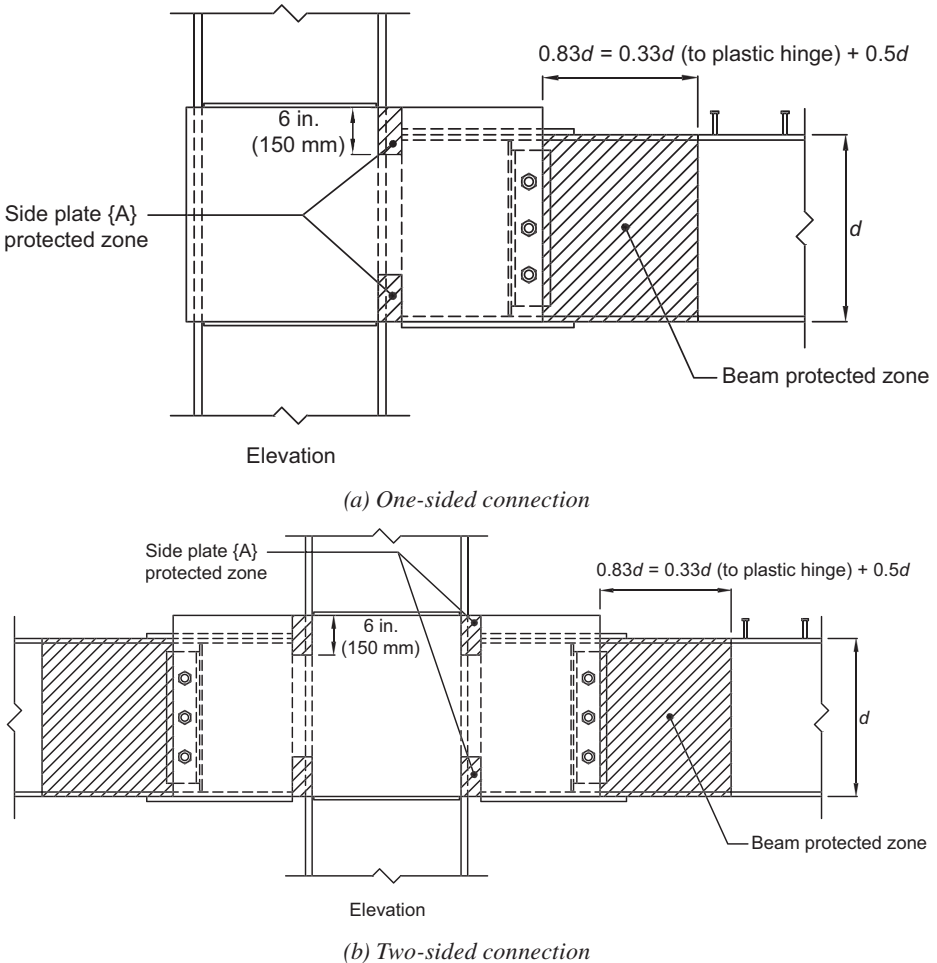


Fig. 11.10. Location of beam and side plate protected zones for the field-welded connection.

(a) Field-welded connection

$$b_{bf} + 1.1t_{bf} + \frac{1}{2} \text{ in.} \leq b_{cf} \quad (11.4-1a)$$

$$b_{bf} + 1.1t_{bf} + 13 \text{ mm} \leq b_{cf} \quad (11.4-1aM)$$

(b) Field-bolted connection

$$b_{bf} + 1.0 \text{ in.} \leq b_{cf} \quad (11.4-1b)$$

$$b_{bf} + 25 \text{ mm} \leq b_{cf} \quad (11.4-1bM)$$

where

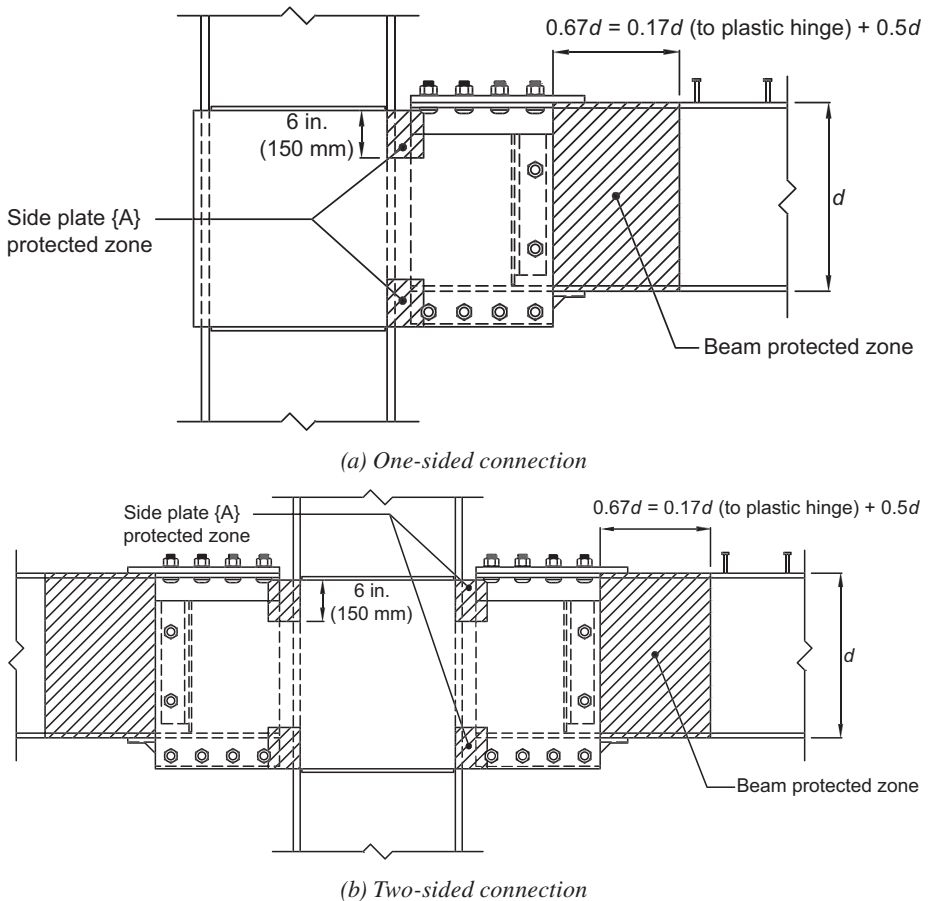
 b_{bf} = width of beam flange, in. (mm) b_{cf} = width of column flange, in. (mm) t_{bf} = thickness of beam flange, in. (mm)

Fig. 11.11. Location of beam protected zone for the field-bolted connection.

- (2) Panel zones shall conform to the applicable requirements of the AISC *Seismic Provisions*.

User Note: The column web panel zone strength shall be determined using AISC *Specification* Section J10.6b.

- (3) Column-beam moment ratios shall be limited as follows:
- (a) For SMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions* as follows:
- (i) For both uniaxial and biaxial connections, the value of $\sum M_{pb}^*$ shall be the sum of the projections of the expected flexural strengths of the beam(s) at the plastic hinge locations to the column centerline (Figure 11.13). The expected flexural strength of the beam shall be calculated as:

$$\sum M_{pb}^* = \sum (1.1R_y F_{yb} Z_b + M_v) \quad (11.4-2)$$

where

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

M_v = additional moment due to shear amplification from the center of the plastic hinge to the centerline of the column. M_v shall

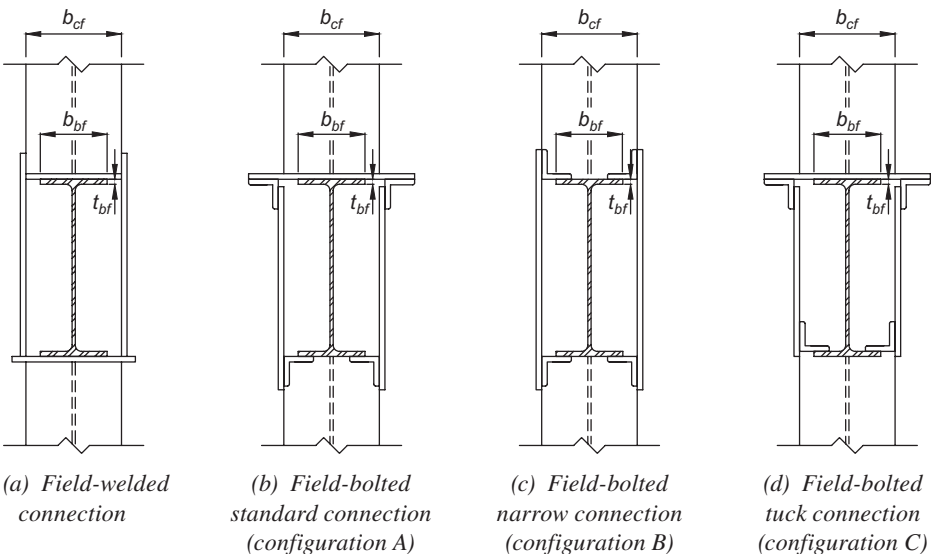


Fig. 11.12. Geometric compatibility.

be computed as the quantity $V_h s_h$, where V_h is the shear at the point of theoretical plastic hinging, calculated in accordance with Equation 11.4-3, and s_h is the distance of the assumed point of plastic hinging to the column centerline, which is equal to half the depth of the column plus the extension of the side plates beyond the face of column plus the distance from the end of the side plates to the plastic hinge, $d/3$.

$$V_h = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (11.4-3)$$

where

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

M_{pr} = probable maximum moment at plastic hinge, kip-in. (N-mm)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1 L + 0.15S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

R_y = ratio of expected yield stress to specified minimum yield stress, F_y , as specified in the AISC *Seismic Provisions*

Z_b = nominal plastic section modulus of beam about the x -axis, in.³ (mm³)

User Note: The load combination of $1.2D + f_1 L + 0.15S$ is in conformance with ASCE/SEI 7-22. When using the 2015 International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.15 for S (snow) when the roof configuration is such that it does not shed snow off the structure.

- (ii) For the uniaxial connection, the value of $\sum M_{pc}^*$ shall be the sum of the projections of the nominal flexural strengths of the column, M_{pc} , above and below the connection joint, at the location of theoretical hinge formation in the column (i.e., one-quarter the column depth above and below the extreme fibers of the side plates), to the beam centerline, with a reduction for the axial force in the column (Figure 11.13). The nominal flexural strength of the column shall be computed as:

$$\sum M_{pc}^* = \sum Z_{ec} (F_{yc} - P_{uc}/A_g) \quad (11.4-4)$$

where

F_{yc} = minimum specified yield strength of the column at the connection, ksi (MPa)

P_{uc}/A_g = ratio of column axial compressive load using load and resistance factor design (LRFD) load combinations, to gross area of the column, ksi (MPa)

Z_{ec} = equivalent plastic section modulus of column at a distance of $1/4$ column depth from top and bottom edge of side plates, projected to beam centerline, in.³ (mm³), and calculated as:

$$Z_{ec} = \frac{Z_c(H/2)}{H_h/2} = \frac{Z_c H}{H_h} \tag{11.4-5}$$

where

H = story height, in. (mm)

H_h = distance along column height from $1/4$ of column depth above top edge of lower-story side plates to $1/4$ of column depth below bottom edge of upper-story side plates, in. (mm)

Z_c = plastic section modulus of column, in.³ (mm³)

(iii) For the biaxial connection, the value of $\sum M_{pc}^*$ about each axis for the square HSS or built-up box columns shall be taken as:

$$\sum M_{pc}^* = 0.67 \sum Z_{ec} F_{ye} \left(1 - \frac{P_{uc}}{A_g F_{ye} + 0.85 A_c f'_c} \right) \tag{11.4-6}$$

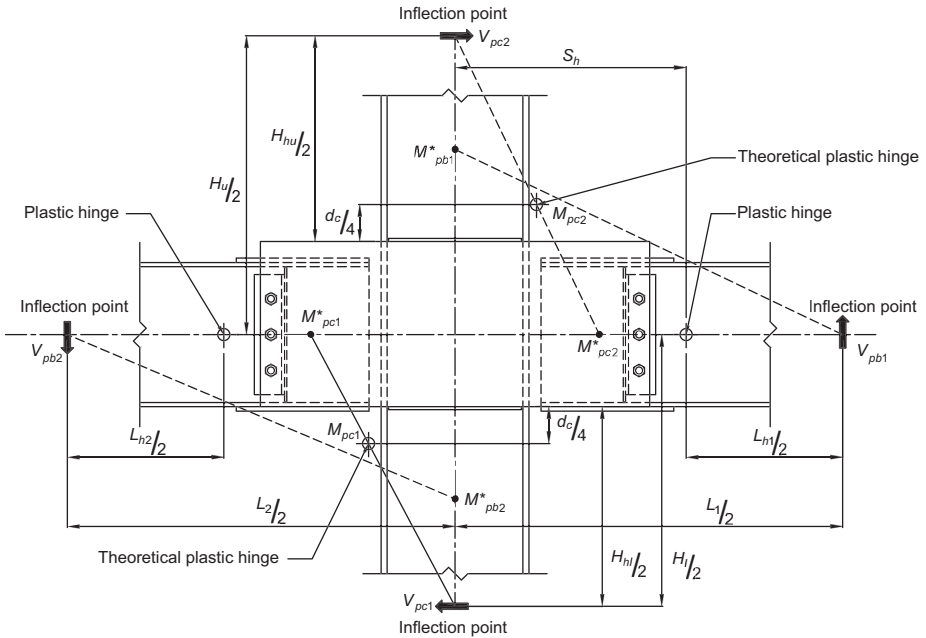


Fig. 11.13. Force and distance designations for calculation of column-beam moment ratios.

where

A_c = area of concrete in the column, if applicable, in.² (mm²)

A_g = area of the steel column, in.² (mm²)

F_{ye} = actual yield strength of the column at the connection. In the absence of F_{ye} , the expected yield strength of the column ($R_y F_{yc}$) may be used, ksi (MPa)

f'_c = specified compressive strength of the concrete infill, if applicable, ksi (MPa)

For column sections with unequal properties about both axes, interaction equations based on rational analysis shall be used.

User Note: Guidance for checking columns subject to biaxial bending and axial force is provided in AISC *Seismic Provisions* Section E3 Commentary.

For the purpose of satisfying strong-column/weak-beam requirements, it shall be permitted to take the actual yield strength of the column material as the specified minimum yield stress and to consider the full composite behavior of the column for axial and flexural loading action if it is filled with concrete.

- (b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*.

11.5. CONNECTION WELDING LIMITATIONS

Filler metals for the welding of beams, columns, and plates in the SidePlate connection shall meet the requirements for seismic force-resisting system welds in the AISC *Seismic Provisions*.

User Note: Mechanical properties for filler metals for seismic force-resisting system welds are detailed in AWS D1.8/D1.8M as referenced in the AISC *Seismic Provisions*.

The following welds are considered demand critical welds:

- (1) Shop fillet weld {2} that connects the inside face of the side plates to the wide-flange or HSS columns (see plan views in Figures 11.14, 11.15, and 11.16) and for biaxial cruciform dual-strong axis configurations connects the outside face of the secondary side plates to the outside face of primary side plates (see Figure 11.5).
- (2) Shop fillet weld {5} that connects the edge of the beam flange to the beam flange cover plate or angles (see Figures 11.17 and 11.18).
- (3) Shop fillet weld {5a} that connects the outside face of the beam flange to the beam flange U-shaped cover plate or angles (see Figures 11.17 and 11.18).

- (4) Field fillet weld {7} that connects the beam flange cover plates to the side plates [see Figure 11.19(a)] or connects the HSS beam flange to the side plates.
- (5) Fillet weld {8} that connects the top angles to the side plates in the field-bolted connection.
- (6) Shop weld {9} that connects side plate {A} to the column face (see Figures 11.20 and 11.21).
- (7) Shop fillet weld {10} that connects the intersecting orthogonal side plates to construct the side plate interlock assembly in biaxial connections (see Figures 11.20 and 11.21).

11.6. CONNECTION DETAILING

The following designations are used herein to identify plates and welds in the Side-Plate connection shown in Figures 11.14 through 11.21:

1. Plates/Angles

- {A} Side plate, located in a vertical plane parallel to the web(s) of the beam, connecting frame beam to column.
- {B} Beam flange cover plate bridging between side plates {A}, as applicable.
- {C} Vertical shear plate.
- {D} Horizontal shear plate (HSP). This element transfers horizontal shear from the top and bottom edges of the side plates {A} to the web of a wide-flange column.
- {E} Erection angle. One of the possible vertical shear elements {F}.
- {F} Vertical shear elements (VSE). These elements, which may consist of angles and plates or bent plates, transfer shear from the beam web to the outboard edge of the side plates {A}.
- {G} Longitudinal angles welded to the side plates {A} for connecting the beam flange cover plate (field-bolted connection).
- {H} Longitudinal angles welded to the beam flange for connecting to the side plates {A} (field-bolted connection).
- {T} Horizontal plates welded to the side plates {A} for connecting the beam flange cover plate as an alternative for angle {G} (field-bolted connection).

2. Welds

- {1} Shop fillet weld connecting exterior edge of side plate {A} to the horizontal shear plate {D} or to the face of a built-up box column or HSS section.
- {2} Shop fillet weld connecting inside face of side plate {A} to the tip of the column flange or to the corner of an HSS or built-up column section; and for biaxial dual-strong axis configurations connects outside face of secondary side plates to outside face of primary side plates.

- {3} Shop fillet weld connecting horizontal shear plate {D} to wide-flange column web. Weld {3} is also used at the column flanges where required to resist orthogonal loads through the connection due to collectors, chords, or cantilevers.
- {4} Shop fillet weld connecting vertical shear elements {F} to the beam web, and where applicable, the vertical shear plate {C} to the erection angle {E}.
- {5} Shop fillet weld connecting beam flange tip to cover plate {B}/angles {H}.
- {5a} Shop weld connecting outside face of beam flange to cover plate {B} (or to the face of the beam flange with the angles {H}).
- {6} Field vertical fillet weld connecting vertical shear element (angle or bent plate) {F} to end of side plate {A} (field-welded connection).
- {7} Field horizontal fillet weld connecting the cover plate {B} to the side plate {A}, or connecting HSS beam corners to side plates (field-welded connection).
- {8} Shop weld connecting the longitudinal angles {G} or horizontal plate {T} to the side plate {A} (field-bolted connection).
- {9} Shop fillet weld connecting side plate {A} to HSS/built-up box column in bi-axial configuration.
- {10} Shop weld connecting the intersecting orthogonal side plates to construct the side plate interlock assembly.

Figure 11.14 shows the connection detailing for a one-sided moment connection configuration in which one beam frames into a column (A-type). Figure 11.15 shows the connection detailing for a two-sided moment connection configuration in which the beams are identical (B-type). Figure 11.16 shows the connection detailing for a two-sided moment connection configuration in which the beams differ in depth (C-type). Figures 11.17 and 11.18 show the beam assembly shop detail for the field-welded and field-bolted connections, respectively. Figure 11.19 shows the beam-to-side plate field erection detail. If two beams frame into a column to form a corner, the connection detailing is referred to as a D-type (not shown). The connection detailing for a three-sided and four-sided moment connection configuration is referred to as an E-type and F-series, respectively (not shown). Figures 11.14, 11.15, and 11.16 show the field-welded connection. The same details are applicable to the field-bolted connection by using the beam end details for the field-bolted connection.

3. Bolts

- (1) Bolts shall be arranged symmetrically about the axis of the beam.
- (2) Types of holes
 - (a) Standard holes shall be used in the horizontal angles {G} and {H}.
 - (b) Either standard or oversized holes shall be used in the side plates and cover plates.

- (c) Either standard or short-slotted holes (with the slot parallel to the beam axis) shall be used in the angle of the vertical shear element (VSE), if applicable.
- (3) Bolt hole fabrication using thermal cutting is not permitted for plates thicker than 2 in. (50 mm).
- (4) All bolts shall be installed as pretensioned high-strength bolts.
- (5) Bolts shall be pretensioned high-strength bolts conforming to ASTM F3125/F3125M Grade A490 or A490M, or Grade F2280 or ASTM F3148 for fixed spline assemblies. The bolt diameter is limited to 1½ in. (36 mm) maximum.
- (6) The use of shim plates between the side plates and the cover plate or angles is permitted at either or both locations, subject to the limitations of the RCSC *Specification*.
- (7) Faying surfaces of side plates, cover plates, and angles shall have a Class A slip coefficient or higher.

User Note: The use of oversized holes in the side plates and cover plates with pretensioned bolts that are not designed as slip critical is permitted, consistent with AISC *Seismic Provisions* Section D2.2. Although standard holes are permitted in the side plate and cover plate, their use may result in field modifications to accommodate erection tolerances.

11.7. DESIGN PROCEDURE

Step 1. Choose trial frame beam and column section combinations that satisfy geometric compatibility based on Equation 11.4-1a (11.4-1aM) or 11.4-1b (11.4-1bM).

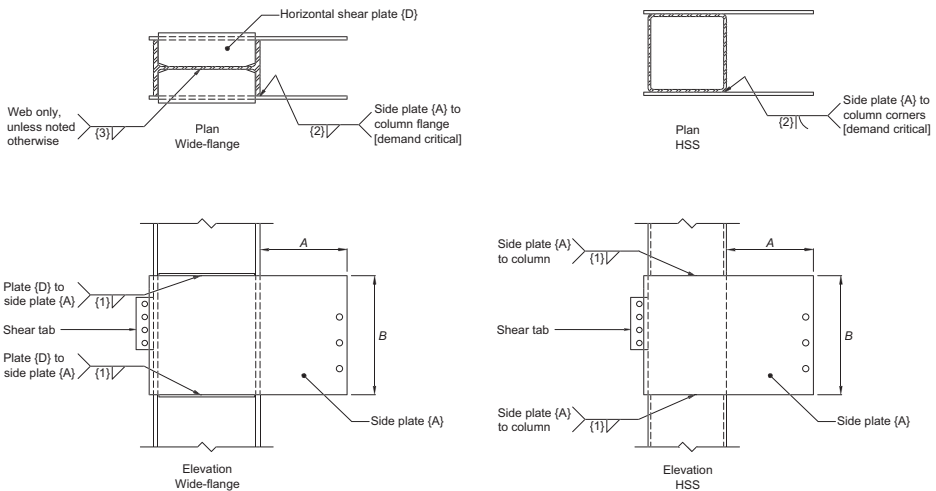


Fig. 11.14. One-sided SidePlate moment connection (A-type), column shop detail.

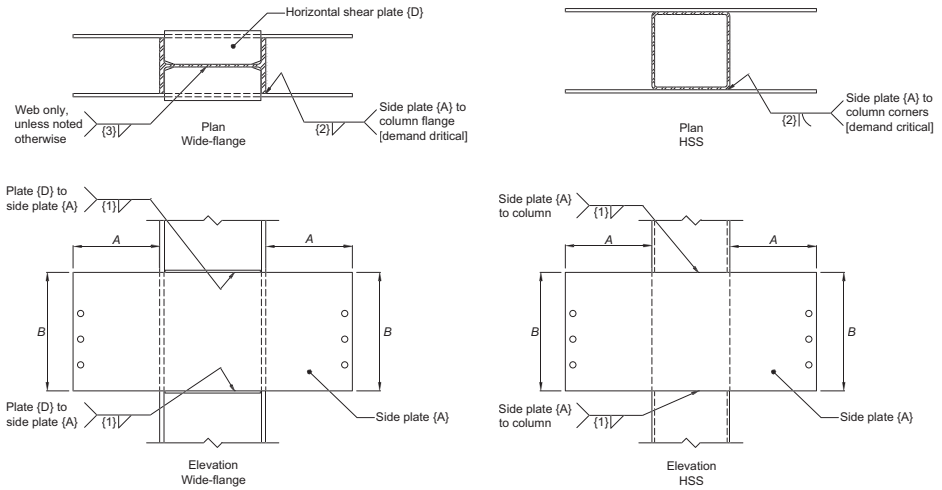


Fig. 11.15. Two-sided SidePlate moment connection (B-type), column shop detail.

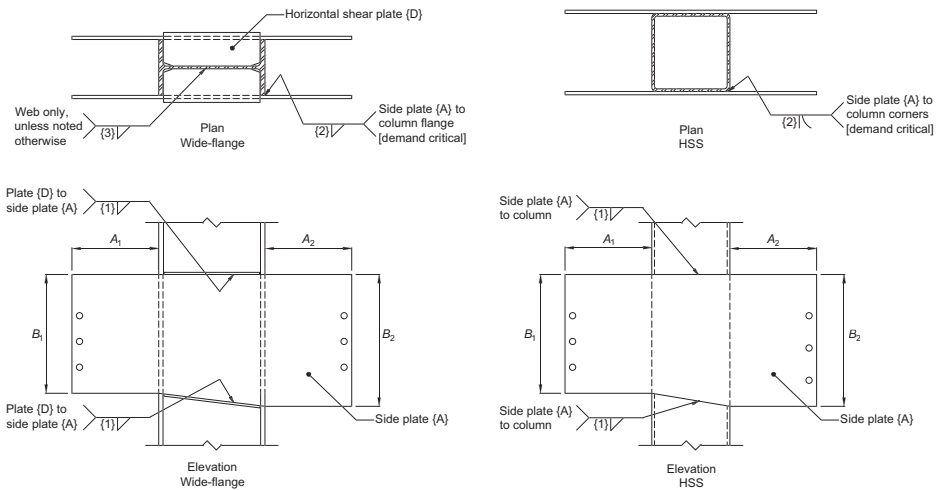


Fig. 11.16. Two-sided SidePlate moment connection (C-type), column shop detail.

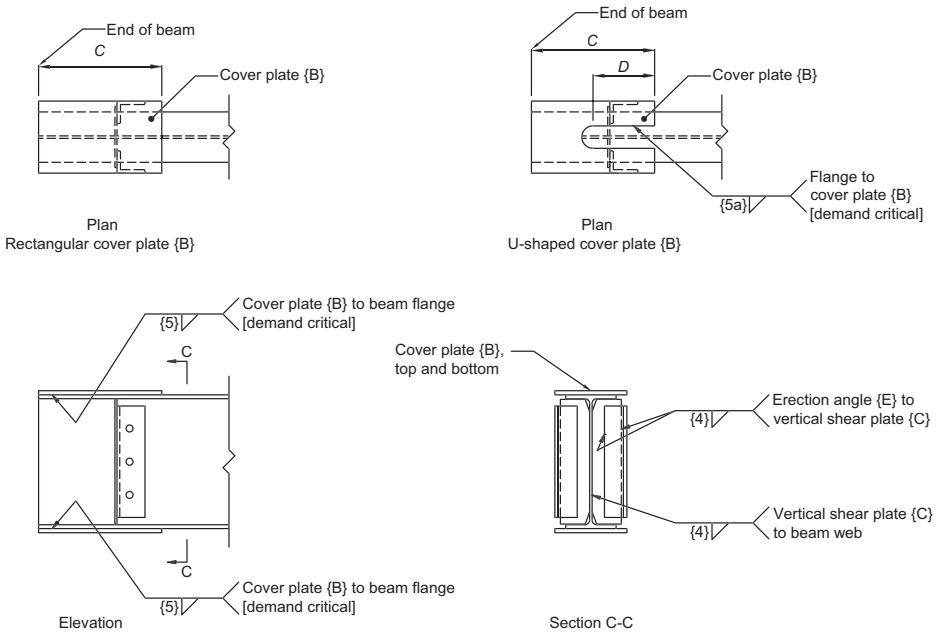


Fig. 11.17. Beam shop detail (field welded).

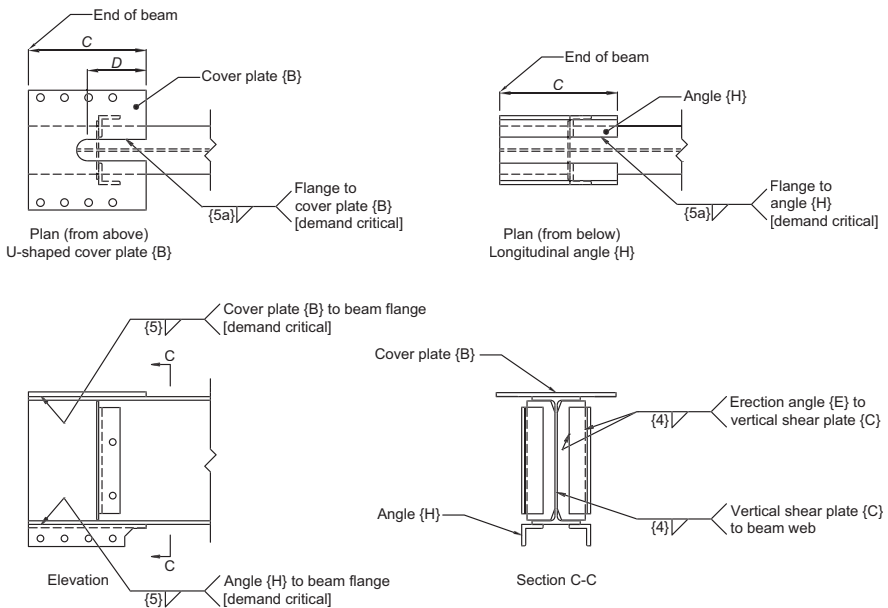


Fig. 11.18. Beam shop detail, field-bolted standard (configuration A).

For SMF systems, check that the section combinations satisfy the preliminary column-beam moment ratio given by

$$\sum(F_{yc}Z_{xc}) > 1.7 \sum(F_{yb}Z_{xb}) \tag{11.7-1}$$

where

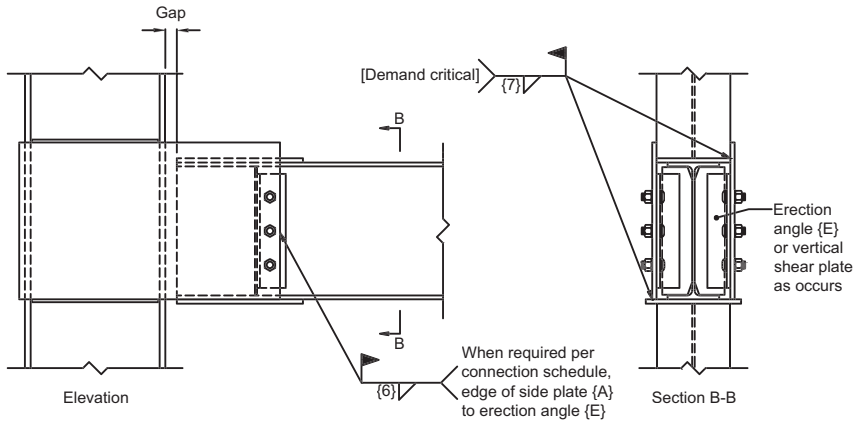
F_{yb} = specified minimum yield stress of beam, ksi (MPa)

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

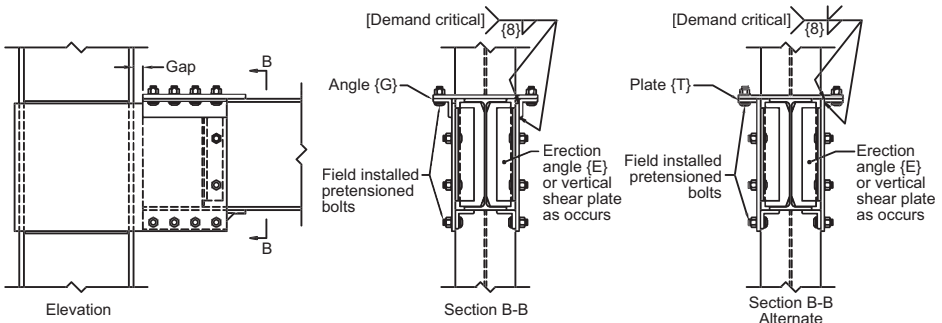
Z_{xb} = plastic section modulus of beam about x -axis, in.³ (mm³)

Z_{xc} = plastic section modulus of column about x -axis, in.³ (mm³)

Step 2. Approximate the effects on global frame performance of the increase in lateral stiffness and strength of the SidePlate moment connection due to beam hinge location and side plate stiffening in the mathematical elastic steel frame computer model by using a 100% rigid offset in the panel zone and by increasing the moment of



(a) Elevation and Section B-B, field welded



(b) Elevation and Section B-B, field-bolted standard (configuration A)

Fig. 11.19. Beam-to-SidePlate field erection detail.

inertia, elastic section modulus, and plastic section modulus of the beam to approximately three times that of the beam for a distance of approximately 77% of the beam depth beyond the column face (approximately equal to the extension of the side plate beyond the face of the column), illustrated in Figure 11.22.

SMF beams that have a combination of shallow depth and heavy weight (i.e., beams with a relatively large flange area such as those found in the widest flange series of a particular nominal beam depth) require that the extension of the side plate {A} be increased up to the nominal depth of the beam, d , for field-welded connections and $1.7d$ for field-bolted connections.

User Note: This increase in extension of side plate {A} of the field-welded connection lengthens fillet weld {7}, thus limiting the extremes in the size of fillet weld {7}. Regardless of the extension of the side plate {A}, the plastic hinge occurs at a distance of $d/3$ and $d/6$ from the end of the side plates for the field-welded and field-bolted connections, respectively.

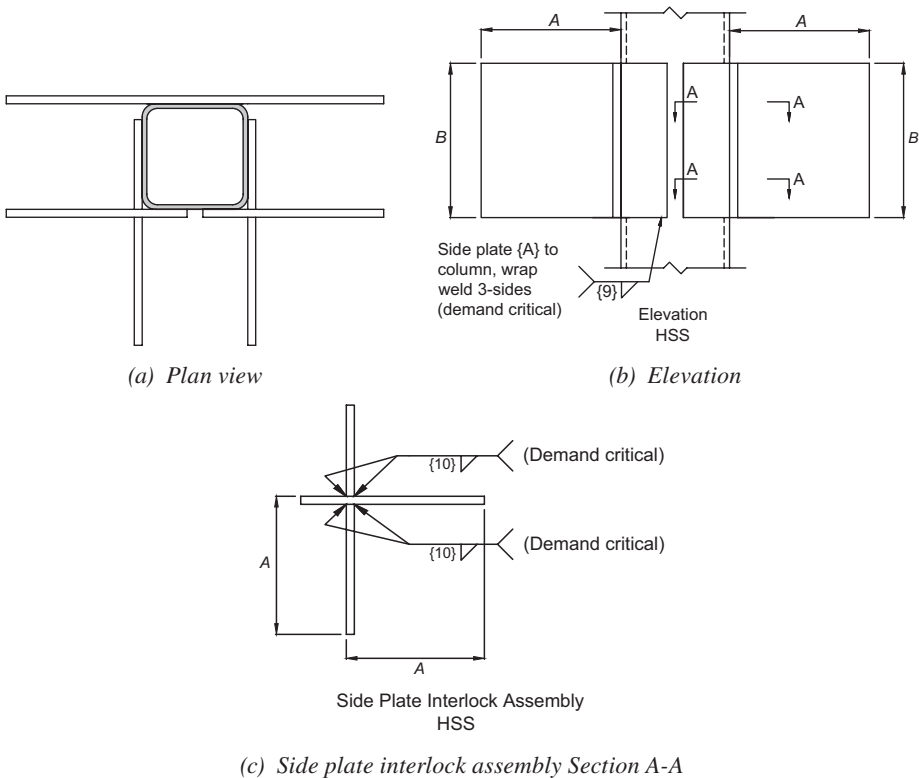


Fig. 11.20. Biaxial HSS column assembly shop detail.

Step 3. Confirm that the frame beams and columns satisfy all applicable building code requirements, including, but not limited to, stress or strength checks and design story drift checks.

Step 4. Confirm that the frame beam and column sizes comply with prequalification limitations in accordance with Section 11.3.

Step 5. Upon completion of the preliminary and/or final selection of lateral load-resisting frame beam and column member sizes using SidePlate connection technology, the engineer of record submits a computer model to SidePlate Systems Inc. In addition, the engineer of record shall submit the following additional information, as applicable:

$V_{gravity}$ = factored gravity shear in moment frame beam resulting from the load combination of $1.2D + f_1L + 0.15S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

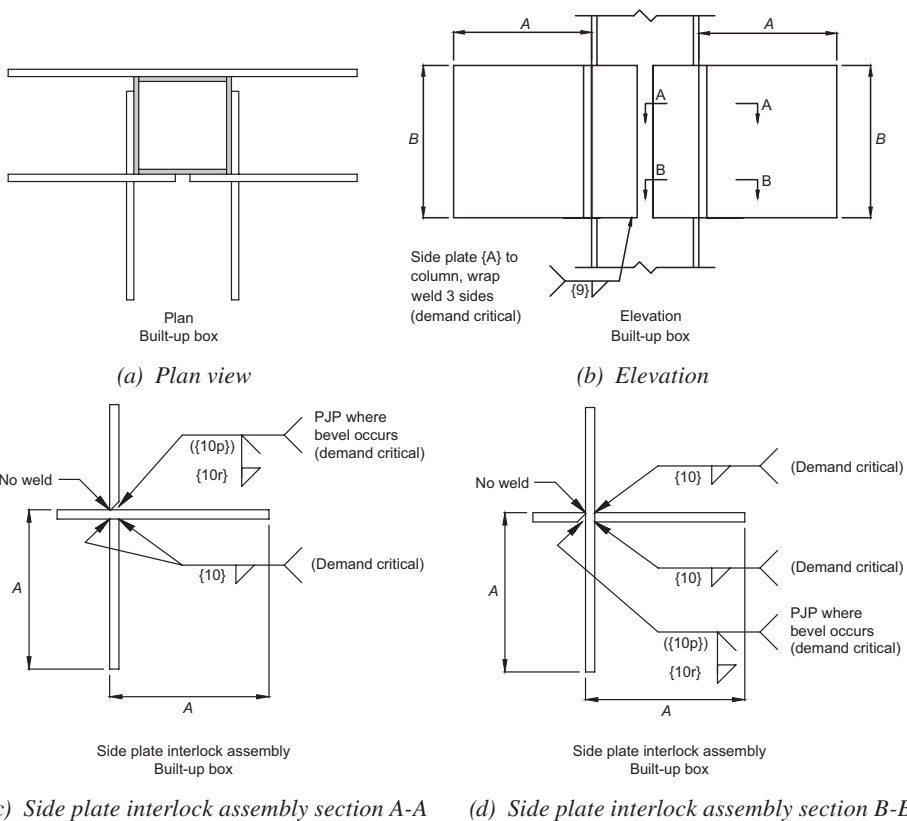


Fig. 11.21. Biaxial built-up box column assembly shop detail.

User Note: The load combination of $1.2D + f_1L + 0.15S$ is in conformance with ASCE/SEI 7-22. When using the 2015 International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.15 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

- (a) Factored gravity shear loads, V_1 and/or V_2 , from gravity beams that are not in the plane of the moment frame, but connect to the exterior face of the side plate(s),

where

V_1, V_2 = beam shear force resulting from the load combination of $1.2D + f_1L + 0.15S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

- (b) Factored gravity loads, M_{cant} and V_{cant} , from cantilever gravity beams that are not in the plane of the moment frame, but connect to the exterior face of the side plate(s),

where

M_{cant} = cantilever beam moment resulting from code applicable load combinations, kip-in. (N-mm)

V_{cant} = cantilever beam shear force resulting from code applicable load combinations, kips (N)

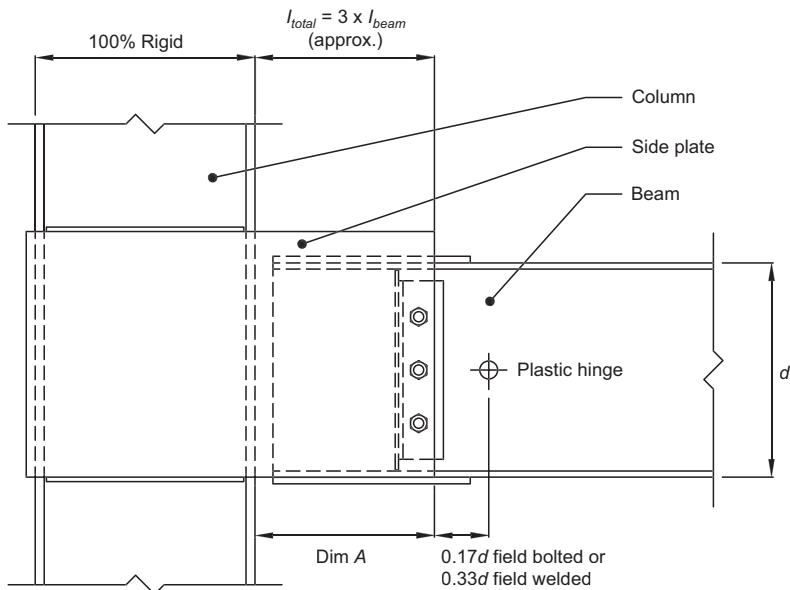


Fig. 11.22. Modeling of component stiffness for linear-elastic analysis.

User Note: Code applicable load combinations may need to include the following when looking at cantilever beams: $1.2D + f_1L + 0.15S$ and $(1.2 + 0.2S_{DS})D + \rho Q_E + f_1L + 0.15S$, which are in conformance with ASCE/SEI 7-22. When using the 2015 International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.15 for S (snow) when the roof configuration is such that it does not shed snow off of the structure.

- (c) Perpendicular amplified seismic lateral drag or chord axial forces, A_{\perp} , transferred through the SidePlate connection,

where

A_{\perp} = amplified seismic drag or chord force resulting from the applicable building code, kips (N)

User Note: Where linear-elastic analysis is used to determine perpendicular collector or chord forces used to design the SidePlate connection, such forces should include the applicable load combinations specified by the building code, including consideration of the amplified seismic load, Ω_0 . Where nonlinear analysis or capacity design is used, collector or chord forces determined from the analysis are used directly, without consideration of additional amplified seismic load.

- (d) In-plane factored chord axial forces, A_{\parallel} , transferred along the frame beam through the SidePlate connection,

where

A_{\parallel} = amplified seismic chord force resulting from applicable building code, kips (N)

Step 6. Upon completion of the mathematical model review and after additional information has been supplied by the engineer of record, SidePlate engineers provide project-specific connection designs. Strength demands used for the design of critical load-transfer elements (plates, welds, and columns) throughout the SidePlate beam-to-column connection and the column are determined by superimposing maximum probable moment, M_{pr} , at the known beam hinge location, then amplifying the moment demand to each critical design section based on the span geometry, as shown in Figure 11.9, and including additional moment due to gravity loads. For each of the design elements of the connection, the moment demand is calculated using Equation 11.7-2, and the associated shear demand is calculated as

$$M_{group} = M_{pr} + V_h x \quad (11.7-2)$$

where

M_{pr} = maximum probable moment at the plastic hinge as defined in Section 2.4.3,

Equation 2.4-1, kip-in. (N-mm).

V_h = maximum shear demand from probable maximum moment and factored gravity loads as defined in Section 2.4.4, Equation 2.4-3, kips (N).

x = distance from plastic hinge location to centroid of connection element, in. (mm)

When calculating M_{pr} , the value of C_{pr} is provided by SidePlate as part of the connection design.

User Note: The value of C_{pr} for SidePlate connections is determined by testing and nonlinear analysis and ranges from 1.15 to 1.35.

Step 7. SidePlate designs all connection elements according to the proprietary connection design procedures contained in the SidePlate Connection Design Software (version 16 for field-welded and version 17 for field-bolted connections). The version is clearly indicated on each page of calculations. The final design includes structural notes and details for the connections.

User Note: The procedure uses an ultimate strength design approach to size plates and welds, incorporating strength, plasticity, and fracture limits. For welds, an ultimate strength analysis incorporating the instantaneous center of rotation may be used as described in AISC *Specification* Section J2.4b. For bolt design, eccentric bolt group design methodology incorporating ultimate strength of the bolts is used. Refer to the Commentary for an in-depth discussion of the process.

In addition to the column web panel zone strength requirements, the column web shear strength shall be sufficient to resist the shear loads transferred at the top and bottom of the side plates. The design shear strength of the column web shall be determined in accordance with AISC *Specification* Section G2.1.

Step 8. Engineer of record reviews SidePlate calculations and drawings to ensure that all project-specific connection designs have incorporated the information provided in Step 5.

CHAPTER 12

SIMPSON STRONG-TIE STRONG FRAME MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by patent rights. By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license. The statement may be obtained from the standard's developer.*

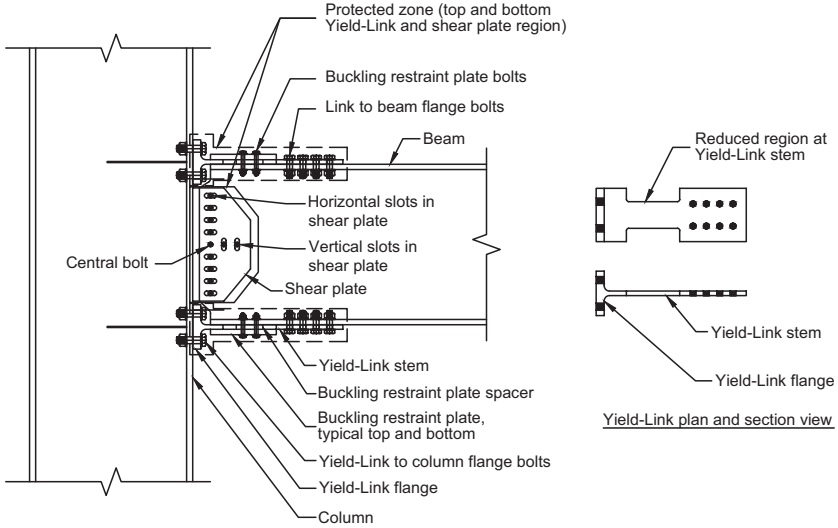
12.1. GENERAL

The Simpson Strong-Tie® (SST) Strong Frame® moment connection is a partially restrained (Type PR) connection that uses a modified shear plate connection (single-plate shear connection) for shear transfer and a modified T-stub or end-plate connection (the Yield-Link® structural fuse) for moment transfer, as shown in Figures 12.1(a) and 12.1(b). The central bolt in the vertical line of bolts in the shear plate uses a standard bolt hole and defines the center of rotation for the joint, while the rest of the bolt holes are slotted to allow rotation of the beam around the central bolt. Matching holes in the beam web are all standard holes. This prevents moment transfer through the shear plate connection. The central bolt and all horizontally slotted shear plate bolts participate in shear resistance. The central bolt is also designed, together with the additional horizontally aligned bolts, to resist the axial force in the beam at the connection. The modified T-stub and end-plate link connections, which bolt to both the beam flange and column flange, are configured as yielding links and contain a reduced yielding area in the stem of the link that is prevented from buckling in compression via a separate buckling restraint plate. The connection is based on a capacity-based design approach, wherein connection response remains elastic under factored load combinations, and seismic inelastic rotation demand is confined predominantly within the connection with little, if any, inelastic behavior expected from the members.

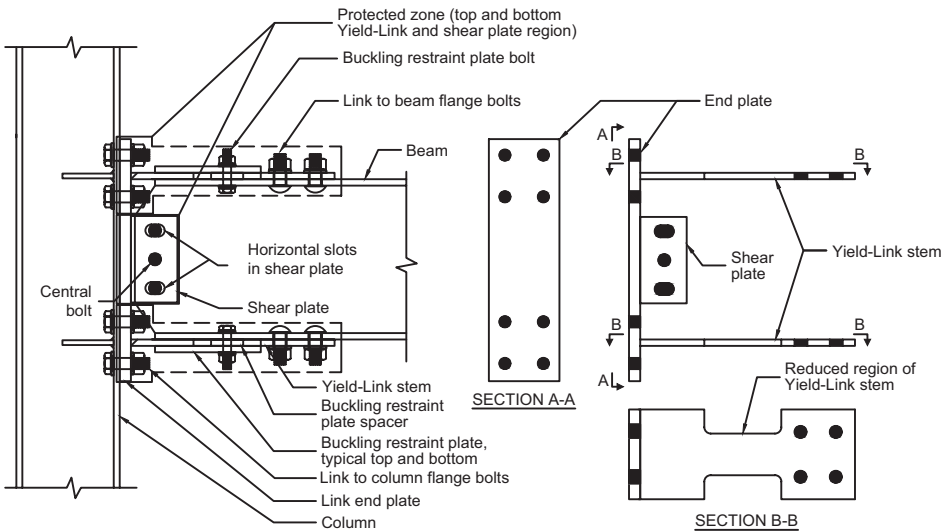
*The proprietary design of the Yield-Link structural fuse and its use in moment-resisting connections is protected under U.S. Patent Nos. 8,375,652; 8,001,734; 8,763,319; European Patent Nos. 1936053; 2468986; Japanese Pat. Nos. 5398980; 6001856; Chinese Pat. No. ZL200710301531.4; Canadian Patent No. 2616474; Australian Patent Nos. 2007254648; 2014274605; Korean Patent No. 10-1460258; Great Britain Patent Nos. 1936053; 2468986; German Patent Nos. 1936053; 602011054470.5; French Patent Nos. 1936053; 2468986. Other U.S and foreign patent protection are pending.

12.2. SYSTEMS

The Simpson Strong-Tie connection is prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions.



(a) T-stub Yield-Link



(b) End-plate Yield-Link

Fig. 12.1. Simpson Strong-Tie Strong Frame moment connection.

Exception: Simpson Strong-Tie connections with concrete structural slabs are only prequalified if the concrete slab is isolated in accordance with Section 2.3.4.

12.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or welded built-up I-shaped members.
- (2) Beam depth is limited to
 - (a) For T-stub Yield-Links, a maximum of W36 (W920) for rolled shapes. Beam depth for built-up members shall not exceed the maximum depth of the permitted W36 (W920) shapes.
 - (b) For end-plate Yield-Links, minimum W8 (W200) to maximum W12 (W310) for rolled shapes. Beam depth for built-up members shall comply with the minimum and maximum depths permitted for W8 (W200) to W12 (W310) sections.
- (3) There are no limits on the beam web width-to-thickness ratio beyond those listed in the AISC *Specification*. The beam flange width-to-thickness ratio shall not exceed λ_r as defined in AISC *Specification* Table B4.1b. Flange thickness shall be designed in accordance with Step 10 in the Design Procedure and shall not be less than 0.40 in. (10 mm).
- (4) Lateral bracing of beams and joints: there are no requirements for stability bracing of beams or joints beyond those in the AISC *Specification*.
- (5) The protected zone shall consist of the Yield-Links, the shear plate, and the portions of the beam in contact with the Yield-Links and shear plate.

User Note: Limits on beam weight and span-to-depth ratio are not required for the SST moment connection because plastic hinging in the connection occurs solely within the Yield-Links. Span-to-depth ratio is typically limited to control moment gradient and beam shear, both of which are limited by the shear plate connection within the design procedure.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled wide-flange shapes, built-up I-shapes, or flanged cruciform sections permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) Column depth is limited to a maximum of W36 (W920) for rolled shapes. Column depth for built-up members shall not exceed the maximum depth permitted for W36 (W920) shapes.
- (4) There is no limit on the weight per foot of columns.

- (5) There are no additional requirements for flange thickness.
- (6) Column width-to-thickness ratios shall comply with the following:
 - (a) Where column-to-foundation connections are designed to restrain column end rotation, column width-to-thickness ratios shall comply with AISC *Seismic Provisions* Table D1.1 for highly ductile members within the first story.
 - (b) At other locations and for other conditions, column width-to-thickness ratios shall comply with the AISC *Specification*.
- (7) Lateral bracing of columns shall be provided in accordance with the AISC *Seismic Provisions*.

Exception: When columns are designed in accordance with Section 12.9 and the maximum nominal flexural strength, M_n , outside the panel zone is limited such that $M_n \leq F_y S_x$, it is permitted that bracing be provided at the level of the top flange of the beam only.

3. Bolting Limitations

Bolts shall conform to the requirements of Chapter 4.

The following exceptions apply:

- (1) The following connections shall be made with ASTM F3125/F3125M Grade A325 or A325M bolts installed either as snug-tight or pretensioned, except as noted. For pretensioned applications, ASTM F3125/F3125M Grade F1852 bolts are permitted.
 - (a) Yield-Link flange or end plate-to-column flange bolts
 - (b) Buckling restraint plate bolts installed snug tight
 - (c) Shear-plate bolts
- (2) The Yield-Link stem-to-beam flange bolts shall be pretensioned ASTM F3125/F3125M Grades A325, A325M, A490, A490M, F1852, or F2280 bolt assemblies. Faying surface preparation between the Yield-Link stem and beam flange is not required, but faying surfaces shall not be painted.

12.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam connection-to-column connections shall satisfy the following limitations:

- (1) Panel zones shall conform to the requirements of the AISC *Specification*.
- (2) Column-beam connection moment ratios shall be limited as follows:
 - (a) For SMF systems, the column-beam connection moment ratio shall conform to the requirements of the AISC *Seismic Provisions*. The value of $\sum M_{pb}^*$ shall be taken equal to $\sum (M_{pr} + M_{uw})$, where M_{pr} is calculated according to Equation 12.9-28, and M_{uw} is the additional moment due to shear amplification from the center of the vertical line of bolts in the shear plate to the centerline of the column. M_{uw} is calculated as $V_u (a + d_c/2)$, where V_u is the

shear at the shear-plate connection calculated in Section 12.9, Step 12, a is the distance from the centerline of the shear-plate shear bolts to the face of the column as shown in Figure 12.3(c), and d_c is the depth of the column.

- (b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*.

12.5. CONTINUITY PLATES

Continuity plates shall satisfy the following limitations:

- (1) The need for continuity plates shall be determined in accordance with Section 12.9.
- (2) Where required, design of continuity plates shall be in accordance with the AISC *Specification*.
- (3) Continuity plates may be welded to the column flange and column web with fillet welds.

12.6. YIELD-LINK FLANGE-TO-STEM WELD LIMITATIONS

Yield-Link flange-to-stem connections may be CJP groove welds or double-sided fillet welds.

- (1) CJP groove welds shall conform to the requirements of demand critical welds in the AISC *Seismic Provisions*.
- (2) Double-sided fillet welds shall be designed to develop the tensile strength of the unreduced Yield-Link stem at the column side, $b_{col-side}$, and shall be demand critical.

12.7. FABRICATION OF YIELD-LINK CUTS

The reduced section of the Yield-Link shall be cut using the following methods: laser, plasma, or water-jet method. Maximum roughness of the cut surface shall be 250 μin . (6.25 μm) in accordance with ASME B46.1 (ASME, 2019). All transitions between the reduced section of the Yield-Link and the nonreduced sections of the Yield-Link shall utilize a smooth radius, R , as shown in Figure 12.2(a), where R equals the thickness of the link stem, t_{stem} .

Cutting tolerance at the reduced section shall be plus or minus $\frac{1}{16}$ in. (2 mm) from the theoretical cut line.

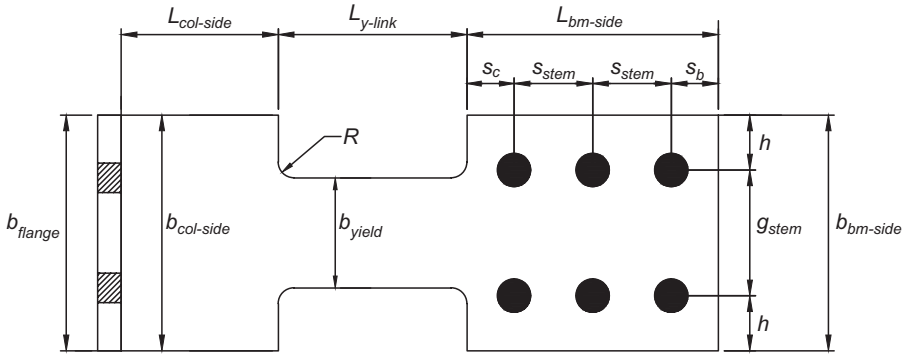
12.8. CONNECTION DETAILING

1. Beam Coping

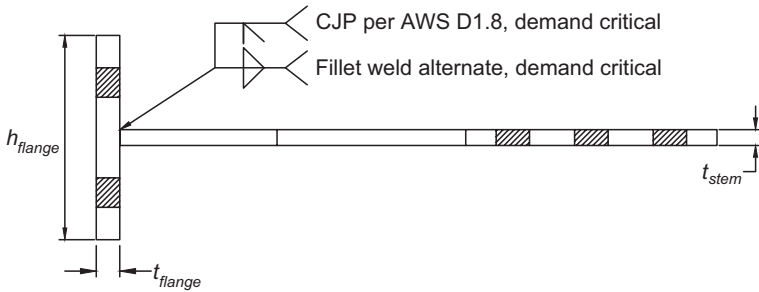
Beams shall be coped in accordance with Figure 12.3(a).

2. Yield-Links

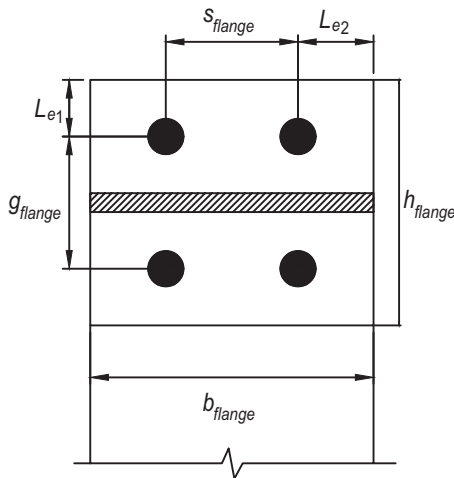
Yield-Links shall conform to the requirements of Figures 12.2 and 12.3 and shall be fabricated using ASTM A572/A572M Grade 50 (Grade 345) material or rolled



(a) Yield-Link plan view



(b) Yield-Link elevation view



(c) Yield-Link flange view

Fig. 12.2. Yield-Link geometries.

sections conforming to the ASTM A992/A992M or ASTM A913/A913M Grade 50 (Grade 345) specification. Each pair of Yield-Link stems at a connection shall be cut from the same heat of material. Minimum Yield-Link stem thickness shall be $\frac{1}{2}$ in. (13 mm), and maximum Yield-Link stem thickness shall be 1 in. (25 mm), with a thickness tolerance in accordance with ASTM A6/A6M. Yield-Link flange edge distances, L_{e1} and L_{e2} , shall conform to AISC *Specification* Tables J3.4 or J3.4M.

3. Shear-Plate Connection Bolts

Shear-plate connection bolts shall be designed to resist the required axial and shear forces; see Figure 12.1 and Section 12.9, Step 15.

4. Shear-Plate Shear Connection Welds

The single-shear plate connection shall be welded to the column flange or end plate using double-sided fillet welds, PJP groove welds, or CJP groove welds, sized in accordance with Section 12.9, Step 15.4.

5. Bolt Hole Requirements

- (1) Standard bolt holes shall be provided in the beam flanges and beam webs. Oversized holes or vertical slots are permitted in the column flanges with T-stub Yield-Links. Standard size bolt holes shall be used in the end plate and column flanges when using end-plate Yield-Links.
- (2) The central bolt hole in the shear plate shall be a standard hole. Remaining bolt holes for bolts resisting shear and axial forces shall be slotted to accommodate a connection rotation of at least 0.07 rad.

6. Buckling Restraint Assembly

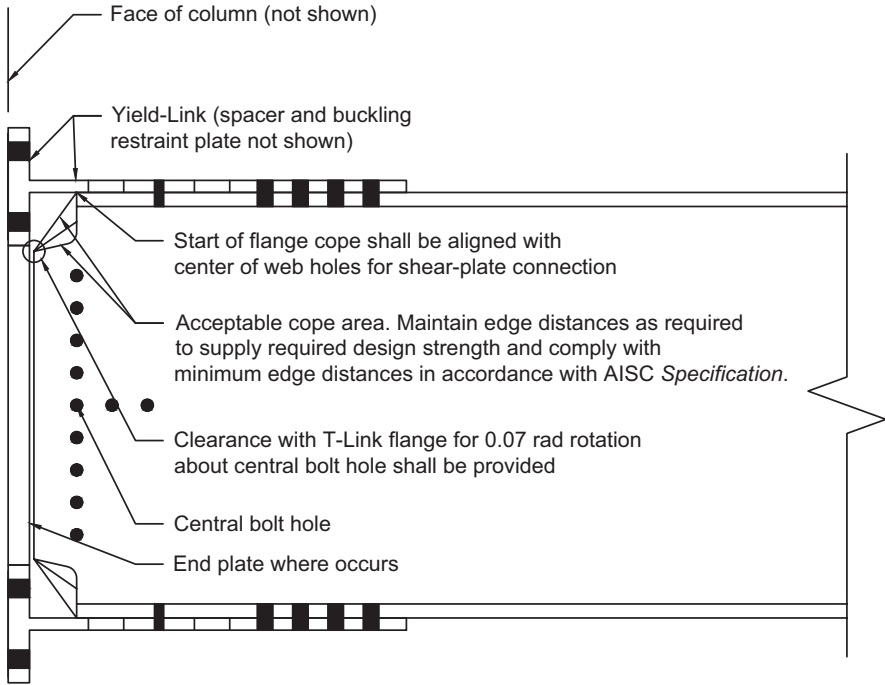
The buckling restraint assembly consists of the buckling restraint plate, the buckling restraint spacer plate, and the buckling restraint bolts and shall conform to the requirements of Figure 12.3. Design of the buckling restraint plate assembly shall be in accordance with Section 12.9, Step 10. The buckling restraint plate shall be a minimum of $\frac{7}{8}$ in. (22 mm) thick, with a specified minimum yield stress $F_y \geq 50$ ksi (345 MPa). The buckling restraint plate shall extend from the centerline of the vertical shear bolt holes to the end of the cut region of the Yield-Link plate. The buckling restraint spacer plate shall have the same thickness as the Yield-Link stem, with a specified minimum yield stress $F_y \geq 36$ ksi (250 MPa). Buckling restraint bolts shall have a minimum diameter of $\frac{5}{8}$ in. (16 mm).

7. Shims

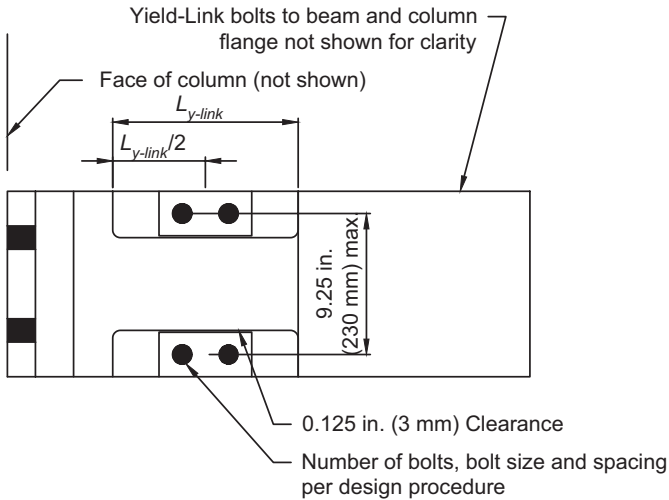
The use of finger shims at the T-stub or end-plate Yield-Link flange-to-column flange is permitted, subject to the limitations of the RCSC *Specification*.

12.9. DESIGN PROCEDURE

Step 1. Choose trial values for the beam sections and column sections subject to the prequalification limits of Section 12.3 assuming fully restrained beam-to-column



(a) Beam coping



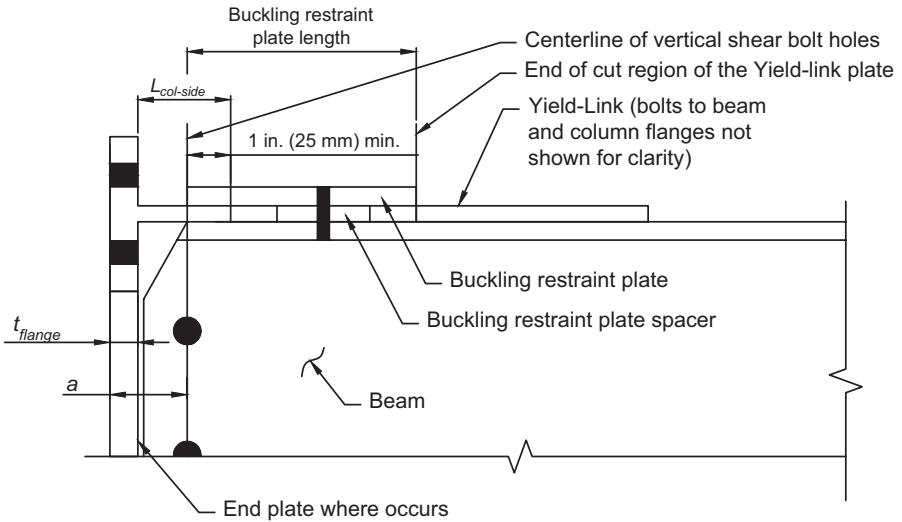
(b) Buckling restraint spacer plate placement

Fig. 12.3(a-b) Connection detailing.

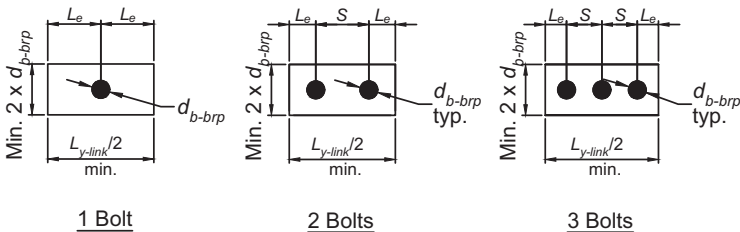
connections and all load combinations specified by the applicable building code. Estimate the design story drift for compliance with the applicable limits specified by the applicable building code as 1.2 times larger than the value calculated assuming fully restrained connections.

Step 2. Check the strength and deflection of the beam assuming the beam is simply supported between shear-plate connections. Check the beam strength for the applicable vertical load combinations of the applicable building code. Check that the deflection of the beam under dead and live loads is less than $L_h/360$, where L_h is the length of the beam between the center of the shear-plate shear bolts at each end of the beam.

User Note: The deflection check serves to estimate beam stiffness needed to limit member end rotations. Other values may be acceptable.



(c) Buckling restraint plate and Yield-Link $L_{col-side}$ limitations



(d) Buckling restraint spacer plate dimensions

Fig. 12.3(c-d). Connection detailing.

Step 3. Estimate the required Yield-Link yield strength from Step 1 and then calculate the required Yield-Link area.

$$P'_{y-link} = M_u / (\phi_b d) \quad (12.9-1)$$

$$A'_{y-link} = P'_{y-link} / F_{y-link} \quad (12.9-2)$$

where

F_{y-link} = specified minimum yield stress of Yield-Link stem material, ksi (MPa)

M_u = moment demand from elastic analysis assuming fully restrained connections, kip-in. (N-mm)

d = depth of beam, in. (mm)

ϕ_b = 0.90

Step 4. Determine the nonreduced width and length of the Yield-Link at column side; see Figure 12.2(a).

Step 4.1. Determine nonreduced Yield-Link stem widths, $b_{col-side}$ and $b_{bm-side}$.

User Note: Try setting $b_{col-side}$ and $b_{bm-side}$ equal to the minimum of beam flange width and column flange width, respectively.

Step 4.2. Nonreduced Yield-Link stem length at column side, $L_{col-side}$, shall have a maximum length equal to 5 in. (125 mm) and a minimum length equal to $a - t_{flange} + 1$ in. ($a - t_{flange} + 25$ mm). See Figure 12.3(c).

Step 5. Determine the width of the yielding section of the Yield-Link stem, b_{yield} , where the thickness of the Yield-Link stem, t_{stem} , shall be taken as $\frac{1}{2}$ in. (13 mm) minimum and 1 in. (25 mm) maximum.

$$b_{yield, req'd} \geq A'_{y-link} / t_{stem} \quad (12.9-3)$$

The value of $b_{yield, req'd}$ shall not exceed the least of $0.5b_{col-side}$, $0.5b_{bm-side}$, or 6 in. (150 mm).

Step 6. Determine the minimum yielding length of the Yield-Link stem, L_{y-link} , such that the axial strain in the straight portion of the Yield-Link is less than or equal to 0.085 in./in. at 0.05 rad of connection rotation.

$$L_{y-link} = \frac{0.05}{0.085} \left(\frac{d + t_{stem}}{2} \right) + 2R \quad (12.9-4)$$

Step 7. Compute the expected yield strength and probable maximum tensile strength of the Yield-Link.

$$P_{y-link} = A_{y-link} R_y F_{y-link} \quad (12.9-5)$$

$$P_{r-link} = A_{y-link} R_t F_{u-link} \quad (12.9-6)$$

where

A_{y-link} = area of reduced Yield-Link section (b_{yield})(t_{stem}), in.² (mm²)

F_{u-link} = specified minimum tensile strength of Yield-Link stem material, ksi (MPa)

R_t = ratio of expected tensile strength to specified minimum tensile strength, F_{tu} , as related to overstrength in material yield stress, R_y ; taken as 1.2 for Yield-Link stem material

R_y = ratio of the expected yield stress to specified minimum yield stress, F_y ; taken as 1.1 for Yield-Link stem material

Step 8. Determine the nonreduced width, $b_{bm-side}$, and length, $L_{bm-side}$, at beam side of the Yield-Link using P_{r-link} from Step 7.

Step 8.1. Design bolts for shear transfer between the Yield-Link stem and the beam flange according to the AISC *Specification* and determine the bolt diameter, d_{b-stem} .

Step 8.2. Determine the nonreduced width of the Yield-Link stem on the beam side, $b_{bm-side}$.

User Note: Try setting $b_{bm-side}$ equal to $b_{col-side}$ from Step 4.1.

Step 8.3. Compute the nonreduced length of the Yield-Link stem at beam side, $L_{bm-side}$:

$$L_{bm-side} = s_c + [(n_{rows} - 1)s_{stem}] + s_b \quad (12.9-7)$$

where

n_{rows} = number of rows of bolts from Step 8.1.

s_b = distance from center of last row of bolts to beam-side end of Yield-Link stem, from AISC *Specification* Table J3.4 or Table J3.4M, in. (mm)

s_c = distance from center of first row of bolts to reduced section of Yield-Link, from AISC *Specification* Table J3.4 or Table J3.4M, in. (mm)

s_{stem} = spacing between rows of bolts for Yield-Link stem-to-beam flange connection, minimum $2\frac{2}{3}d_{b-stem}$, in. (mm)

Step 8.4. Check the Yield-Link stem at the beam side for tensile yielding, tensile rupture, block shear rupture, and bolt bearing (where deformation at the hole is a design consideration) according to the AISC *Specification*. Check the beam flange for bolt bearing (where deformation at the bolt hole is a design consideration) and block shear rupture according to the AISC *Specification*.

Step 9. Design the Yield-Link flange-to-column flange or end-plate connection using P_{r-link} from Step 7.

Step 9.1. Design bolts for tension force transfer between the Yield-Link flange or end plate and the column flange according to the AISC *Specification* and

determine the diameter of the flange bolts, $d_{b-flange}$. The required tensile force per bolt in the Yield-Link flange-to-column flange connection, r_t , is:

For T-stub Yield-Link

$$r_t = \frac{P_{r-link}}{4} \quad (12.9-8)$$

For end-plate Yield-Link

$$r_t = \frac{M_{pr}}{2(h_1 + h_2)} + \frac{V_u a}{2h_2} \quad (12.9-9)$$

where h_1 and h_2 are defined in Table 6.2 of Chapter 6, Bolt Unstiffened (BUEEP) and Stiffened Extended End-Plate (BSEEP) Moment Connections.

Step 9.1a. For end-plate Yield-Link connections, check bolt shear rupture strength of the connection provided by bolts at the compression flange only using Equation 6.7-11 in Chapter 6.

Step 9.2. Compute the thickness of the Yield-Link flange, t_{flange} , required to prevent prying action:

$$t_{flange} = \sqrt{\frac{4r_t b'}{p\phi_d F_u}} \quad (12.9-10)$$

$$b' = (b - d_{b-flange}/2) \quad (12.9-11)$$

where

b = vertical distance from centerline of bolts in Yield-Link flange to face of Yield-Link stem, in. (mm)

$d_{b-flange}$ = diameter of bolt connecting Yield-Link flange and column flange, in. (mm)

p = minimum of $b_{flange}/2$ or s_{flange} , in. (mm)

Step 9.3. Check the thickness of the Yield-Link flange, t_{flange} , for shear yielding and shear rupture according to the AISC *Specification*.

Step 9.3a. For end-plate Yield-Link connections, check shear yielding and shear rupture of the extended portion of the end plate using Equations 6.7-7 and 6.7-8 in Chapter 6.

Step 9.3b. For end-plate Yield-Link connections, check bolt bearing and tearout failure of the end plate and column flange using Equation 6.7-12 in Chapter 6.

Step 9.4. Design the stem-to-flange weld of the Yield-Link as either a CJP weld or a double-sided fillet weld that will develop the tensile strength of the Yield-Link at the column side, P_{r-weld} :

$$P_{r-weld} = b_{col-side} t_{stem} R_t F_u - link \quad (12.9-12)$$

Step 10. Design the buckling restraint assembly.

Step 10.1. Determine the minimum thickness of the buckling restraint plate (BRP) to prevent yielding during compression of the link stem. BRP thickness shall not be less than $\frac{7}{8}$ in. (22 mm).

$$t_{BRP_min} = 0.51 \sqrt{\frac{(L_{cant})(P_{r-link})}{(F_{y_BRP})(R_{y_BRP})b_n}} \quad (12.9-13)$$

where

F_{y_BRP} = specified minimum yield stress of BRP material, ksi (MPa)

L_{cant} = lever arm from start of reduced region to edge of spacer plate bolt hole, plus plate stretch from 0.05 rad of rotation, in. (mm). See Figure 12.4(a).

R_{y_BRP} = ratio of the expected yield stress to specified minimum yield stress, F_{y_BRP} , taken as 1.1 for BRP material

b_n = net width of BRP, in. (mm)

Step 10.2. Determine minimum beam flange thickness to prevent yielding and BRP bolt induced prying, t_{bf_min} . Minimum flange thickness shall not be less than 0.4 in.

$$t_{bf_min} = \sqrt{\frac{4T_{ux}b'}{\phi_d P_e F_{ub}}} \quad (12.9-14)$$

where

F_{ub} = specified minimum tensile strength of the beam material, ksi (MPa)

T_{ux} = vertical thrust force transferred by one restraint bolt, kips (kN)

$$= \frac{Q}{n_{BRP_bolts}} \quad (12.9-15)$$

Q = total vertical thrust force on the beam flange, kips (kN)

$$= N_{design} Q_i \quad (12.9-16)$$

n_{BRP_bolts} = total number of BRP bolts

N_{design} = number of contact points between the reduced region of the link stem and the BRP or beam flange (rounded to the nearest integer)

$$= \left(\frac{1}{2} \right) \left\lfloor \frac{L_{y-link}}{l_o} \right\rfloor \quad (12.9-17)$$

l_o = effective buckling wave length [see Fig. 12.4(b)]:

$$= \sqrt{\frac{\pi^2 E_t I_y}{P_{r-link}} \left[1 + \left(\frac{b_{yield}}{2g} + 1.013 \right)^{-1} \right]} \quad (12.9-18)$$

E_t = tangent modulus of elasticity

= 193 ksi (1330 MPa)

I_y = weak axis moment of inertia of reduced link stem region, in.⁴ (mm⁴)

g = gap increase due to transverse shortening of the Yield-Link thickness,
in. (mm)

$$= 0.25\epsilon_{target}t_{stem} \tag{12.9-19}$$

$$\epsilon_{target} = \frac{0.04\left(\frac{d + t_{stem}}{2}\right)}{L_{y-link} + 2R} \tag{12.9-20}$$

$$Q_i = \frac{4gP_{r-link}}{l_o} \tag{12.9-21}$$

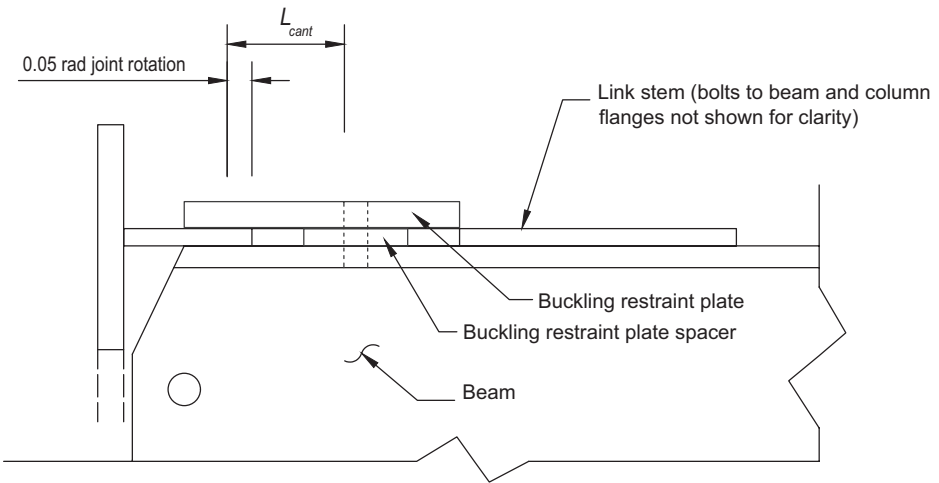
b' = distance from the bolt centerline to the beam centerline, in. (mm)
 p_e = effective (tributary) length per bolt from the yield line pattern, in. (mm)

Step 10.3. Determine the BRP bolt size and quantity.

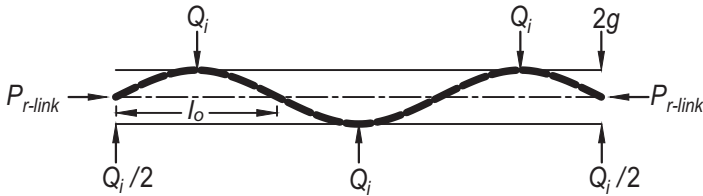
Design BRP bolts for tension + shear interaction for out-of-plane thrust according to the AISC *Specification*, where T_{ux} is from Step 10.2 and the shear is

$$V_{ux} = \mu_k T_{ux} \tag{12.9-22}$$

where μ_k is the coefficient of dry kinetic friction, taken as 0.3.



(a) BRP cantilever length



(b) Buckling wavelength and thrust forces

Fig. 12.4. Buckling restraint assembly parameters.

Check the BRP bolts for the in-plane shear thrust force exerted on each spacer plate, V_{uy} , in the strong-axis direction:

$$V_{uy} = \frac{4cP_{r-link}}{\sqrt{\frac{\pi^2 E_t I_x}{P_{r-link}} \left[1 + \left(\frac{t_{stem}}{2c} + 1.013 \right)^{-1} \right]}} \quad (12.9-23)$$

where

I_x = strong-axis moment of inertia of reduced link stem region, in.⁴ (mm⁴)

c = clearance between the Yield-Link and the buckling restraint spacer plate
= 1/8 in. (3 mm)

Step 11. Verify the elastic frame drift and connection moment demand by accounting for actual connection stiffness.

Step 11.1. Model the connection using a pair of nonlinear axial links or a nonlinear rotational spring at each connection determined from the following properties:

K_1 = elastic axial stiffness contribution due to bending stiffness in Yield-Link flange, kip/in. (N/mm)

$$(0.75)(192)E \left(\frac{w_{col-side} t_{flange}^3}{12} \right) = \frac{g_{flange}^3}{g_{flange}^3} \quad (12.9-24)$$

K_2 = elastic axial stiffness contribution due to nonyielding section of Yield-Link, kip/in. (N/mm)

$$= \frac{t_{stem} b_{col-side} E}{L_{col-side} + s_c + l_v} \quad (12.9-25)$$

l_v = 0 when four or fewer bolts are used at Yield-Link-to-beam connection
= $s_{stem}/2$ when more than four bolts are used at Yield-Link-to-beam connection

K_3 = elastic axial stiffness contribution due to yielding section of Yield-Link, kip/in. (N/mm)

$$= \frac{t_{stem} b_{yield} E}{L_{y-link}} \quad (12.9-26)$$

K_{eff} = effective elastic axial stiffness of Yield-Link, kip/in. (N/mm)

$$= \frac{K_1 K_2 K_3}{(K_1 K_2 + K_2 K_3 + K_1 K_3)} \quad (12.9-27)$$

M_{pr} = probable maximum moment capacity of Yield-Link pair, kip-in. (N-mm)

$$= P_{r-link} (d + t_{stem}) \quad (12.9-28)$$

$$\begin{aligned}
 M_{ye-link} &= \text{expected yield moment of Yield-Link pair, kip-in. (N-mm)} \\
 &= P_{ye-link}(d + t_{stem}) \quad (12.9-29)
 \end{aligned}$$

$$\begin{aligned}
 n_{bolt} &= \text{number of bolts in Yield-Link stem-to-beam flange connection} \\
 \Delta_{0.04} &= \text{axial deformation in Yield-Link at a connection rotation of 0.04 rad} \\
 &= \frac{0.04(d + t_{stem})}{2} \quad (12.9-30)
 \end{aligned}$$

$$\begin{aligned}
 \Delta_{0.07} &= \text{axial deformation in Yield-Link at a connection rotation of 0.07 rad} \\
 &= \frac{0.07(d + t_{stem})}{2} \quad (12.9-31)
 \end{aligned}$$

$$\begin{aligned}
 \Delta_y &= \text{axial deformation in Yield-Link at expected yield, in. (mm)} \\
 &= \frac{P_{ye-link}}{K_{eff}} \quad (12.9-32)
 \end{aligned}$$

$$\begin{aligned}
 \theta_y &= \text{connection rotation at expected yield of Yield-Link, rad} \\
 &= \frac{\Delta_y}{0.5(d + t_{stem})} \quad (12.9-33)
 \end{aligned}$$

All other terms were previously defined or shown in Figure 12.2. Refer to Figure 12.5(a) for a plot of Yield-Link axial force versus Yield-Link axial deformation. Refer to Figure 12.5(b) for the moment versus rotation relationship required for the analysis and modeling of the SST moment connection.

Step 11.2. Considering the applicable drift limit and all applicable load combinations specified by the applicable building code, but not including the overstrength seismic load, verify that:

- The connection moment demand, M_u , is less than or equal to the connection available flexural strength, ϕM_n , taking ϕ as 0.90 and M_n as $M_{ye-link}/R_y$.
- The drift complies with applicable limits.

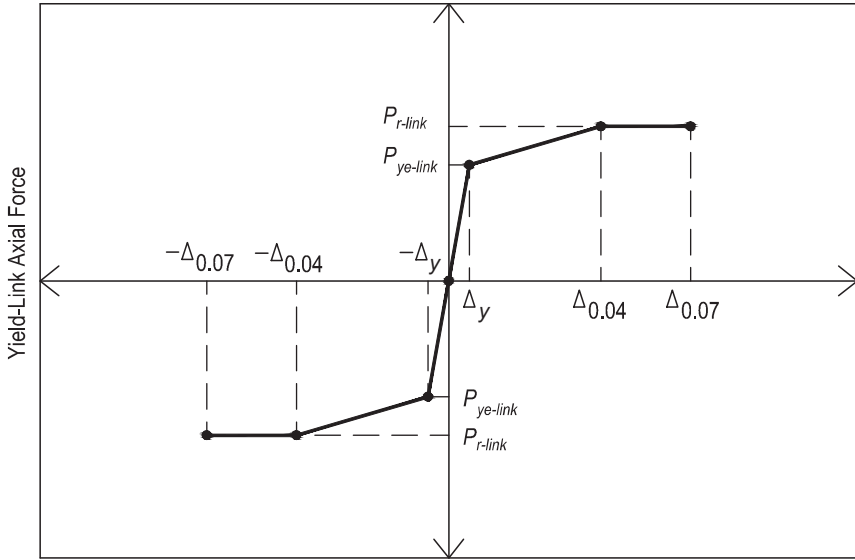
Adjust connection stiffness and/or number of connections as needed to comply.

Step 12. Determine the required shear strength, V_u , of the beam and beam web-to-column flange connection using

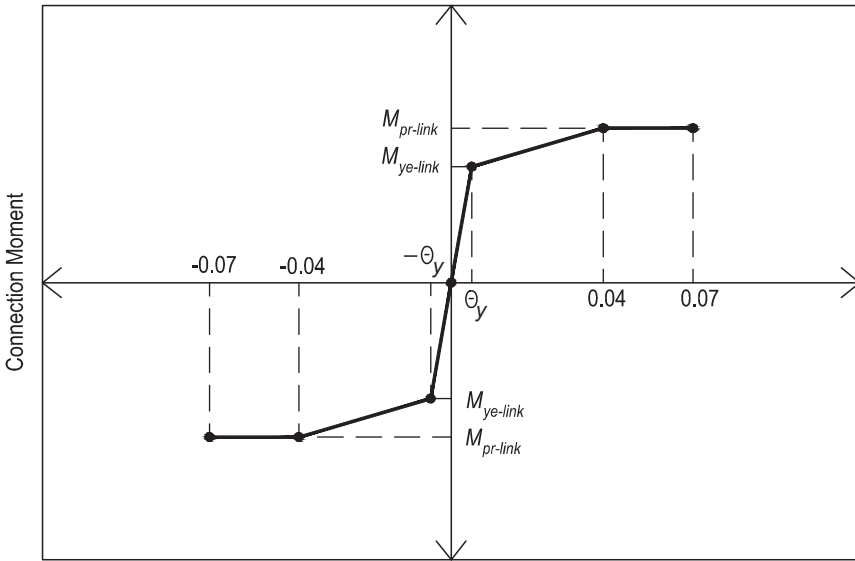
$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (12.9-34)$$

where

- L_h = horizontal distance between centerlines of the shear bolts in shear plate at each end of beam, in. (mm)
- $V_{gravity}$ = shear force in the beam, kips (N), resulting from $1.2D + f_1L + 0.15S$ (where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5). The shear force at the shear



(a) Yield-Link axial force vs. Yield-Link axial deformation



(b) Connection moment vs. rotation

Fig. 12.5. Simpson Strong-Tie moment connection modeling parameters.

plate connection shall be determined from a free-body diagram of the portion of the beam between the shear plate connections.

User Note: The load combination of $1.2D + f_1L + 0.15S$ is in conformance with ASCE/SEI 7-22. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.15 for S (snow) when the roof configuration is such that it does not shed snow off the structure.

Step 13. Verify the beam and column sizes selected in Step 1.

Step 13.1. Beams shall satisfy the AISC *Specification* considering the following:

- (a) Vertical load from all applicable load combinations.
- (b) Axial force due to seismic effects determined as the minimum of the maximum the system can deliver or as determined from the overstrength seismic load.
- (c) The application of M_{pr} at each end of the beam as required.

Step 13.2. Column strength shall satisfy the AISC *Specification* considering loads from all applicable load combinations in the applicable building code, where the seismic effects are determined from the minimum of either the maximum the system can deliver or the overstrength seismic loads. According to Section 12.3.2(7), if column bracing is only provided at the level of the top flange of the beam, in addition to the requirements of the AISC *Specification*, the maximum available flexural strength of the column outside the panel zone, $\phi_b M_n$, shall be taken as $\phi_b M_n \leq \phi_b F_y S_x$, where $\phi_b = 0.90$.

Step 14. Check the column-beam relationship limitations according to Section 12.4.

Step 15. Design the beam web-to-column flange connection for the following required strengths:

M_{u-sp} = moment in shear plate at column face, kip-in. (N-mm)
 $= V_u a$

P_{u-sp} = required axial strength of the connection, taken as the minimum of the following:

- (1) The maximum axial force the system can deliver.
- (2) The axial force calculated using the load combinations of the applicable building code, including the overstrength seismic load.

V_u = V_u from Step 12.

a = horizontal distance from centerline of the shear bolt holes in shear plate to face of the column, in. (mm). See Figure 12.3(c).

Step 15.1.

- (a) Calculate the maximum shear plate bolt shear, V_{u-bolt} , by sizing the shear plate central bolt to take the required axial and shear load from the beam.

$$V_{u-bolt} = \sqrt{\left(\frac{P_{u-sp}}{n_{bolt-sp-horz}}\right)^2 + \left(\frac{V_u}{n_{bolt-sp-vert}}\right)^2} \quad (12.9-35)$$

where

$n_{bolt-sp-horz}$ = total number of horizontal bolts resisting axial force in the shear plate in line with the central bolt

$n_{bolt-sp-vert}$ = total number of vertical bolts resisting shear force in the shear plate

- (b) Select a bolt diameter, d_{b-sp} , that satisfies the AISC *Specification*.

Step 15.2. Determine the shear-plate geometry required to accommodate a connection rotation of ± 0.07 rad.

$$L_{slot_horz} = d_{b-sp} + 1/8 \text{ in.} + 0.14s_{vert} \left(\frac{n_{bolt-sp-vert} - 1}{2}\right) \quad (12.9-36)$$

$$L_{slot_horz} = d_{b-sp} + 3 \text{ mm} + 0.14s_{vert} \left(\frac{n_{bolt-sp-vert} - 1}{2}\right) \quad (12.9-36M)$$

$$L_{slot_vert} = d_{b-sp} + 1/8 \text{ in.} + 0.14s_{horz}(n_{bolt-sp-horz} - 1) \quad (12.9-37)$$

$$L_{slot_vert} = d_{b-sp} + 3 \text{ mm} + 0.14s_{horz}(n_{bolt-sp-horz} - 1) \quad (12.9-37M)$$

where

d_{b-sp} = diameter of bolts in shear plate, in. (mm)

s_{horz} = horizontal bolt spacing, in. (mm)

s_{vert} = vertical bolt spacing, in. (mm)

Step 15.3. Check the shear plate for tension and shear yielding, tension and shear rupture, block shear, combined tension and flexural yielding at the column face, and bolt bearing, where deformation at the bolt hole is a design consideration, according to the AISC *Specification*.

Step 15.4. Size the weld at the shear plate-to-column flange or end-plate joint to develop the plate in shear, tension, and bending. For double fillet welds, the minimum leg size shall be $\geq t_p$.

Step 15.5. Check the beam web for tension and shear yielding, tension and shear rupture, block shear, and bolt bearing, where deformation at the bolt hole is a design consideration, according to the AISC *Specification*.

Step 15.6. Detail the beam flange and web cope such that the flange begins at a point aligned with the centerline of the shear-plate shear bolts. Check entering and tightening clearances as appropriate. See Figure 12.3(a).

User Note: Checking the beam web for flexure at the cope is not required because the flange copes do not extend beyond the centerline of the shear bolts in the beam shear-plate connection.

Step 16. Check the column panel-zone shear strength in accordance with the AISC *Specification*. The required shear strength shall be determined from the summation of the probable maximum axial strengths of the Yield-Link. Doubler plates shall be used as required.

Step 17. Check the column web for the concentrated force(s) of P_{r-link} according to the AISC *Specification*.

Step 18. Check the minimum column flange thickness for flexural yielding:

$$t_{cf_min} = \sqrt{\frac{1.11 M_{pr}}{\phi_d F_{yc} Y_c}} \quad (12.9-38)$$

where

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

Y_c = column flange yield line mechanism parameter from Table 6.5 or 6.3.

For connections away from column ends, Table 6.5 shall be used. For connections at column ends, Table 6.3 shall be used. An unstiffened column flange connection at the end of a column may be used where a rational analysis demonstrates that the unstiffened column flange design flexural strength, as controlled by flexural yielding of the column flange, meets or exceeds the connection moment demand, $M_{pr-link}$

Step 19. If a continuity plate or stiffener plate is required for any of the column limit states in Steps 17 and 18, the required strength, F_{su} , is

$$F_{su} = P_{r-link} - \text{minimum} (\phi R_n) \quad (12.9-39)$$

where

ϕR_n = design strengthss from Step 17, kips (N)

Step 19.1. Design the continuity plate or stiffener plate according to the AISC *Specification*.

Step 19.2. Design the stiffener-to-column web weld and the stiffener to-column flange weld according to the AISC *Specification*.

The continuity plate or stiffener shall conform to AISC *Specification* Section J10.8 and shall have a minimum thickness of $\frac{1}{4}$ in. (6 mm).

CHAPTER 13

DOUBLE-TEE MOMENT CONNECTIONS

13.1. GENERAL

Double-tee connections utilize T-stub components that are bolted to both the column flange and the beam flanges using high-strength bolts. Either four bolts or eight bolts attach the T-stub components to the column flanges. The top and bottom T-stubs shall be identical. T-stubs shall be cut from rolled sections. The beam web is connected to the column with a bolted single-plate shear connection. A detail for this connection is shown in Figure 13.1. Yielding and hinge formation are intended to occur in the beam near the ends of the stems of the T-stubs. Figures 13.2 through 13.6 provide details regarding the dimensioning notation used in this chapter.

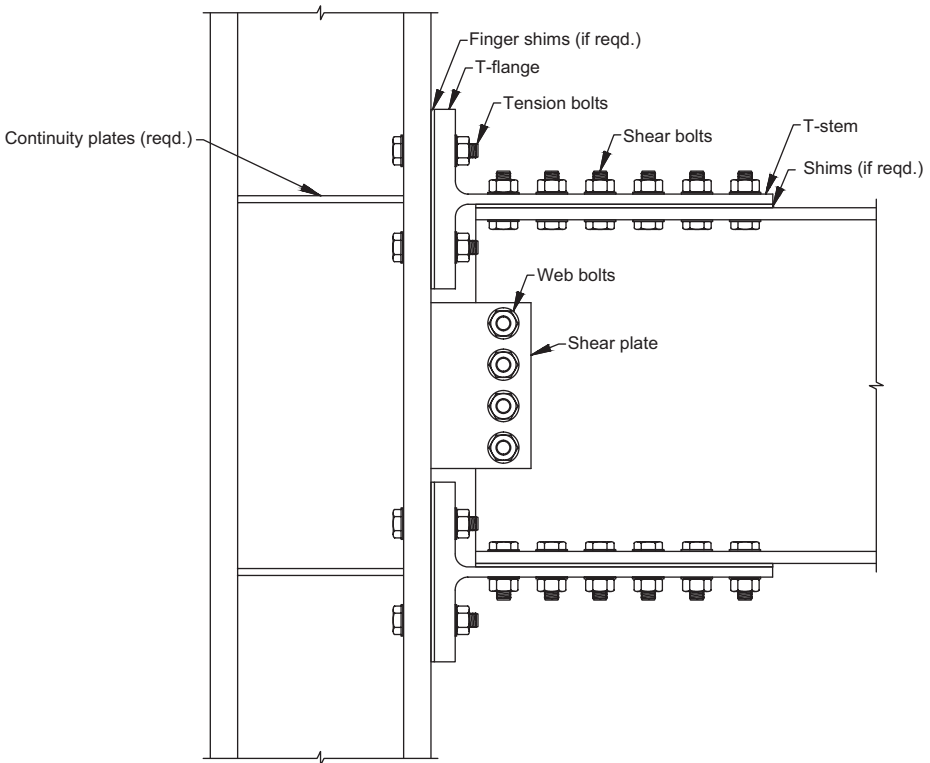


Fig. 13.1. Typical double-tee connection.

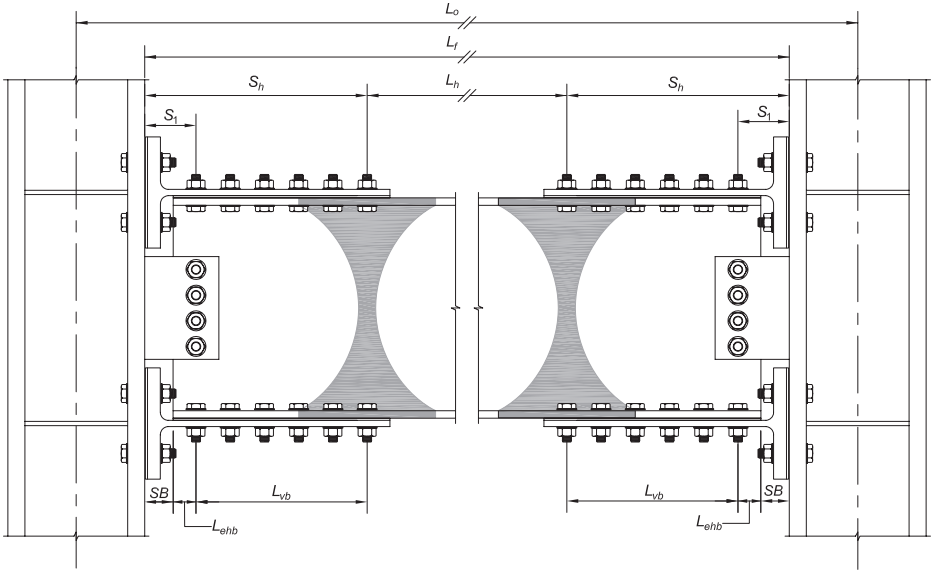


Fig. 13.2. Beam dimensions for double-tee connections. Shaded regions represent plastic hinges.

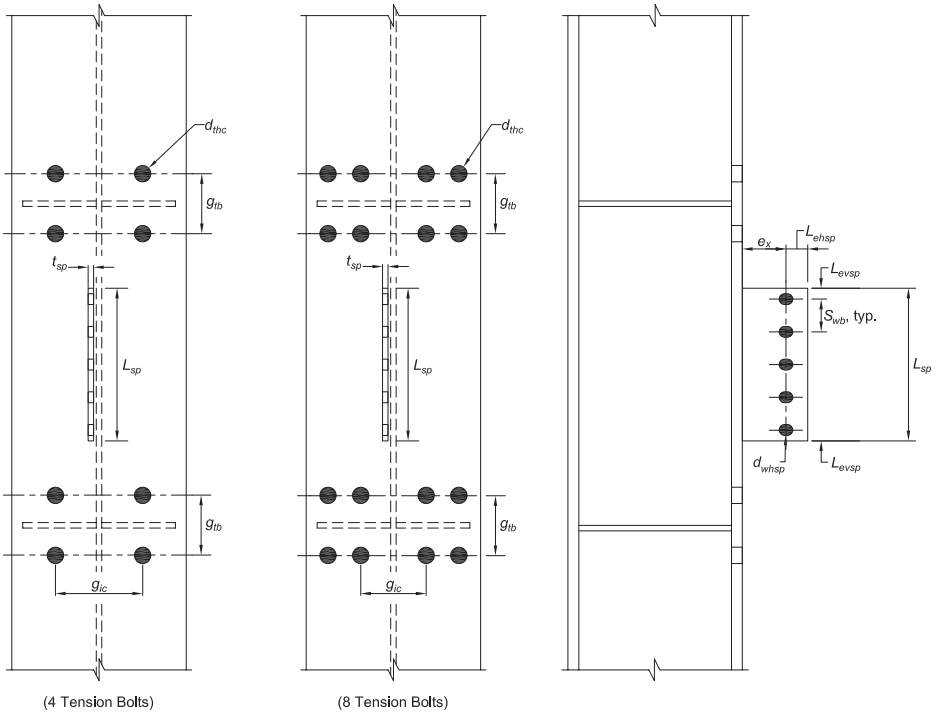


Fig. 13.3. Column and shear plate dimensions for double-tee connections.

13.2. SYSTEMS

Double-tee connections are prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limitation of these provisions.

Exception: Double-tee connections in SMF systems with concrete structural slabs are prequalified only if the concrete slab is isolated in accordance with Section 2.3.4.

User Note: Note that connections designed for use in SMF and IMF systems must be designed as fully restrained (FR) connections. It is possible to design double-tee connections that qualify as partially restrained (PR), even when they satisfy all of the strength requirements stipulated within this specification. As a result, care must be taken during design to ensure that the connections resulting from this chapter have not only adequate strength, but that they also have adequate stiffness.

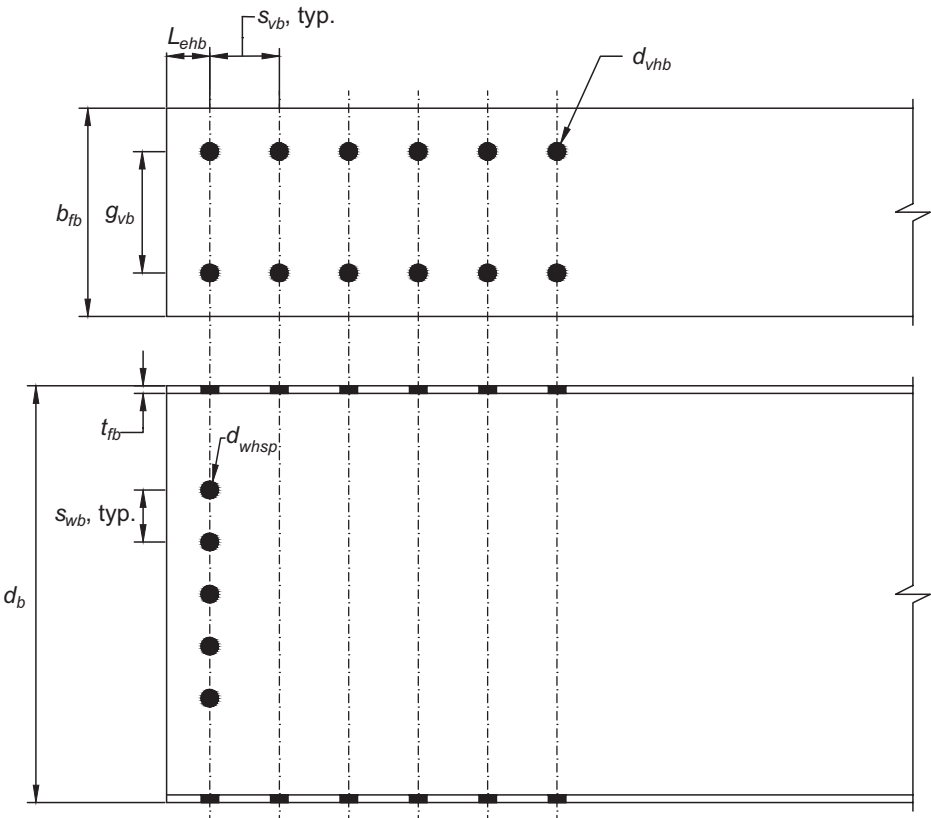


Fig. 13.4. Additional beam dimensions for double-tee connections.

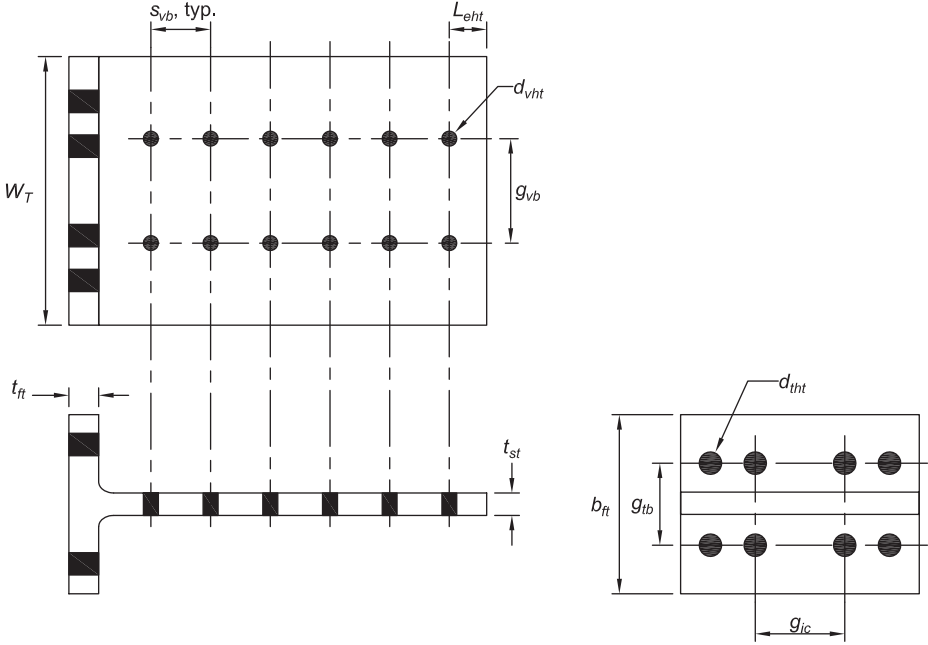


Fig. 13.5. T-stub dimensions for double-tee connections.

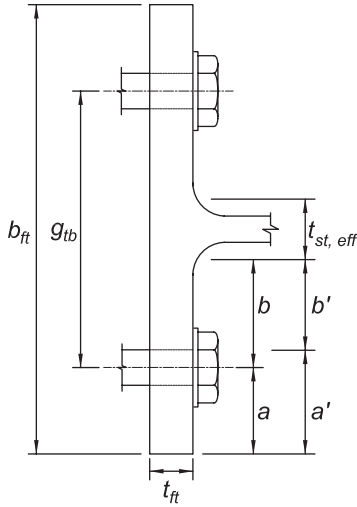


Fig. 13.6. T-stub flange dimensions for double-tee connections.

13.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or welded built-up I-shaped members conforming to the requirements in Section 2.3.
- (2) Beam depth, d_b , is limited to a maximum of W24 (W610) for rolled shapes. The depth of built-up members shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight is limited to a maximum of 55 lb/ft (82 kg/m).
- (4) Beam flange thickness is limited to a maximum of $\frac{5}{8}$ in. (16 mm).
- (5) The clear span-to-depth ratio of the beam shall be limited to 9 or greater for both SMF and IMF systems.
- (6) Width-to-thickness ratios for the flanges and web of the beam shall conform to the limits of the AISC *Seismic Provisions*.
- (7) Lateral bracing of beams shall be in conformance with the AISC *Seismic Provisions* for SMF or IMF systems, as applicable. To satisfy the requirements for lateral bracing at plastic hinges, lateral bracing shall be provided at a location on the beam that is between d_b and $1.5d_b$ beyond the bolt farthest from the face of the column. No attachment of lateral bracing shall be made to the beam within the protected zone.

Exception: For both SMF and IMF systems, where the beam supports a concrete structural slab that is connected between the protected zones with welded steel headed stud anchors, spaced a maximum of 12 in (300 mm) on center, supplemental lateral bracing at plastic hinges is not required.

- (8) The protected zone consists of the T-stubs and the portion of the beam between the face of the column and one beam depth, d , beyond the bolt farthest from the face of the column.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the rolled wide-flange shapes, built-up I-shapes, or flanged cruciform sections permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) Rolled shape column depth shall be limited to W36 (W920) maximum when a concrete structural slab is provided. In the absence of a concrete structural slab, rolled shape column depth is limited to W14 (W360) maximum. The depth of built-up I-shaped columns shall not exceed that for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes.

- (4) Width-to-thickness ratios for the flanges and web of the column shall conform to the applicable limits of the AISC *Seismic Provisions*.
- (5) Lateral bracing of columns shall conform to the applicable limits of the AISC *Seismic Provisions*.

13.4. CONNECTION DETAILING

1. T-Stub Material Specifications

T-stubs shall be cut from rolled sections and shall conform to either ASTM A992/A992M or ASTM A913/A913M Grade 50 (Grade 345).

2. Continuity Plates

Continuity plates shall be provided at the column with a thickness not less than the thickness of the beam flange. Continuity plates shall extend to the edge of the column flange, less $\frac{1}{4}$ in. (6 mm). The continuity plate welds shall be provided in accordance with the AISC *Seismic Provisions*.

3. Single-Plate Shear Connection Welds

The single-plate shear connection shall be welded to the column flange. The single-plate to column-flange connection shall consist of CJP groove welds, two-sided PJP groove welds, or two-sided fillet welds.

4. Bolts

Bolts shall satisfy the following requirements:

- (1) Bolts shall be arranged symmetrically about the axes of the members.
- (2) Shear bolts in the T-stem-to-beam-flange connection shall be limited to two bolts per row. Tension bolts in the T-flange-to-column-flange connection shall be arranged in two horizontal rows of either two or four bolts.
- (3) Types of holes:
 - (a) Standard holes shall be used in the beam flange and column flange.
 - (b) Standard or short-slotted holes (with slots aligned parallel to the axis of the beam) shall be used in either the beam web or the shear plate.
 - (c) Standard or oversized holes shall be used in the T-stem.
 - (d) Standard, oversized, or short-slotted holes (with slots aligned parallel to the axis of the column) shall be used in the T-flange.
- (4) Bolt holes in the T-stubs and beam flanges shall be drilled or sub-punched and reamed.
- (5) The ratio of tension-bolt gage to T-flange thickness, g_{tb}/t_{ft} , shall be no larger than 7.0.

- (6) All bolts shall be installed as pretensioned high-strength bolts.
- (7) Faying surfaces of the beam flange and T-stem shall satisfy the requirements for slip-critical connections in accordance with AISC *Specification* Section J3.9. Faying surfaces shall have a Class A slip coefficient or higher.

User Note: The use of oversized holes in the T-stem with pretensioned bolts that are not designed as slip critical is permitted, consistent with AISC *Seismic Provisions* Section D2.2.

5. T-Stub Shims

- (1) Steel shims with a maximum thickness of ¼ in. (6 mm) may be used between the stems of the tees and the flanges of the beam at either or both locations, subject to the limitations of the RCSC *Specification*.
- (2) The use of finger shims between the flanges of the tees and the flange of the column is permitted at either or both locations, subject to the limitations of the RCSC *Specification*.

13.5. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3, where $Z_e = Z_b =$ plastic section modulus of the beam.

Step 2. Check beam flange tensile rupture. To preclude a net section fracture of the beam flange, the net section of the beam section shall satisfy the following:

$$Z_{b,net}R_tF_{ub} \geq Z_bR_yF_{yb} \quad (13.5-1)$$

where

F_{ub} = specified minimum tensile strength of beam, ksi (MPa)

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

R_t = ratio of expected tensile strength to specified minimum tensile strength

R_y = ratio of expected yield stress to specified minimum yield stress

Z_b = plastic section modulus of beam, in.³ (mm³)

$Z_{b,net}$ = plastic section modulus of net section of the beam at location of the plastic hinge, in.³ (mm³)

User Note: $Z_{b,net}$ of the beam may be computed by accounting for only the holes in the tension flange or, more simply, $Z_{b,net}$ of the beam may be computed by accounting for the holes in both flanges. Note that if the former approach is employed, the plastic neutral axis will not be at the mid-depth of the beam, which complicates the calculations somewhat. If the latter approach is employed, the calculations are a bit simpler and the requirement of Equations 13.5-1 may be met with a maximum shear bolt-hole diameter that is determined by:

$$d_{hole} \leq \left[\frac{Z_b}{2t_f(d-t_f)} \right] \left(1 - \frac{R_y F_{yb}}{R_t F_{ub}} \right) - 1/16 \text{ in.} \quad (13.5-2)$$

$$d_{hole} \leq \left[\frac{Z_b}{2t_f(d-t_f)} \right] \left(1 - \frac{R_y F_{yb}}{R_t F_{ub}} \right) - 2 \text{ mm} \quad (13.5-2M)$$

where

d = depth of beam, in. (mm)

d_{hole} = nominal hole diameter for bolts between flange plate and beam flange as specified in AISC *Specification* Table J3.3 or Table J3.3M, in. (mm)

t_f = flange thickness of the beam, in. (mm)

After the maximum shear bolt hole diameter is determined, the maximum shear bolt diameter can be determined from AISC *Specification* Table J3.3 or Table J3.3M.

Step 3. Determine the design shear strength per shear bolt based on the limit states of shear fracture and material bearing as follows:

$$\phi r_{nv} = \min \begin{cases} \text{bolt shear} & \phi_n F_{nv} A_{vb} \\ \text{beam flange bearing} & \phi_d 2.4 d_{vb} t_{fb} F_{ub} \\ \text{T-stem bearing} & \phi_d 2.4 d_{vb} t_{st} F_{ut} \end{cases} \quad (13.5-3)$$

where

A_{vb} = gross area of a shear bolt measured through its shank, in.² (mm²)

F_{nv} = nominal shear stress of a bolt from the AISC *Specification*, ksi (MPa)

d_{vb} = diameter of the shear bolts between the T-stem and the beam flange, in. (mm)

r_{nv} = nominal shear strength of a shear bolt, kips/bolt (N/bolt)

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

t_{st} = stem thickness of the T-stub, in. (mm)

Step 4. Estimate the number of shear bolts, n_{vb} , required in each beam flange as follows:

$$n_{vb} \geq \frac{1.25 M_{pr}}{d_b \phi r_{nv}} \quad (13.5-4)$$

where n_{vb} is an even integer.

Step 5. Determine the distance of the plastic hinge from the face of the column, S_h , based on the estimated number of shear bolts, the horizontal end distance, and bolt spacing.

$$S_h = S_1 + L_{vb} \quad (13.5-5)$$

where

$$L_{vb} = S_{vb} \left(\frac{n_{vb}}{2} - 1 \right), \text{ in. (mm)} \quad (13.5-6)$$

S_1 = distance between the face of the column and the first row of shear bolts, in. (mm)

s_{vb} = spacing of the shear bolts, in. (mm)

Step 6. Compute the shear force at the plastic hinge, V_h , in accordance with Section 2.4.4.

Step 7. Compute the probable maximum moment at the face of the column, M_f , in accordance with Section 2.4.5.

Step 8. Compute the probable force in the T-stub, F_{pr} , due to M_f :

$$F_{pr} = \frac{M_f}{1.05d_b} \quad (13.5-7)$$

where $1.05d_b$ is used to estimate the sum of the depth of the beam and the thickness of the T-stem.

Step 9. Determine the size of the T-stem required. The stem thickness shall be determined based on the limit states of gross section yielding and net section fracture (checked in this step) and compression due to flexural buckling (checked in Step 16).

In sizing the T-stem, the Whitmore width, W_{Whit} , shall be taken as

$$W_{Whit} = 2L_{vb} \tan 30^\circ + g_{vb} \quad (13.5-8)$$

where

g_{vb} = gage of shear bolts in the T-stub, in. (mm)

The minimum stem thickness, t_{st} , based on yielding of the T-stem is

$$t_{st} \geq \frac{F_{pr}}{\min(W_T, W_{Whit}) \phi_d F_{yt}} \quad (13.5-9)$$

where

F_{yt} = specified minimum yield stress of the T-stub, ksi (MPa)

W_T = width of the T-stub measured parallel to the column flange width, in. (mm)

The minimum stem thickness, t_{st} , based on fracture of the T-stem is:

$$t_{st} \geq \frac{F_{pr}}{\phi_n F_{ut} \left[\min(W_T, W_{Whit}) - 2(d_{vht} + 1/16 \text{ in.}) \right]} \quad (13.5-10)$$

$$t_{st} \geq \frac{F_{pr}}{\phi_n F_{ut} \left[\min(W_T, W_{Whit}) - 2(d_{vht} + 2 \text{ mm}) \right]} \quad (13.5-10M)$$

where

- F_{ut} = specified minimum tensile stress of T-stub, ksi (MPa)
 d_{vht} = diameter of the holes in the T-stem for the shear bolts, in. (mm)

To ensure that compression buckling of the T-stem will not control, select the thickness of the T-stem such that

$$t_{st} \geq \frac{S_1 - t_{ft}}{9.60} \quad (13.5-11)$$

where

- S_1 = distance from the face of the column to the first row of shear bolts, in. (mm)
 t_{ft} = flange thickness of the T-stub, in. (mm)

Step 10. Determine the size of the bolts connecting the T-stub to the column flange. The minimum diameter of the tension bolts, d_{tb} , shall be determined as

$$d_{tb} \geq \sqrt{\frac{4F_{pr}}{n_{tb}\phi_n\pi F_{nt}}} \quad (13.5-12)$$

where

- F_{nt} = nominal tensile stress of bolt from the AISC *Specification*, ksi (MPa)
 n_{tb} = number of tension bolts connecting the T-flange to the column flange

Step 11. Determine the preliminary configuration of the T-flange. The flange width of the T-stub, b_{ft} , shall be computed as

$$b_{ft} \geq g_{tb} + 2a \quad (13.5-13)$$

where

- $a = 1.5d_{tb} \leq 1.25b$ (13.5-14)
 b = distance between effective T-stem and bolt line in the T-flange, in. (mm)
 g_{tb} = gage of the tension bolts in the T-stub, in. (mm)

User Note: The limit of $a \leq 1.25b$ in Equation 13.5-14 is a computation limit only and is not a physical limitation on the dimension a . The dimension b may be estimated as $0.40g_{tb}$ for preliminary estimation of prying forces.

The design strength of a single tension bolt, ϕr_{nt} , shall be computed as

$$\phi r_{nt} = \phi_n A_{tb} F_{nt} \quad (13.5-15)$$

where

- A_{tb} = gross area of a tension bolt measured through its shank, in.² (mm²)

The required T-flange strength, T_{req} , in units of kips per tension bolt (N per tension bolt) shall be computed as

$$T_{req} = F_{pr}/n_{tb} \quad (13.5-16)$$

The minimum flange thickness, t_{fi} , based on a mixed-mode failure, which will typically govern, shall be computed as:

$$t_{fi} \geq 2 \sqrt{\frac{T_{req}(a' + b') - \phi r_{nt} a'}{\phi_d F_{yt} p}} \quad (13.5-17)$$

where

$$a' = a + \frac{1}{2} d_{tb} \quad (13.5-18)$$

$$b' = b + \frac{1}{2} d_{tb} \quad (13.5-19)$$

$$p = \frac{2W_T}{n_{tb}} \quad (13.5-20)$$

In certain situations, the term under the radical in Equation 13.5-17 can be negative, resulting in an erroneous required flange thickness. An alternate formulation of Equation 13.5-17 that can be applied in these cases is

$$t_{st} \geq 2 \sqrt{\frac{\phi r_{nt} a' b'}{\phi_d F_{yt} p [a' + \delta(a' + b')]}} \quad (13.5-21)$$

where

$$\delta = 1 - \frac{d_{tht}}{p} \quad (13.5-22)$$

User Note: In all cases, the flange thickness required to eliminate prying action can be computed as:

$$t_{fi,crit} = \sqrt{\frac{4\phi r_{nt} b'}{\phi_d F_{yt} p}} \quad (13.5-23)$$

Step 12. Select a W-shape from which the T-stubs will be cut based on the following:

1. The minimum depth required to accommodate the setback and horizontal end distance of the beam, S_1 , and the length of the shear-bolt group, L_{vb} , found in Step 5.
2. The minimum web thickness, t_{st} , found in Step 9.
3. The minimum flange width, b_{fi} , and flange thickness, t_{fi} , found in Step 11.

Step 13. Check the connection rotational stiffness to ensure that the connection is classified as fully restrained. The following shall be satisfied:

$$K_i \geq \frac{18EI_{beam}}{L_o} \quad (13.5-24)$$

where

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

I_{beam} = strong axis moment of inertia of the beam, in.⁴ (mm⁴)

L_o = theoretical length of the connected beam measured between the working points of the adjacent columns, in. (mm)

$$K_i = \frac{d_b^2 K_{ten} K_{comp}}{K_{ten} + K_{comp}} \quad (13.5-25)$$

$$K_{ten} = \left(\frac{1}{K_{flange}} + \frac{1}{K_{stem}} + \frac{1}{K_{slip}} \right)^{-1} \quad (13.5-26)$$

$$K_{comp} = \left(\frac{1}{K_{stem}} + \frac{1}{K_{slip}} \right)^{-1} \quad (13.5-27)$$

$$K_{flange} = \frac{12n_{tb}EI_{ft}(a'\beta_a + 3b'\beta_b)}{b'^3\beta_b(4a'\beta_a + 3b'\beta_b)} \quad (13.5-28)$$

$$K_{stem} = \frac{t_{st}E(W_T - b_{fb})^2}{L_{stem} \left[(W_T - b_{fb}) + b_{fb} \ln \left(\frac{b_{fb}}{W_T} \right) \right]} \quad (13.5-29)$$

and

L_{stem} = length of stem, in. (mm)

$$K_{slip} = \frac{P_{slip}}{\Delta_{slip}} \quad (13.5-30)$$

$$P_{slip} = n_{vb}\alpha(0.70F_{nt}A_{vb})\mu \quad (13.5-31)$$

where

α = 1.00 for ASTM F3125/F3125M Grades A325, A325M, and F1852 bolts

= 0.88 for ASTM F3125/F3125M Grades A490, A490M, and F2280 bolts

$$I_{ft} = \frac{pt_{ft}^3}{12} \quad (13.5-32)$$

$$\beta_a = 1 + \frac{12EI_{ft}}{Gpt_{ft}a'^2} \quad (13.5-33)$$

$$\beta_b = 1 + \frac{12EI_{ft}}{Gpt_{ft}b'^2} \quad (13.5-34)$$

$$\Delta_{slip} = 0.0076 \text{ in. (0.19 mm)} \quad (13.5-35)$$

Step 14. Compute the maximum force in the T-stub due to M_f . Using the actual T-stem thickness, the actual flange force that is to be carried by the T-stubs, F_f , shall be computed as follows:

$$F_f = \frac{M_f}{d_b + t_{st}} \quad (13.5-36)$$

Step 15. Back-check the strength of the shear bolts with the actual flange force. Use ϕr_{nv} from Step 3 to confirm that the number of shear bolts, n_{vb} , estimated in Step 4 is adequate to resist the actual flange force, F_f .

$$\phi R_n \geq F_f \quad (13.5-37)$$

Step 16. Back-check the strength of the T-stem using the maximum beam flange force. Back-check that the gross section yielding, net section fracture, and flexural buckling strengths of the T-stem are adequate to resist the maximum flange force, F_f .

$$\phi R_n \geq F_f$$

For stem gross section yielding

$$\phi R_n = \phi_d F_{yt} \min(W_T, W_{Whit}) t_{st} \quad (13.5-38)$$

For stem net section fracture

$$\phi R_n = \phi_n F_{ut} \left[\min(W_T, W_{Whit}) - 2(d_{vht} + 1/16 \text{ in.}) \right] t_{st} \quad (13.5-39)$$

$$\phi R_n = \phi_n F_{ut} \left[\min(W_T, W_{Whit}) - 2(d_{vht} + 2 \text{ mm}) \right] t_{st} \quad (13.5-39M)$$

For stem flexural buckling

$$\frac{KL}{r} = \frac{(0.75)(S_1 - t_{ft})}{\sqrt{\frac{W_T t_{st}^3}{12 W_T t_{st}}}} = 2.60 \left(\frac{S_1 - t_{ft}}{t_{st}} \right) \quad (13.5-40)$$

If $KL/r \leq 25$, then

$$\phi R_n = \phi_d F_{yt} \min(W_T, W_{Whit}) t_{st} \quad (13.5-41)$$

If $KL/r > 25$, then ϕR_n is determined using the provisions in AISC *Specification* Section E3 using KL/r as determined previously and taking ϕ equal to ϕ_n .

Step 17. Back-check the flange strength of the T-stub. The flange strength of the T-stub shall be computed as follows:

$$\phi R_n = n_{tb} \phi T \quad (13.5-42)$$

where ϕT is the minimum of ϕT_1 (plastic flange mechanism), ϕT_2 (mixed-mode failure), and ϕT_3 (tension bolt fracture with no prying), as computed in the following.

For the limit state of a plastic mechanism in the tension flange, the design strength per tension bolt shall be calculated as follows:

$$\phi T_1 = \frac{(1 + \delta)}{4b'} p \phi_d F_{yt} t_{ft}^2 \quad (13.5-43)$$

For the limit state of tension flange yielding followed by fracture of the bolts (a mixed-mode failure), the design strength per tension bolt shall be calculated as follows:

$$\phi T_2 = \frac{\phi r_{nt} a'}{a' + b'} + \frac{p \phi_d F_{yt} t_{ft}^2}{4(a' + b')} \quad (13.5-44)$$

For the limit state of bolt fracture without yielding of the tension flange, the design strength per tension bolt shall be calculated as follows:

$$\phi T_3 = \phi r_{nt} \quad (13.5-45)$$

where

$$a' = a + \frac{1}{2} d_{tb} \quad (13.5-46)$$

$$a = \frac{1}{2}(b_{ft} - g_{tb}) \leq 1.25b \quad (13.5-47)$$

$$b' = b - \frac{1}{2} d_{tb} \quad (13.5-48)$$

$$b = \frac{1}{2}(g_{tb} - t_{st,eff}) \quad (13.5-49)$$

$$p = \frac{2W_T}{n_{tb}} \quad (13.5-50)$$

$$t_{st,eff} = k_1 + \frac{t_{st}}{2} \quad (13.5-51)$$

$$\delta = 1 - \frac{d_{ht}}{p} \quad (13.5-52)$$

d_{ht} = diameter or width of holes in T-flange for the tension bolts, in. (mm)

Step 18. Check the bearing and tear-out strength of the beam flange and T-stem. Bearing and tearout of the shear bolts shall be checked in a manner consistent with AISC *Specification* Chapter J. For these calculations, bearing and tearout are considered to be ductile failure modes.

Step 19. Check block shear of the beam flange and the T-stem. Block shear of the T-stem and beam flange shall be checked in a manner consistent with AISC *Specification* Chapter J. For the purpose of this design, the block shear failure shall be considered a ductile failure mode and ϕ_d shall be used.

The alternate block shear mechanism illustrated in Figure 13.7 need not be checked.

Step 20. Check design shear strength of beam in accordance with the AISC *Specification*.

The required shear strength of the beam shall be determined in accordance with Section 2.5.

Step 21. Design the beam web-to-column connection.

The beam web-to-column connection shall be a single-plate shear connection meeting the requirements of the AISC *Specification*.

The required shear strength of the single-plate connection shall be taken equal to the required shear strength of the beam.

User Note: Because of the large setback required, the shear connection may need to be designed as an extended single-plate shear connection. Most importantly, the length of the shear connection, L_{sc} , should be determined so as to fit between the flanges of the T-stubs allowing ample clearance.

Step 22. Check the column flange for flexural yielding (see Figure 13.8):

The column flange flexural design strength is

$$\phi R_n = \phi_d F_{yc} Y_C t_{fc} \quad (13.5-53)$$

where

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

$$Y_C = \left(\frac{2}{b} \right) \left(s + p_s + \frac{a_c b_c + b_c^2}{s} + \frac{a_c b_c + b_c^2}{p_s} \right) \quad (13.5-54)$$

$$a_c = \frac{b_{fc} - g_{ic}}{2} \quad (13.5-55)$$

$$b_c = \frac{g_{ic}}{2} \quad (13.5-56)$$

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

g_{ic} = gage of interior tension bolts in the column flange, in. (mm)

g_{tb} = gage of tension bolts in T-stub, in. (mm)

$$p_s = \frac{g_{tb} - t_{cp}}{2} \leq s \quad (13.5-57)$$

$$s = \frac{\sqrt{b_{fc} g_{ic}}}{2} \quad (13.5-58)$$

t_{cp} = thickness of continuity plates, in. (mm)

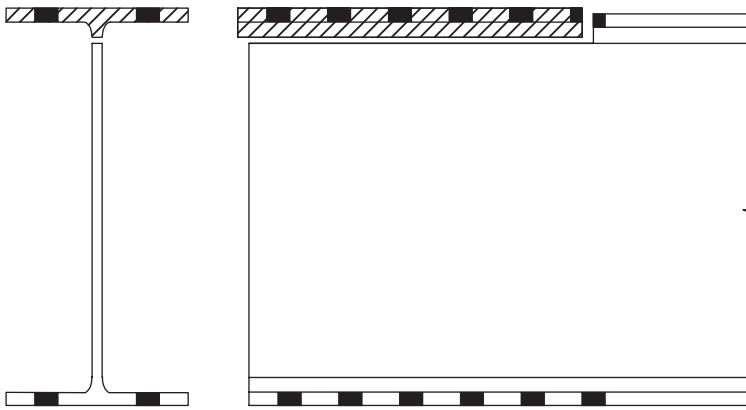


Fig. 13.7. Alternate block shear mechanism.

CHAPTER 14

SLOTTED WEB (SW) MOMENT CONNECTION

14.1. GENERAL

The slotted web moment connection features slots in the web of the beam that are parallel and adjacent to each flange, as shown in Figure 14.1. Inelastic behavior is expected to occur through yielding and buckling of the beam flanges in the region of the slot accompanied by yielding of the web in the region near the end of the shear plate.

14.2. SYSTEMS

The slotted web (SW) connection is prequalified for use in special moment frames (SMF) within the limits of these provisions. The SW connection is exempt from the requirements of Section 2.3.4.

14.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I shaped members conforming to the requirements of Section 2.3.

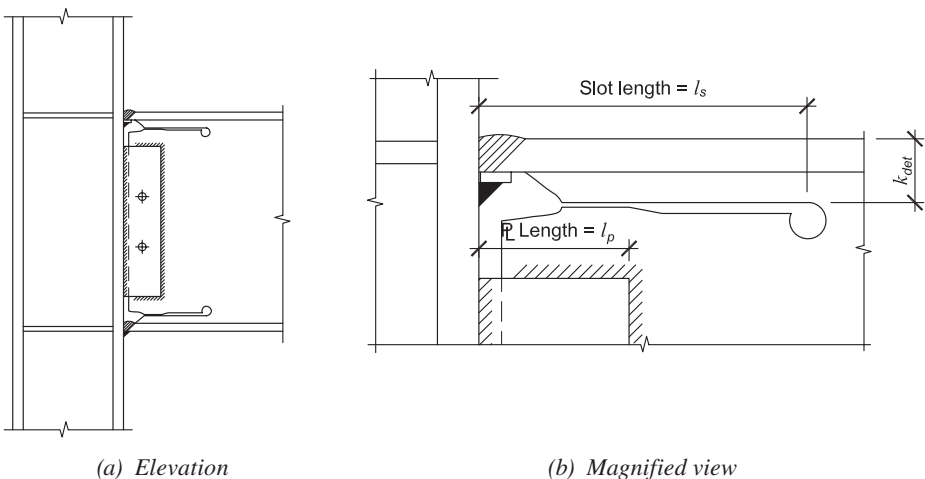


Fig. 14.1. SW beam-to-column moment connection.

- (2) Beam depth shall be limited to a maximum of W36 (W920) for rolled shapes. The depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight shall be limited to a maximum 400 lb/ft (600 kg/m).
- (4) Beam flange thickness shall be limited to a maximum of 2¼ in. (56 mm).
- (5) The clear span-to-depth ratio of the beam shall be limited to 6.4 or greater.
- (6) Width-to-thickness ratios for the flanges and webs of the beam shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of the beams shall be provided in conformance with the AISC *Seismic Provisions*. No supplemental lateral bracing is required at the plastic hinges.
- (8) The protected zones as shown in Figure 14.2 consist of the following:
 - (a) The portion of the beam web between the face of the column to the end of the slots plus one-half the depth of the beam, d , beyond the slot end, and

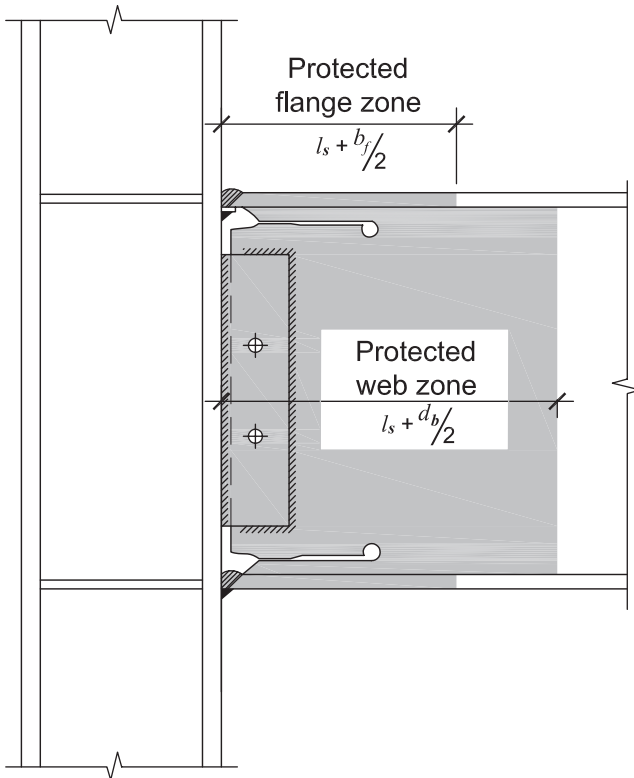


Fig. 14.2. Protected zones.

- (b) The beam flange from the face of the column to the end of the slot plus one-half the beam flange width, b_f .

(2) Column Limitations

- (1) Columns shall be of any of the rolled wide-flange shapes or built-up shapes permitted in Section 2.3.
- (2) The beam shall be connected to the flange of the column.
- (3) Rolled-shape column depths shall be limited to W36 (W920). The depth of built-up wide-flange columns shall not exceed that allowed for rolled shapes. Flanged cruciform columns shall not have a width or depth greater than the depth allowed for rolled shapes. Built-up box columns shall not have a width or depth exceeding 24 in. (600 mm). Boxed wide-flange columns shall not have a width or depth exceeding 24 in. (600 mm) if participating in orthogonal moment frames.
- (4) There is no limit on the weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Width-thickness ratios for the flanges and web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

14.4. BEAM FLANGE-TO-COLUMN FLANGE WELD LIMITATIONS

Beam flange-to-column flange connections shall satisfy the following limitations:

- (1) Beam flanges shall be connected to the column flanges using CJP groove welds. Beam flange welds shall conform to the requirements of demand critical welds in the AISC *Seismic Provisions*.
- (2) Weld access hole geometry shall conform to the requirements of the AISC *Specification*.

14.5. BEAM WEB AND SHEAR PLATE CONNECTION LIMITATIONS

Beam web and shear plate connections shall satisfy the following limitations.

The shear plate shall be welded to the column flange using a CJP groove weld, a PJP groove weld, or a combination of PJP and fillet welds. The shear plate shall be bolted to the beam web and fillet welded to the beam web. The horizontal fillet welds at the top and bottom of the shear plate shall be terminated at a distance not less than one fillet weld size from the end of the beam. The beam web shall be connected to the column flange using a CJP groove weld extending the full height of the shear plate. The shear plate connection shall be permitted to be used as backing for the CJP groove weld. The beam web-to-column flange CJP groove weld shall conform to the requirements for demand critical welds in the AISC *Seismic Provisions*.

- (a) If weld tabs are used, they need not be removed.

- (b) If weld tabs are not used, the CJP groove weld shall be terminated in a manner that minimizes notches and stress concentrations, such as with the use of cascaded welds. Cascaded welds shall be performed at a maximum angle of 45° relative to the axis of the weld. Nondestructive testing (NDT) of the cascaded weld ends need not be performed.

14.6. FABRICATION OF BEAM WEB SLOTS

The beam web slots shall be made using thermal cutting or milling of the slots and holes or by drilling the holes to produce surface roughness in the slots or holes not exceeding $1,000 \mu\text{in.}$ ($25 \mu\text{m}$). Gouges and notches that may occur in the cut slots shall be repaired by grinding. The beam web slots shall terminate at thermally cut or drilled $1\frac{1}{16}\text{-in.}$ (27-mm -) diameter holes for beams nominal 24 in. (600 mm) deep or greater or $\frac{13}{16}\text{-in.}$ (21-mm -) diameter holes for beams less than nominal 24 in. (600 mm) deep. The slot widths and tolerances are shown in Figure 14.3. The length of the $\frac{1}{8} \text{ in.}$ (3 mm) slot shall be at least equal to the width of the shear plate but need not exceed half the slot length, l_s . The transition from the $\frac{1}{8} \text{ in.}$ (3 mm) slot to the $\frac{1}{4} \text{ in.}$ (6 mm) slot shall not have a slope greater than 1 vertical to 3 horizontal.

14.7. DESIGN PROCEDURE

Step 1. Compute the probable maximum moment at the plastic hinge, M_{pr} , in accordance with Section 2.4.3, where $Z_e = Z_b =$ plastic section modulus of the beam, in.^3 (mm^3).

Step 2. Determine the distance of the plastic hinge from the face of the column, S_h , where $S_h = l_p =$ width of the shear plate, in. (mm).

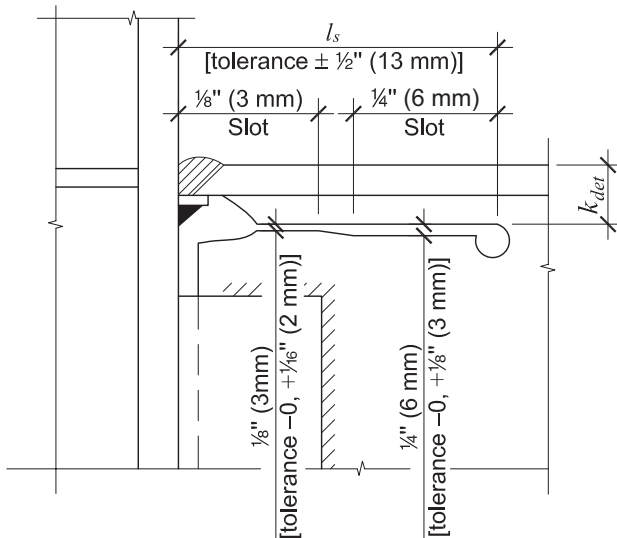


Fig. 14.3. Slot widths and tolerances.

Step 3. Compute the shear force at the plastic hinge, V_h , at each end of the beam in accordance with Section 2.4.4.

Step 4. Compute the probable maximum moment at the face of the column, M_f , in accordance with Section 2.4.5.

User Note: M_f is needed to check continuity plate requirements (Step 11) and panel-zone requirements (Step 12).

Step 5. Design the beam web slots. The beam slot length, l_s , shall be the least of the following within $\pm 10\%$:

$$l_s = 1.5b_f \quad (14.7-1)$$

$$l_s = 0.60t_{bf} \sqrt{\frac{E}{R_y F_{yb}}} \quad (14.7-2)$$

$$l_s = \frac{d}{2} \quad (14.7-3)$$

$$l_s = l_p + \frac{(l_b - l_p)}{10} \quad (14.7-4)$$

where

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

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

R_y = ratio of expected yield stress to specified minimum yield stress

b_f = width of beam flange, in. (mm)

d = depth of beam, in. (mm)

l_b = half the clear span length of beam, in. (mm)

l_p = width of the shear plate, in. (mm)

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

Step 6. Design the shear plate. Steel with a specified minimum yield stress of 50 ksi (345 MPa) shall be used. The shear plate width shall not be greater than half the length of the beam web slot or 6 in. (150 mm) but not less than one-third the beam slot length. The height, h , of the shear plate is determined as

$$h = T - 2 \text{ in.} \pm 1 \text{ in.} \quad (14.7-5)$$

$$h = T - 50 \text{ mm} \pm 25 \text{ mm} \quad (14.7-5M)$$

where T is defined in the AISC *Manual* for wide-flange shapes. The minimum shear plate thickness shall be equal to at least two-thirds of the beam web thickness but not less than $\frac{3}{8}$ in. (10 mm).

The minimum required shear plate thickness, t_p , is based upon the additional moment due to shear amplification from the end of the shear plate to the face of the column.

Use the plate elastic section modulus to conservatively compute the shear plate minimum thickness:

$$t_p = C_{pr} \left(\frac{6}{h^2} \right) \left(\frac{Z_b l_p}{l_b - l_p} \right) \quad (14.7-6)$$

where

$$Z_b = \text{plastic modulus of the beam, in.}^3 \text{ (mm}^3\text{)}$$

Step 7. Design the shear plate-to-beam web weld. The shear plate shall be welded to the beam web with an eccentrically loaded fillet weld group. The weld shall be designed to resist M_{weld} and V_{weld} to account for the resulting eccentricity, e_x . These values are determined as follows:

$$M_{weld} = C_{pr} \left(\frac{t_p}{t_p + t_{bw}} \right) \left(\frac{h}{T} \right)^2 Z_{web} R_y F_y \quad (14.7-7)$$

$$V_{weld} = V_h \left(\frac{t_p}{t_p + t_w} \right) \quad (14.7-8)$$

$$e_x = \frac{M_{weld}}{V_{weld}} \quad (14.7-9)$$

where

M_{weld} = moment resisted by the shear plate, kip-in. (N-mm)

V_h = the larger of the two values of shear at the plastic hinge computed in accordance with Section 2.4.4.

V_{weld} = shear resisted by the shear plate, kips (N)

Z_{web} = plastic section modulus of the beam web, in.³ (mm³)

$$= \frac{t_w T^2}{4} \quad (14.7-10)$$

e_x = eccentricity of the shear plate weld, in. (mm)

t_w = thickness of the beam web, in. (mm)

User Note: The AISC *Manual* design tables for “Eccentrically Loaded Weld Groups” may be used to design the shear plate-to-beam web fillet weld. Use the height and width of the shear plate and the shear eccentricity, e_x , as shown in Figure 14.4, to determine the weld design table coefficients.

Step 8. Design the shear plate-to-column flange weld.

The required strength of the weld connecting the shear plate to the column flange shall be equal to the nominal strength of the eccentrically loaded weld group as calculated according to Step 7.

Step 9. Select the high strength pretensioned bolts in standard holes for the shear plate-to-beam web connection to serve as both erection bolts and to stabilize the beam web from lateral buckling at the column flange. These bolts shall have a maximum bolt spacing of 6 in. (150 mm) on center over the full height of the plate. The diameter of the bolts shall be equal to or greater than the thickness of the beam web.

Step 10. Check the design shear strength of the beam in accordance with the AISC *Specification*.

The required shear strength of the beam shall be determined in accordance with Section 2.5.

Step 11. Check continuity plate requirements in accordance with Section 2.6.

Step 12. Check the panel zone in accordance with Section 2.7.

Step 13. Check the column-beam moment ratio in accordance with Section 2.8.

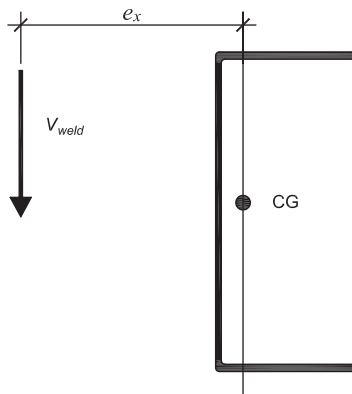


Fig. 14.4. Eccentrically loaded weld group.

CHAPTER 15

DURAFUSE FRAMES MOMENT CONNECTION

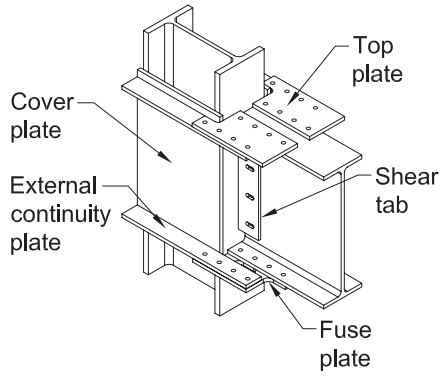
The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by multiple U.S. and foreign patents. By publication of this standard, no position is taken with respect to the validity of any claim(s) or of any patent rights in connection therewith. The patent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license, and the statement may be obtained from the standard's developer.*

15.1 GENERAL

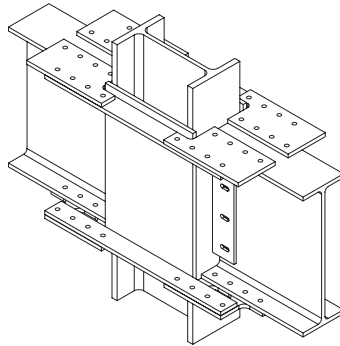
The DuraFuse Frames connection utilizes plates to connect beams to columns. For I-shaped columns, the column has cover plates on each side that are fillet welded to the column flanges [Figure 15.1(a) and (b)]. For box or HSS columns, the sides of the column may function as the cover plates [Figure 15.1(c)]. Four external continuity plates that extend past the face of the column are fillet welded to the column cover plates. The column has a single-plate shear connection with horizontal slotted holes that is fillet welded to the column face. The beam web, with standard holes, is attached to the single-plate shear connection with pretensioned bolts. The beam flanges are attached to the external continuity plates via top plates and a fuse plate (Figure 15.1). The beam flanges and external continuity plates have standard holes, while the top plates and fuse plate have oversized holes. The fuse plate is proportioned such that certain regions of the plate experience shear yielding when the connection is subjected to severe earthquake loading. The fuse plate is bolted in place so that it could be removed and replaced following a severe earthquake. The top plates are intended to experience minimal yielding such that they would not require repair following a severe earthquake. The various plates in the connection are proportioned such that the beam remains essentially elastic.

The inelastic mechanism is the same for DuraFuse Frames biaxial configurations (Figure 15.2). Fuse plates are proportioned to be the yielding element. The other plates and welds in the connections are capacity-based design elements, sized in accordance with the AISC *Specification* and the AISC *Seismic Provisions* using the maximum forces that can be delivered by the fuse plates.

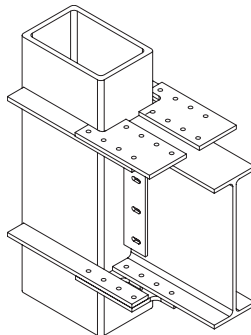
*The DuraFuse Frames connection configurations illustrated herein are protected by one or more of the following U.S. and foreign patents: U.S. Pat. Nos. 10,584,477; 10,689,876; 10,760,261; Canadian Pat. No. 3,007,316; Chilean Pat. No. 62.448. Other U.S. and foreign patent protection are pending.



(a) One-sided configuration with wide-flange beam and column



(b) Two-sided configuration with wide-flange beams and column



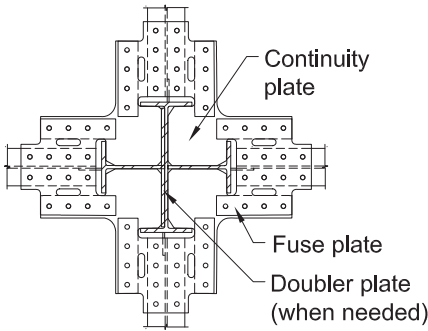
(c) Wide-flange beam with HSS column

Fig. 15.1. DuraFuse Frames connection configurations.
(Two-sided HSS column connection permitted but not shown.)

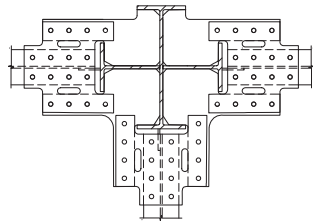
15.2 SYSTEMS

The DuraFuse Frames connection is prequalified for use in special moment frame (SMF) and intermediate moment frame (IMF) systems within the limits of these provisions.

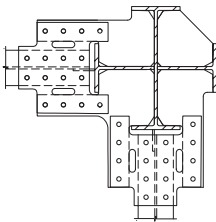
Exception: DuraFuse Frames connections in SMF systems with concrete structural slabs are only prequalified if the concrete structural slab is isolated in accordance with Section 2.3.4.



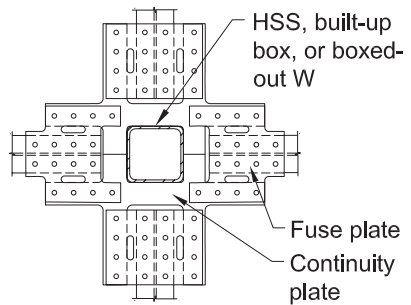
(a) Four-sided flanged cruciform



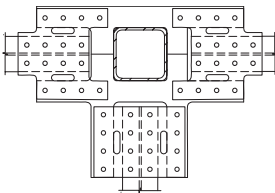
(b) Three-sided flanged cruciform



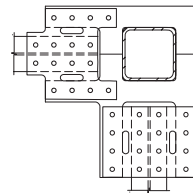
(c) Two-sided flanged cruciform



(d) Four-sided HSS or box



(e) Three-sided HSS or box



(f) Two-sided HSS or box

Fig. 15.2. Bottom views of DuraFuse Frames biaxial configurations.

15.3 PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

- (1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.
- (2) Beam depth shall be limited to a maximum of W40 (W1020) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.
- (3) Beam weight shall be limited to a maximum of 309 lb/ft (460 kg/m).
- (4) There are no limits on the beam web width-to-thickness ratio beyond those listed in the AISC *Specification*. The beam flange width-to-thickness ratio shall not exceed λ_p per AISC *Specification* Table B4.1b.
- (5) Lateral bracing of beams and joints—there are no requirements for stability bracing of beams or joints beyond those in the AISC *Specification*.

2. Column Limitations

Columns shall satisfy the following limitations:

- (1) Columns shall be any of the shapes permitted in Section 2.3.
- (2) The beam shall be connected to the column with connection plates.
- (3) Rolled shape column depth shall be limited to W36 (W920) maximum. Column depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes. Built-up box-columns shall not have a width exceeding 17 in. (430 mm).
- (4) There is no limit on weight per foot of columns.
- (5) There are no additional requirements for flange thickness.
- (6) Width-to-thickness ratios for the flanges and the web of columns shall conform to the requirements of the AISC *Seismic Provisions*.
- (7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

3. Plate Limitations

- (1) All connection plates shall be fabricated from structural steel that complies with ASTM A572/A572M Grade 50 (Grade 345). Fuse plates shall be fabricated from plates that have mill certified tensile strengths less than or equal to 85 ksi (590 MPa), unless independent material testing determines that the tensile strength is less than or equal to 85 ksi (590 MPa).
- (2) The thickness of the external continuity plates, top plates, and fuse plates shall not exceed 2 in. (50 mm) and shall not be less than 1/2 in. (13 mm).

- (3) The width of the yielding regions in the fuse plates shall not exceed 4 in. (100 mm) and shall not be less than 1½ in. (38 mm).
- (4) The width-to-thickness ratio of the yielding regions in the fuse plates shall not exceed 4.25 and shall not be less than 1.5.
- (5) The width-to-depth ratio of the yielding regions in the fuse plates shall not exceed 1.25 and shall not be less than 0.5.
- (6) The protected zone of the connection consists of the fuse plate.

15.4 COLUMN-BEAM RELATIONSHIP LIMITATIONS

Beam-to-column connections shall satisfy the following limitations.

Column-beam moment ratios shall be limited as follows.

- (a) For SMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*. The value of $\sum M_{pb}^*$ shall be taken equal to:

$$\sum (M_{pr} + M_{uw})$$

where

M_{pr} = probable maximum moment at the fuse determined in accordance with Section 2.4.3, kip-in. (N-mm)

$M_{uw} = V_b(d_c/2)$, kip-in. (N-mm)

$V_b = 2M_{pr}/L_h + V_g$, kips (N)

L_h = clear distance between column faces, in. (mm)

V_g = beam shear caused by gravity loads based on the load combination $1.2D + f_1L + 0.15S$, where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5, kips (N)

d_c = depth of column, in. (mm)

The value of $\sum M_{pc}^*$ shall be the sum of the projections of the nominal flexural strengths, M_{pc} , of the column above and below the joint, at the potential hinge location located one-quarter of the column depth above and below the cover plates edges.

- (b) For IMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*.

15.5 CONNECTION DETAILING

Figures 15.3 through 15.7 define the symbols for various plate thicknesses, dimensions, weld sizes, and bolt quantities that are pertinent to the connection and discussed in the design steps.

1. Welds

The welds designated [W1] and [W2] in Figure 15.3 shall be held back one weld size. When [W2] is made to an HSS column, the weld holdback shall be measured from

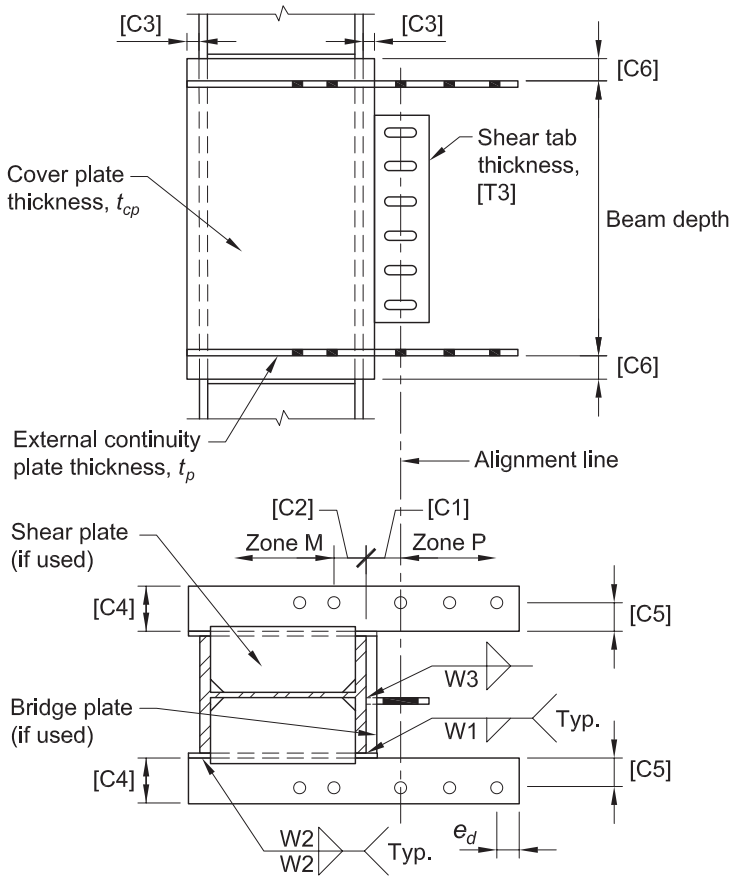


Fig. 15.3. Column assembly detail.

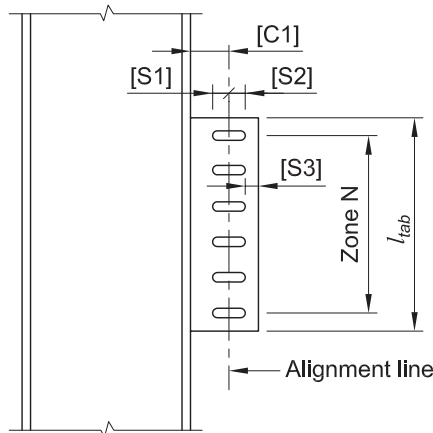


Fig. 15.4. Single-plate shear connection detail.

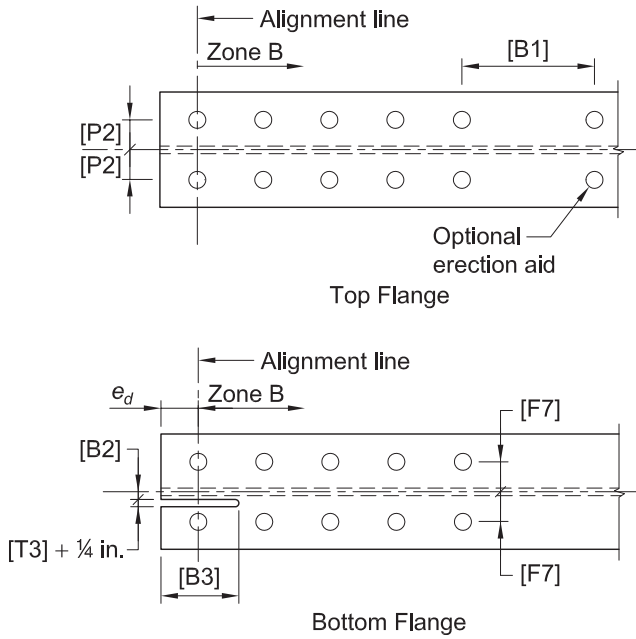


Fig. 15.5. Beam end detail.

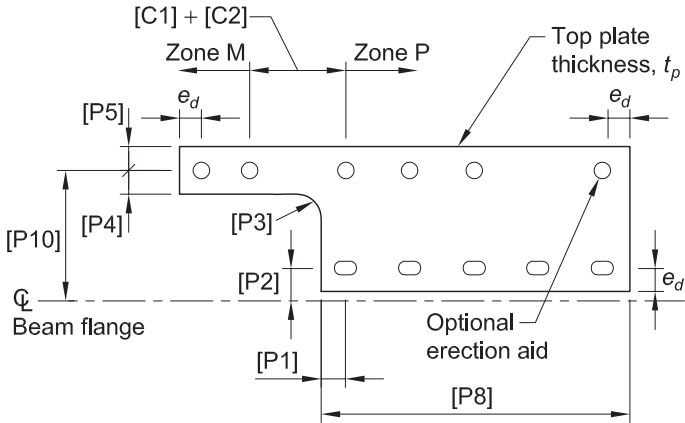


Fig. 15.6. Top plate detail.

the end of the workable flat. [W1] is omitted when the walls of an HSS or box column function as sole cover plates.

2. Bolts

- (1) Bolts shall be arranged symmetrically about the axis of the beam and shall be limited to two lines of bolts in each beam flange.
- (2) Standard holes shall be used in the beam flanges and in the external continuity plates. Holes in the top plates and fuse plate shall be oversized except for the short-slotted holes indicated in Figure 15.6.
- (3) Bolt holes in beam flanges and plates shall be made by drilling, sub-punching and reaming, laser cutting, plasma cutting, or water-jet cutting.
- (4) All bolts shall be installed as pretensioned high-strength bolts.
- (5) Bolts shall be ASTM F3125/F3125M Grades A325, A325M, A490, A490M, F1852, or F2280 assemblies. Threads shall be excluded from the shear plane. The bolt diameter shall not exceed 1¼ in. (30 mm).
- (6) Faying surfaces shall have a Class A slip coefficient but shall not have a slip coefficient greater than 0.50 (Class B).
- (7) A maximum gap of ¼ in. (6 mm) between the top plate and the external continuity plate or beam flange and between the fuse plate and the external continuity plate or beam flange is permitted and may be closed through deformation of the plates. Shim plates with a maximum overall thickness of ¼ in. (6 mm) are permitted.

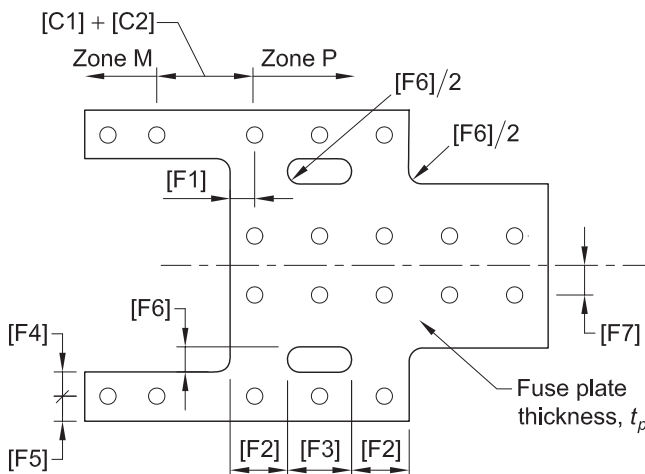


Fig. 15.7. Fuse plate detail.

3. Beam Slot

Beam bottom flanges are permitted to be slotted in accordance with Figure 15.5. The distance between the edge of the bolt holes and the cut shall be at least $\frac{3}{8}$ in. (10 mm).

User Note: The purpose of the slot in the beam bottom flange is to permit the beam to drop down past the single-plate shear connection during erection.

4. Plate Fabrication and Repair

The roughness of all thermally cut surfaces shall be no greater than an ANSI surface roughness of 1000 $\mu\text{in.}$ (25 $\mu\text{m.}$). Gouges and notches in the protected zone shall be repaired in accordance with AWS D1.8/D1.8M.

15.6 DESIGN PROCEDURE

Step 1. Choose trial sizes for the beam section(s) and column section. Confirm that the column-beam moment ratio is satisfied per Section 15.4.

Step 2. Compute the probable maximum force that will develop at the bottom flange level on each side, V_{fe} :

$$V_{fe} = \frac{M_{pr}}{2(d_b + t_p)} \quad (15.6-1)$$

where

M_{pr} = probable maximum moment at the fuse location, kip-in. (N-mm). This moment shall not exceed M_p and shall not be less than the beam flexural demand defined by the applicable building code. If M_{pr} is less than M_p , analysis shall be performed to demonstrate the connection is fully restrained.

User Note: The fuse plate is proportioned in Step 19 to have an expected ultimate capacity corresponding to M_{pr} .

d_b = depth of the beam, in. (mm)

t_p = thickness of the top plates, fuse plate, and external continuity plates, in. (mm). This thickness is typically equal to the beam flange thickness, t_{bf} , rounded up to the next standard plate size.

Step 3. Design the cover plates. Compute the required cover plate shear strength:

$$R_{u,cp} = \sum (V_{fe} + P_d) \quad (15.6-2)$$

where

P_d = drag force to be transmitted through the cover plate, computed as the beam axial force defined by the applicable building code divided by 2, kips (N)

User Note: The drag force is divided by 2 because P_d is the force resisted by one side of the connection (one of the two cover plates).

Check the limit state of shear yielding of the cover plates according to AISC *Specification* Section J4.2.

User Note: Because the cover plates will have shear strength greater than ΣV_{fe} , the panel-zone check from the AISC *Seismic Provisions* will be satisfied.

User Note: For built-up box and HSS columns, the sides of the column may function as cover plates.

The thickness of the cover plate, t_{cp} , shall conform to

$$t_{cp} \geq \frac{(d_z + w_z)}{90} \quad (15.6-3)$$

where

d_z = depth of the web of the deeper beam at the connection, in. (mm)

$$= d_b - 2t_{bf}$$

w_z = width of the panel zone between column flanges, in. (mm)

$$= d_c - 2t_{cf}$$

For flanged cruciform columns without cover plates, the required shear strength of the panel zone shall be $2R_{u,cp}$.

User Note: $R_{u,cp}$ is the demand on one cover plate. The total panel-zone shear demand is $2R_{u,cp}$.

Step 4. Determine the maximum bolt diameter. Beam plastic hinging or net section fracture is not expected to occur, but to ensure that beam yielding will occur prior to fracture, the net section of the beam shall satisfy the following:

$$Z_{x,net}R_tF_{ub} \geq Z_xR_yF_{yb} \quad (15.6-4)$$

where

F_{ub} = specified minimum tensile strength of beam, ksi (MPa)

R_t = ratio of expected tensile strength to the specified minimum tensile strength

$Z_{x,net}$ = plastic section modulus of the net section of the beam, in.³ (mm³)

User Note: ASTM F3125/F3125M Grade F2280 bolts, or similar, with a diameter of 1½ in. (28 mm) are reasonable for most beams weighing between 100 and 200 lb/ft (150 and 300 kg/m). A smaller bolt diameter may be more appropriate for lighter beams or a bigger bolt diameter for heavier beams.

$Z_{x,net}$ of the beam may be computed by accounting for only the holes in the tension flange, or more simply $Z_{x,net}$ of the beam may be computed by accounting for the holes in both flanges. Note that if the former approach is employed, the plastic neutral axis will not be at the mid-depth of the beam.

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates, n_b . The number of bolts required for the bolt shear limit state is determined as follows:

$$n_b \geq \frac{V_{fe} + p_d}{\phi r_n} \quad (15.6-5)$$

where

ϕr_n = design shear strength of one bolt, assuming threads excluded from shear plane, kips (N)

Some of the bolts are on or to the right of the alignment line (Figure 15.3). The number of bolts on and ahead of the alignment line on the outside lines is n_p (see Figure 15.6) and shall be sufficient to accommodate the depth of two fuse regions on the fuse plate with 3 in. (75 mm) spacing between.

$$n_p \geq \frac{2[F2_a] + 3 \text{ in.} - 2e_d}{s} + 1 \quad (15.6-6)$$

$$n_p \geq \frac{2[F2_a] + 75 \text{ mm} - 2e_d}{s} + 1 \quad (15.6-6M)$$

where

$[F2_a]$ = the approximate longitudinal dimension of each fuse region on the plate,
in. (mm)
= $V_{fe} / [2(0.6R_{tp}F_{up}t_p)]$ (15.6-7)
 F_{up} = specified minimum tensile strength of the plate, ksi (MPa)
 R_{tp} = tensile overstrength factor of the plate, ksi (MPa)
 e_d = minimum edge distance for bolt in oversized hole, in. (mm)
 s = bolt spacing, in. (mm)

The number of bolts behind the alignment line on the outside lines, n_m , is

$$n_m = n_b - n_p \quad (15.6-8)$$

where

n_b = total number of bolts on the lines of the fuse plate
 n_p = number of bolts on or ahead of the alignment line on each outside line of the fuse plate

The nominal slip resistance of each line of bolts using the slip coefficient determined in accordance with Section 15.5.2 and a resistance factor of 1.00 shall be less than V_{fe} .

User Note: DuraFuse Frames moment connections accommodate inelastic deformations through a combination of fuse plate yielding and bolt slip. Because the number of bolts is based on bearing capacity, this check is performed to ensure that bolt slip will occur as part of the inelastic response.

Check bolts for the limit states of bearing and tearout according to AISC *Specification* Chapter J.

Step 6. Locate the alignment line. The beam-to-column gap and the single-plate shear connection slots in the connection shall accommodate a beam rotation, γ_{max} , of at least 0.06 rad.

$$\gamma_{max} \geq 0.06 \text{ rad} \quad (15.6-9)$$

The alignment line is shown in Figures 15.3, 15.4, and 15.5. The distance from the column face to the alignment line [C1] is

$$[C1] \geq \gamma_{max} d_b + e_d + \alpha [C3] \quad (15.6-10)$$

where

- α = factor that accounts for overlap of beam flange and cover plates
- = 1.0, if the beam flange is wider than the column flange
- = 0, if the beam flange width is equal to or less than the column flange width
- [C3] = distance from the column face to the edge of the cover plate, in. (mm)

Step 7. When cover plates are used, size the cover plate-to-column flange weld, [W1]. The moment in the plane of the cover plate causes shear demands in the cover plate-to-column flange weld, r_{uv1} . A moment in the plane of the external continuity plates causes normal forces in the cover plate-to-column flange weld, r_{un1} .

$$r_{uv1} = \frac{\sum (V_{fe} + P_d)(d_b + t_p)}{d_c(d_b + 2[C6])} \quad (15.6-11)$$

$$r_{un1} = \frac{\sum (V_{fe} + P_d)[C5]}{l_{we1}d_c} \quad (15.6-12)$$

where

- l_{we1} = length at each end of the weld between the cover plates and the column flanges that is considered effective for resisting normal forces, in. (mm).
- = $t_p + t_{cp} + [C6]$ (15.6-13)

[C5] = distance from the cover plate to the line of bolts on the external continuity plates, in. (mm). See Figure 15.3.

[C6] = distance of the cover plate extension above and below the external continuity plates, in. (mm). See Figure 15.3.

The resultant demand is r_{u1} :

$$r_{u1} = \sqrt{r_{uv1}^2 + r_{un1}^2} \quad (15.6-14)$$

Alternatively, weld demands may be determined using the instantaneous center of rotation method.

Check weld strength according to AISC *Specification* Chapter J.

User Note: For built-up box and HSS columns where the sides of the column function as cover plates, [W1] is not required. For flanged cruciform columns without cover plates, [W1] is not required.

Step 8. Size the external continuity plate-to-cover plate weld, [W2]. The applied forces on each external continuity plate result in shear, r_{uv2} , and normal, r_{un2} , demands on the weld.

$$r_{uv2} = \frac{\sum (V_{fe} + P_d)}{(d_c + 2[C3])} \quad (15.6-15)$$

$$r_{un2} = \frac{\sum (V_{fe} + P_d)[C5]}{l_{we2}(d_c + 2[C3]) - 2l_{holdback} - l_{we2}} \quad (15.6-16)$$

where

$l_{holdback}$ = length of the weld holdback, taken as the weld size [W2], in. (mm)

l_{we2} = the length at each end of the weld between the external continuity plates and the cover plates that is considered effective for resisting normal forces, in. (mm)

$$= t_{fc} + t_{cp} + [C3] - l_{holdback} \quad (15.6-17)$$

The resultant demand is r_{u2} :

$$r_{u2} = \sqrt{r_{uv2}^2 + r_{un2}^2} \quad (15.6-18)$$

Alternatively, weld demands may be determined using the instantaneous center of rotation method. Check weld strength according to AISC *Specification* Chapter J.

User Note: See the AISC *Manual* Part 8 for discussion of the instantaneous center of rotation method and design tables.

For flanged cruciform columns, [W2] demands are determined considering $V_{fe} + P_d$ loading on the external continuity plate in both orthogonal directions acting simultaneously.

Step 9. Check external continuity plates for the limit state of tensile rupture in the net section according to AISC *Specification* Chapter D. Three modes shall be checked, corresponding to rupture through the bolt hole on the alignment line and the next two bolt holes in the direction of the column. For each check, the tension demand is $V_{fe} + P_d$ multiplied by the percentage of bolts that are ahead of the bolt in question— n_p/n_b , $(n_p + 1)/n_b$, or $(n_p + 2)/n_b$ —that have transferred load into the external

continuity plate. Check external continuity plates for the limit state of block shear rupture according to AISC *Specification* Chapter J.

For flanged cruciform columns, continuity plate demands are determined considering $V_{fe} + P_d$ loading on the external continuity plate in both orthogonal directions acting simultaneously.

Step 10. The required shear strength, V_u , for the beam and single-plate shear connection shall be determined in accordance with Section 2.5.

Check the design shear strength of the beam according to AISC *Specification* Chapter G.

Step 11. Compute the required block shear rupture strength of the beam flange

$$R_{u,bf} = 2(V_{fe} + P_d) \quad (15.6-20)$$

Check the design strength of the beam flange for the limit state of block shear rupture according to AISC *Specification* Section J4.3.

Step 12. Determine the required number of shear plate bolts, n_n , for the limit state of bolt shear:

$$n_n \geq \frac{V_u}{\phi r_n} \quad (15.6-21)$$

The required length of the shear plate is determined as follows:

$$l_{tab} \leq T - 1 \text{ in.} \quad (15.6-22)$$

$$l_{tab} \leq T - 25 \text{ mm} \quad (15.6-22M)$$

where

T = flat distance on the beam web, in. (mm)

The 1 in. (25 mm) buffer is to ensure that the single-plate shear connection remains clear of the k -area during seismic response.

User Note: The single-plate shear connection is as long as possible, without encroaching on the k -area of the beam, to maximize connection stiffness.

Step 13. The required shear strength of the single-plate shear connection is V_u (Step 10). Compute the required normal strength of the single-plate shear connection:

$$P_u = n_n F_b \quad (15.6-23)$$

where

F_b = maximum slip force that can develop in one bolt, kips (N), calculated as 1.5 times the nominal slip capacity assuming Class A faying surface.

Compute the required flexural strength of the single-plate shear connection:

$$M_u = V_u[C1] \quad (15.6-24)$$

where

[C1] = distance from the column face to the single-plate shear connection bolts,
in. (mm)

Design the shear connection for the required strength meeting the requirements of the *AISC Specification*.

Step 14. Determine the single-plate shear connection slot dimensions. The single-plate shear connection slot dimension, [S1], shall accommodate the maximum beam rotation from Step 6 for the critical case where the center of rotation is assumed at the top flange level.

$$[S1] \geq \gamma_{max} \left(\frac{d_b}{2} + \frac{l_{tab}}{2} - e_d \right) + \frac{d_{bolt}}{2} \quad (15.6-25)$$

where

[S1] = distance from alignment line to inside edge of slot, in. (mm); see
Figure 15.4

d_{bolt} = diameter of the single-plate shear connection bolt, in. (mm)

e_d = edge distance at bottom of single-plate shear connection, in. (mm)

l_{tab} = length of the shear connection plate, in. (mm)

γ_{max} = maximum beam rotation considered for design, rad

The slot dimension [S2], from the alignment line to the outside edge of the slot, is typically the same as [S1].

$$[S1] = [S2] \quad (15.6-26)$$

Step 15. Compute the required shear strength of the top plates, $V_{u,tp}$:

$$V_{u,tp} = V_{fe} + P_d \quad (15.6-27)$$

Check the top plate for shear yielding and shear rupture according to *AISC Specification* Section J4.2.

User Note: If the top plate has insufficient shear capacity, the plate can be thickened, the bolt size can be reduced, or the bolt spacing can be increased.

Step 16. Compute the required tensile strength of the top plates in the narrow portion:

$$R_{u,tpn} = \left(\frac{n_m}{n_b} \right) (V_{fe} + P_d) \quad (15.6-28)$$

Check the top plate for the limit states of tensile yielding and tensile rupture in the net section of the narrow portion according to *AISC Specification* Chapter D.

Step 17. Check that [P2], as shown in Figure 15.6, is sufficient for entering and tightening requirements:

$$[P2] \geq k_1 + d_h \quad (15.6-29)$$

where

k_1 = distance from web center line to flange toe of fillet, in. (mm)

d_h = hole diameter used for rupture limit state calculations, in. (mm)

Check that [P2] is sufficient to preclude yielding of the top plate due to combined flexure and shear:

$$[P2] \geq [P10] - \frac{(0.9F_{yp})e(m+e)t_p}{V_{fe} + P_d} \quad (15.6-30)$$

where

F_{yp} = specified minimum yield stress of end-plate material, ksi (MPa)

[P10] = distance from beam flange centerline to outer bolt line, in. (mm); see Figure 15.6

$$e = \frac{[P8] - m}{2} \quad (15.6-31)$$

[P8] = inside length of the top plate, in. (mm); see Figure 15.6

$$m = \frac{V_{fe} + P_d}{0.9(0.6F_{yp})t_p} \quad (15.6-32)$$

Step 18. The width of the yielding regions in the fuse plates, [F6], shall not exceed 4 in. (100 mm) and shall not be less than 1½ in. (38 mm). The width-to-thickness ratio, [F6]/[T2], of the yielding region in the fuse plates shall not exceed 4.25 and shall not be less than 1.5.

Compute the required strength of the fuse plate ahead of the first yielding region (YR):

$$R_{u,fp} = 2 \left(\frac{n_m}{n_b} \right) (V_{fe} + P_d) \quad (15.6-33)$$

Check the fuse plate for the limit states of tensile yielding and tensile rupture in the net section ahead of the first YR according to AISC *Specification* Chapter D.

User Note: The expected tensile strength of the plate, $R_{tp}F_{up}$, may be used when calculating the capacity for rupture of the net section.

Check the fuse plate for edge distances according to AISC *Specification* Chapter J.

Step 19. Determine the yielding region (YR) depth, [F2]. The YR depth is determined such that the maximum force that will develop in the strain-hardened regions on one side of the fuse plate will not exceed V_{fe} . The expected strain-hardening in the

fuse is a function of the width-to-depth ratio and the width-to-thickness ratio. The formula for [F2] that accounts for strain hardening is

$$[F2] = \frac{0.95V_{fe} + 2t_p(0.6R_{tp}F_{up})B[F6]}{2t_p(0.6R_{tp}F_{up})\left(A - \frac{C[F6]}{t_p}\right)} \quad (15.6-34)$$

where

$A = 1.52$, a coefficient in the strain hardening expression, calibrated from experiments

$B = 0.16$, a coefficient in the strain hardening expression, calibrated from experiments

$C = 0.09$, a coefficient in the strain hardening expression, calibrated from experiments

The width-to-depth ratio of the yielding regions in the fuse plates, $[F6]/[F2]$, shall not exceed 1.25 and shall not be less than 0.5.

Step 20. The required tensile strength of the fuse plates in the narrow extensions is:

$$R_{u,fpn} = \left(\frac{n_m}{n_b}\right)(V_{fe} + P_d) \quad (15.6-35)$$

Check the fuse plate for the limit states of tensile yielding and tensile rupture in the net section of the narrow extension according to AISC *Specification* Chapter D.

User Note: The expected tensile strength, $R_{tp}F_{up}$, of the plate may be used when calculating the capacity for rupture of the net section for the fuse plate extension.

APPENDIX A

CASTING REQUIREMENTS

A1. CAST STEEL GRADE

Cast steel grade shall be in accordance with ASTM A958/A958M Grade SC8620 class 80/50.

A2. QUALITY CONTROL (QC)

1. Inspection and Nondestructive Testing Personnel

Visual inspection and nondestructive testing shall be conducted by the manufacturer in accordance with a written practice by qualified inspectors. The procedure and qualification of inspectors is the responsibility of the manufacturer. Qualification of inspectors shall be in accordance with ASNT-TC-1a or an equivalent standard. The written practice shall include provisions specifically intended to evaluate defects found in cast steel products. Qualification shall demonstrate familiarity with inspection and acceptance criteria used in evaluation of cast steel products.

2. First Article Inspection (FAI) of Castings

The first article is defined as the first production casting made from a permanently mounted and rigged pattern. FAI shall be performed on the first casting produced from each pattern. The first article casting dimensions shall be measured and recorded. FAI includes visual inspection in accordance with Section A2.3, nondestructive testing in accordance with Section A2.4, tensile testing in accordance with Section A2.6, and Charpy V-notch testing in accordance with Section A2.7.

3. Visual Inspection of Castings

Visual inspection of all casting surfaces shall be performed to confirm compliance with ASTM A802/A802M and MSS SP-55 with a surface acceptance Level I.

4. Nondestructive Testing (NDT) of Castings

4a. Procedures

Radiographic testing (RT) shall be performed by quality assurance (QA) according to the procedures prescribed in ASTM E446 and ASTM E186 with an acceptance Level III or better.

Ultrasonic testing (UT) shall be performed by QA according to the procedures prescribed by ASTM A609/A609M Procedure A with an acceptance Level 3, or better.

Magnetic particle testing (MT) shall be performed by QA according to the procedures prescribed by ASTM E709 with an acceptance Level V, or better, in accordance with ASTM A903/A903M.

4b. Required NDT

(1) First Article

RT and MT shall be performed on the first article casting.

(2) Production Castings

UT shall be performed on 100% of the castings.

MT shall be performed on 50% of the castings.

(3) Reduction of Percentage of UT

The UT rate is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The UT rate may be reduced to 25%, provided the number of castings not conforming to Section A2.4a is demonstrated to be 5% or less. A sampling of at least 40 castings shall be made for such reduction evaluation. This reduction is not permitted for castings with weld repairs.

(4) Reduction of Percentage of MT

The MT rate is permitted to be reduced if approved by the engineer of record and the authority having jurisdiction. The MT rate may be reduced to 10%, provided the number of castings not conforming to Section A2.4a is demonstrated to be 5% or less. A sampling of at least 20 castings shall be made for such reduction evaluation. This reduction is not permitted for castings with weld repairs.

5. Weld Repair Procedures

Castings with discontinuities that exceed the requirements of Section A2.4a shall be weld repaired. Weld repair of castings shall be performed in accordance with ASTM A488/A488M. The same testing method that discovered the discontinuities shall be repeated on repaired castings to confirm the removal of all discontinuities that exceed the requirements of Section A2.4a.

6. Tensile Requirements

Tensile tests shall be performed for each heat in accordance with ASTM A370 and ASTM A781/A781M.

7. Charpy V-Notch (CVN) Requirements

CVN testing shall be performed in accordance with ASTM A370 and ASTM A781/A781M. Three notched specimens shall be tested with the first heat and with each subsequent 20th ton (18 000 kg) of finished material. The specimens shall have a minimum CVN toughness of 20 ft-lb (27 J) at 70°F (21°C).

8. Casting Identification

The castings shall be clearly marked with the pattern number and a unique serial number for each individual casting providing traceability to heat and production records.

A3. MANUFACTURER SUBMITTALS

The following documents shall be submitted to the engineer of record and authority having jurisdiction prior to the initiation of production:

- (1) Material chemical composition report
- (2) First article inspection report

The following documents shall be submitted to the engineer of record and the authority having jurisdiction, prior to, or with shipment as applicable:

- (a) Production inspection and NDT reports
- (b) Tensile and CVN test reports
- (c) Weld repair reports

APPENDIX B

FORGING REQUIREMENTS

B1. FORGED STEEL GRADE

Raw material shall conform to the requirements of ASTM A572/A572M, Grade 50 (Grade 345). The forging process shall conform to the requirements of ASTM A788 and ASTM A668. Mechanical properties shall conform to the requirements of Table B1.1.

B2. BAR STOCK

Bar stock shall be cut to billets appropriate to the part being forged. All billets shall be marked with the heat number.

B3. FORGING TEMPERATURE

Billets shall be forged at a minimum temperature of 2,150° F (1,180° C) and a maximum temperature of 2,250° F (1,230° C).

B4. HEAT TREATMENT

Immediately following impression forging, the part being forged shall be normalized for 1 hr at 1,650 °F (899 °C) then air cooled.

B5. FINISH

Finished forgings shall have shot blast finish, clean of mill scale.

B6. QUALITY ASSURANCE

One sample of bar stock from each heat shall be cut to a length of 6 in. (150 mm) and forged to a 5-in. by 2-in.-thick bar (125 mm by 50 mm). Samples shall be marked with longitudinal and transverse directions. Chemistry and physical properties in accordance with Table B1.1 shall be verified to ASTM A572/A572M Grade 50 (Grade 345) for both the longitudinal and transverse directions on each sample.

Magnetic particle testing shall be conducted on the initial 12 pieces from each run to verify tooling and forging procedures. Cracks shall not be permitted. If cracks are found, the tooling or forging procedure shall be modified, and an additional 12 initial pieces shall be tested. This process shall be repeated until 12 crack-free samples are obtained prior to production.

TABLE B1.1
Required Mechanical Properties

Yield strength	50 ksi (345 MPa) minimum
Tensile strength	65 ksi (450 MPa) minimum
Elongation in 2 in. (50 mm)	22% minimum
Reduction of area	38% minimum
Charpy V-notch toughness	20 ft-lb at 70° F (27 J at 21° C)

B7. DOCUMENTATION

Laboratory test data documenting chemistry, strength, elongation, reduction of area, and Charpy requirements for the samples tested in accordance with Section B6 shall be submitted by the manufacturer to the purchaser.

Inspection reports documenting satisfactory performance of magnetic particle tests per Section B6 shall be submitted by the manufacturer to the purchaser.

Certification of conformance with the requirements of this appendix shall be submitted by the manufacturer to the purchaser and to the engineer of record.

COMMENTARY

on Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications

August 18, 2022

This Commentary is not part of ANSI/AISC 358-22, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*. It is included for informational purposes only.

INTRODUCTION

The standard is intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations, and limits of the standard.

The standard and Commentary are intended for use by design professionals with demonstrated engineering competence.

CHAPTER 1

GENERAL

1.1. SCOPE

Design of special moment frames (SMF) and intermediate moment frames (IMF) in accordance with the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2022a), hereafter referred to as the AISC *Seismic Provisions*, and applicable building codes includes an expectation that they will experience substantial inelastic deformations when subjected to design-level earthquake ground shaking, generally concentrated at the moment-resisting beam-to-column connections. In the 1994 Northridge earthquake, a number of steel moment frame buildings were found to have experienced brittle fractures that initiated at the welded beam flange-to-column flange joints of moment connections. These brittle fractures were unexpected and were quite different from the anticipated behavior of ductile beam flexural yielding in plastic hinge zones. Where they occurred, these brittle fractures prevented the formation of ductile plastic hinge zones and resulted in frame behavior substantially different from that upon which the design requirements for these systems were based.

Following this discovery, the Federal Emergency Management Agency (FEMA) provided funding to a coalition of universities and professional associations, known as the SAC Joint Venture. Over a period of six years, the SAC Joint Venture, with participation from AISC, AISI, AWS, and other industry groups, conducted extensive research into the causes of the damage that had occurred in the Northridge earthquake and effective means of reducing the possibility of such damage in future earthquakes.

Numerous issues were identified in the SAC studies as contributing causes of these brittle fractures. This standard specifically addresses the following four causes that were identified in the SAC study:

- (1) Connection geometries that resulted in large stress concentrations in regions of high triaxiality and limited ability to yield.
- (2) Use of weld filler metals with low inherent notch toughness and limited ductility.
- (3) High variability in the yield strengths of beams and columns resulting in unanticipated zones of weakness in connection assemblies.
- (4) Welding practice and workmanship that fell outside the acceptable parameters of the AWS D1.1/D1.1M, *Structural Welding Code*, at that time.

A more complete listing of the causes of damage sustained in the Northridge earthquake may be found in a series of publications [FEMA 350 (FEMA, 2000b), FEMA 351 (FEMA, 2000c), FEMA 352 (FEMA, 2000a), FEMA 353 (FEMA, 2000d), FEMA 355C (FEMA, 2000f), and FEMA 355D (FEMA, 2000e)] published in 2000

by the SAC Joint Venture that presented recommendations for design and construction of moment resisting frames designed to experience substantial inelastic deformation during design ground shaking. These recommendations included changes to material specifications for base metals and weld filler metals, improved quality assurance procedures during construction, and the use of connection geometries that had been demonstrated by testing and analysis to be capable of resisting appropriate levels of inelastic deformation without fracture. Most of these recommendations have been incorporated into the *AISC Seismic Provisions* as well as into *AWS D1.8/D1.8M, Structural Welding Code—Seismic Supplement* (AWS, 2021).

Following the SAC Joint Venture recommendations, the *AISC Seismic Provisions* required that moment connections used in special or intermediate steel moment frames be demonstrated by testing to be capable of providing the necessary ductility. Two means of demonstration are acceptable. One means consists of project-specific testing in which a limited number of full-scale specimens, representing the connections to be used in a structure, are constructed and tested in accordance with a protocol prescribed in *AISC Seismic Provisions* Section K2. Recognizing that it is costly and time consuming to perform such tests, the *AISC Seismic Provisions* also provide for prequalification of connections. The specific criteria for prequalification of beam-to-column connections are prescribed in *AISC Seismic Provisions* Section K1 and include a rigorous program of testing, analytical evaluation, and review by an independent body, the Connection Prequalification Review Panel (CPRP). Connections contained in this standard have met these criteria for prequalification when applied to framing that complies with the limitations contained herein and when designed and detailed in accordance with this standard.

1.2. REFERENCES

References for this standard are listed at the end of the Commentary.

1.3. GENERAL

Connections prequalified under this standard are intended to withstand inelastic deformation primarily through controlled yielding or other ductile response mechanisms in specific behavioral modes. To obtain connections that will behave in the indicated manner, proper determination of the strength of the connection in various limit states is necessary. The strength formulations contained in the LRFD method are consistent with this approach.

CHAPTER 2

DESIGN REQUIREMENTS

2.1. SPECIAL AND INTERMEDIATE MOMENT FRAME CONNECTION TYPES

Limitations included in this standard for various prequalified connections include specification of permissible materials for base metals, mechanical properties for weld filler metals, member shape and profile, connection geometry, detailing, and workmanship. These limitations are based on conditions demonstrated by testing and analytical evaluation for which reliable connection behavior can be attained. It is possible that these connections can provide reliable behavior outside these limitations; however, this has not been demonstrated. When any condition of base metal, mechanical properties, weld filler metals, member shape and profile, connection geometry, detailing, or workmanship falls outside the limitations specified herein, project-specific qualification testing should be performed to demonstrate the acceptability of connection behavior under these conditions.

Limited testing of connections of I-shaped beams to the webs of I-shaped columns had been conducted prior to the Northridge earthquake by Tsai and Popov (1986, 1988) and others. This testing demonstrated that these “minor-axis” connections were generally incapable of developing reliable inelastic behavior. Only very limited testing of such minor-axis connections has been conducted since the Northridge earthquake. Consequently, the majority of connections in this standard are not prequalified for I-shaped beams framing into the web of an I-shaped column. The exceptions to this are the SidePlate moment connection and the DuraFuse Frames moment connection. Both of these connections have prequalified configurations for connecting an I-shaped beam to the web of an I-shaped column.

Although there has been only limited testing of connections in assemblies subjected to biaxial bending of the column, the judgment of the CPRP was that as long as columns are designed in accordance with the AISC *Seismic Provisions*, which allows only limited yielding in columns, it would be possible to obtain acceptable behavior of beam-column connection assemblies subjected to biaxial loading.

Nearly all moment frame connection tests have been performed on single- or double-sided beam-column subassemblies with the beam perpendicular to the vertical axis of the column (i.e., a level beam) and coplanar with the strong axis of the column (i.e., unskewed). The reality of building construction is that sloping beams occur in most structures, such as at the roof. Occasionally, skewed configurations are required to accommodate architectural requirements. This standard does not contain provisions that explicitly address sloped or skewed moment frame beams because of the wide variety of connections that have obtained prequalification and the lack of systematic

physical testing for each connection. Professional judgment, therefore, is required to determine whether a proposed frame geometry is appropriately covered by the prequalification limits in this standard.

One factor to consider when evaluating the impact of frame beam slope is the absolute angle at which the beam slopes. For example, even so-called “flat” roofs slope at approximately $\frac{1}{4}$ in./ft (21 mm/m), which equates to an angle of 1.2° . Testing of frame beams using reduced beam sections sloping at 28° (Ball et al., 2010) indicates that at this angle, the performance of the connection can be adversely affected depending on how it is configured. The former angle likely is not significant while the latter angle appears to be, at least for the reduced beam section. It is reasonable to assume that different connections will likely have different thresholds above which the slope becomes significant because of characteristics that govern connection behavior.

For example, connection geometry and limit states that govern connection behavior will likely influence the threshold above which frame beam slope becomes significant. Connection performance of sloped beams is expected to be influenced by components of differing length or changes in geometry (e.g., the relative distance from the face of column to the first bolt or the length of the flange plate). These variations may impose different levels of load on components at the top flange versus those on the bottom flange due to changes in relative stiffness. The relative onset of limit states, such as flange local buckling, prying, and similar may be influenced by the angle of slope and is expected to vary from connection type-to-connection type.

Limited analytical studies have been performed relative to the impact of beam skew on connection performance. Prinz and Richards (2016) report that frame beam skew angles between 10° and 20° in reduced beam section connections appear to cause limited increases in column torsional demand and limited additional flange tip yielding. They also report that skew angles of 10° reduce low cycle fatigue capacity in the reduced section region by less than one cycle. Thus, skew angles of less than 5° to 10° might be considered acceptable in reduced beam sections connections. Similar analytical studies have not been conducted for other connection geometries.

2.3. MEMBERS

Table C-2.1 summarizes the prequalified beam sections for each connection, and Table C-2.2 summarizes the prequalified column sections for each connection. Each connection imposes additional limitations on beam and column sections that may include size, weight per foot, and other limitations.

2b. Built-up Columns

Four built-up column cross-section shapes are covered by this standard. These are illustrated in Figure C-2.1 and include the following:

- (1) I-shaped welded columns that resemble standard rolled wide-flange shapes in cross-section shape and profile.

TABLE C-2.1
Prequalified Beam Sections

Connection	Prequalified Beam Section		
	Rolled Wide-Flange	Built-up I-Shape	HSS
RBS (Chapter 5)			
BUEEP and BSEEP (Chapter 6)			
BFP (Chapter 7)			
WUF-W (Chapter 8)			
CBB (Chapter 9)			
ConXL (Chapter 10)	√	√	
Simpson Strong-Tie Strong Frame (Chapter 12)			
Double-Tee (Chapter 13)			
SW (Chapter 14)			
DuraFuse Frames (Chapter 15)			
SidePlate (Chapter 11)	√	√	√

TABLE C-2.2
Prequalified Column Sections

Connection	Prequalified Column Section					
	Rolled Wide-Flange	Built-up I-Shape	Boxed Wide-Flange	Built-up Box	Flanged Cruciform	HSS
RBS (Chapter 5)						
BFP (Chapter 7)						
WUF-W (Chapter 8)	√	√	√	√	√	
CBB (Chapter 9)						
Slotted Web (Chapter 14)						
ConXL (Chapter 10)				√		√
BUEEP and BSEEP (Chapter 6)						
Simpson Strong-Tie Strong-Frame (Chapter 12)	√	√			√	
Double-Tee (Chapter 13)						
SidePlate (Chapter 11)						
DuraFuse Frames (Chapter 15)	√	√	√	√	√	√

- (2) Cruciform W-shape columns, fabricated by splitting a wide-flange section in half and welding the webs on either side of the web of an unsplit wide-flange section at its mid-depth to form a cruciform shape, each outstanding leg of which terminates in a rectangular flange.
- (3) Box columns, fabricated by welding four plates together to form a closed box-shaped cross section.
- (4) Boxed W-shape columns constructed by adding side plates to the sides of an I-shaped cross section.

The preponderance of connection tests reviewed as the basis for prequalifications contained in this standard consisted of rolled wide-flange beams connected to the flanges of rolled wide-flange columns. A limited number of tests of connections of wide-flange beams to built-up box-section columns were also reviewed (Anderson and Linderman, 1991; Kim et al., 2008).

The flanged cruciform column and boxed wide-flange columns have not specifically been tested. However, it was the judgment of the CPRP that as long as such column sections met the limitations for I-shaped sections and box-shaped sections, respectively, and connection assemblies are designed to ensure that most inelastic behavior occurred within the beam as opposed to the column, the behavior of assemblies employing these sections would be acceptable. Therefore, prequalification has been extended to these cross sections for connection types where the predominant inelastic behavior is in the beam rather than the column.

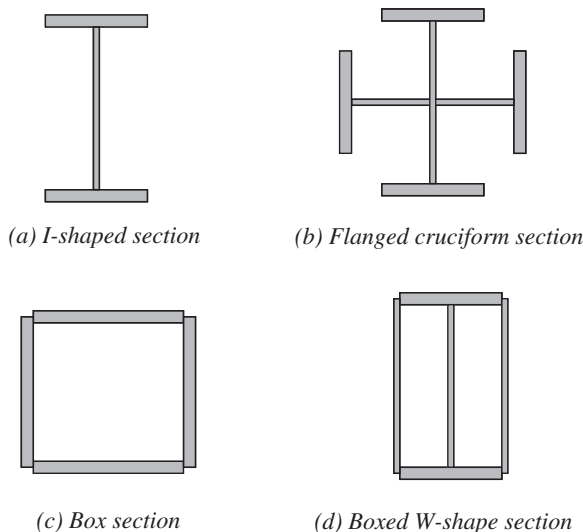


Fig. C-2.1. Column shapes. Plate preparation and welds are not shown.

4. Isolation of Concrete Slab

Contact between the concrete slab and the face of the column or the face of connection elements can increase deformation demands on the beam bottom flange connection under positive bending (slab in compression). This increased demand is the result of composite action between the slab and the beam and from an upward shift in the neutral axis location. In some cases, this can cause a premature fracture at the beam bottom flange connection. To avoid premature failure, this section requires that the concrete slab be isolated in a manner that allows the beam to rotate with minimum engagement of the slab with the face of the column or face of connection elements. This requirement can be satisfied by providing a minimum of a 1 in. (25 mm) gap between the slab and the column on both sides of each flange, as shown in Figure C-2.2. A compressible fire-resistant material can be used to fill the gap. Where a connection element is present at the face of the column, the gap should be provided between the slab and the connection element, as shown in Figure C-2.2(b).

Slab isolation is not required for the reduced beam section (RBS), welded unreinforced flange-welded web (WUF-W), SidePlate, and Slotted Web moment connections because connection tests conducted on subassemblies with a composite slab demonstrated satisfactory performance without isolation of the concrete slab.

2.4. CONNECTION DESIGN PARAMETERS

1. Resistance Factors

A significant factor considered in the formulation of resistance factors is the occurrence of various limit states. Limit states that are considered brittle (nonductile) and subject to sudden catastrophic failure are typically assigned lower resistance factors than those that exhibit yielding (ductile) failure. Because, for the prequalified connections, design demand is determined based on conservative estimates of the material strength of weak elements of the connection assembly, and because materials,

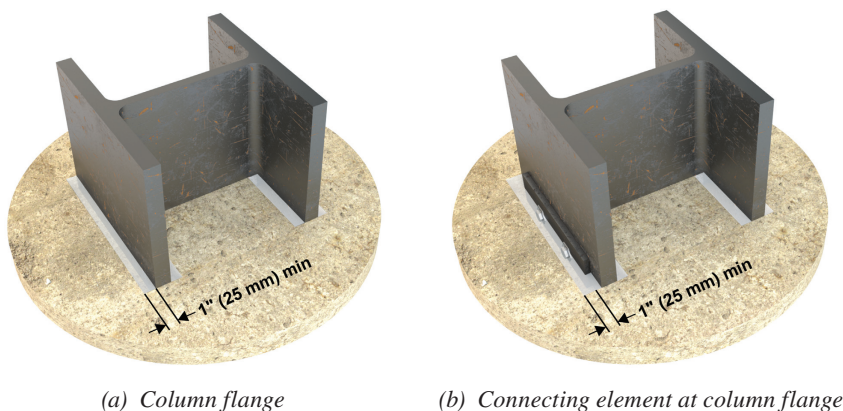


Fig. C-2.2. Separation of slab from column flange and connecting element at column flange.

workmanship, and quality assurance are more rigorously controlled than for other structural elements, resistance factors have been set somewhat higher than those traditionally used. It is believed that these resistance factors, when used in combination with the design, fabrication, erection, and quality assurance requirements contained in this standard, will provide reliable service in the prequalified connections.

2. Plastic Hinge Location

For most prequalified connections, the intended behavior under earthquake loading is that inelastic deformation is developed primarily through flexural yielding of the beam—that is, through the development of a plastic hinge in the beam. For such prequalified connections, this standard specifies the presumed location of the plastic hinge. In reality, the formation of a plastic hinge requires flexural yielding over some length of the beam. Defining a single point as the plastic hinge location is therefore an idealization of actual behavior, but it is a useful concept for purposes of analysis and design. The plastic hinge locations specified herein are based on observed behavior during connection assembly tests and analysis of the connection, and they are generally at or near the locations of the largest ratio of flexural demand-to-capacity in the beam when the frame is subjected to lateral load. For each connection, the design procedure and design parameters are adjusted based on the specified plastic hinge location.

For the Simpson Strong-Tie Strong Frame connection and the DuraFuse Frames connection, the intended behavior under earthquake loading is that inelastic deformation is developed by yielding of connection elements rather than by flexural yielding of the beam. Consequently, the concept of a plastic hinge in the beam is not applicable to these connections.

3. Probable Maximum Moment at Plastic Hinge

The probable plastic moment at the plastic hinge is intended to be a conservative estimate of the maximum moment likely to be developed in the beam at the plastic hinge location under cyclic inelastic loading. It includes consideration of likely material overstrength, accounted for by the factor R_y , and the effects of strain hardening and other factors that may affect the ultimate flexural strength of the plastic hinge, accounted for in the factor C_{pr} . The value specified for C_{pr} for each connection is based on an evaluation of connection test data.

4. Shear Force at Plastic Hinge

A number of the prequalified connections require calculation of the shear force at the plastic hinge, V_h , as defined in this section. V_h is the shear force at the plastic hinge location when the bending moment at the plastic hinge is equal to the probable maximum moment, M_{pr} , and with the applicable gravity load acting on the beam. V_h can be determined from a free-body diagram of the segment of the beam between plastic hinge locations. An example is shown in Figure C-2.3 for the case of a uniform gravity load on the beam and for the case where M_{pr} is applied in a counterclockwise direction. The shear force at each plastic hinge due to M_{pr} acting at the plastic hinge

locations is equal to $2M_{pr}/L_h$. The shear force due to the gravity load, $w_u L_h/2$ for this example, has the same sign as $2M_{pr}/L_h$ at the left end of the free-body diagram and is of the opposite sign at the other end. Consequently, the shear force at each end of the free-body diagram is different. V_h is taken as the larger of these two values, as shown in the figure. Because M_{pr} can reverse direction, V_h can occur at both ends of the free-body diagram. For gravity loads on the beam other than a uniform load, the shear force due to gravity load can be determined directly from equilibrium of the free-body diagram.

5. Probable Maximum Moment at Face of Column

A number of the prequalified connections require calculation of the probable maximum moment at the face of the column, M_f , as defined in this section. M_f is the bending moment at the face of the column when the bending moment at the plastic hinge is equal to the probable maximum moment, M_{pr} , and the shear force at the plastic hinge is V_h . M_f can be determined from a free-body diagram of the portion of the beam between the plastic hinge location and the face of the column, as illustrated in Figure C-2.4. The equation for M_f given in this figure and also shown as Equation 2.4-4 neglects the gravity load on the portion of the beam between the plastic hinge location and the face of the column because this typically has a negligible contribution to M_f .

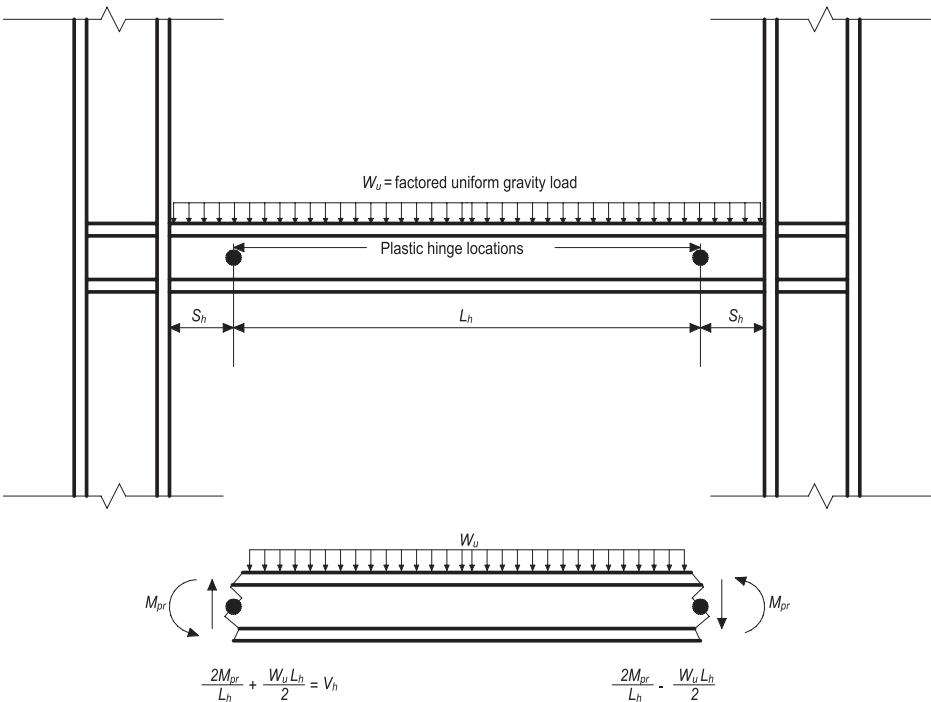


Fig. C-2.3. Free-body diagram for determination of V_h .

2.5. REQUIRED SHEAR STRENGTH OF BEAM

A number of the prequalified connections require calculation of the required shear strength of the beam, as defined in this section. Unlike the calculation for M_{pr} , the required shear strength of the beam, shown as V_u in Figure C-2.4, includes the gravity load on the portion of the beam between the plastic hinge location and the face of the column.

2.6. CONTINUITY PLATES

Beam flange continuity plates serve several purposes in moment connections. They help to distribute beam flange forces to the column web, stiffen the column web to prevent local crippling and yielding, and limit local flange bending under the concentrated beam-flange forces. Continuity plates also reduce stress concentrations that can occur in the joint between the beam flange and column due to nonuniform stiffness of the attached elements. Unless otherwise specified in the standard, continuity plates for prequalified connections follow the requirements of the AISC *Seismic Provisions* [Section E2.6f for intermediate moment frames (IMF) and Section E3.6f for special moment frames (SMF)]. The continuity plate requirements in the AISC *Seismic Provisions* are primarily applicable for an I-shaped beam connected to the flange of an I-shaped column.

No criteria for continuity plates are provided for built-up box columns in the AISC *Seismic Provisions* or in the standard, as there is insufficient research available at present to support definitive design criteria. As such, it is recommended that continuity plates, also referred to as diaphragm plates when used in box columns, be

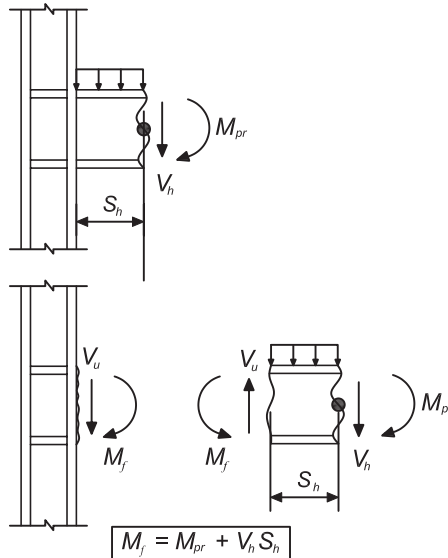


Fig. C-2.4. Free-body diagram for determination of M_j .

provided for built-up box columns unless justified otherwise by testing or analysis. During fabrication of built-up box columns, a continuity plate can be welded to three sides when the box is still open. The weld between the continuity plate and the closing side of the box column is normally made using the electro-slag welding (ESW) process. Tests have shown that premature fractures can occur at the ESW weld due to a notch condition introduced by the containment plates of the ESW weld (Lee et al., 2014). In cases where moment frame beams do not connect to all four sides of the box column, it is recommended that the ESW side of the column be located where there is no moment frame beam. Other suggestions to mitigate brittle fracture are provided by Lee et al. (2014).

When beams are moment connected to the side plates of boxed wide-flange column sections, continuity plates should be provided opposite the beam flanges unless justified otherwise by testing or analysis.

Almost all connection assembly testing has been conducted on specimens that include a significant length (typically one-half story height) of column above and below the beam or beams framing into the column. Thus, the condition that typically exists in a structure's top story, where the column terminates at the level of the beam top flange, has not specifically been tested to demonstrate acceptable detailing. A cap plate detail similar to that illustrated in Figure C-2.5 is believed to be capable of providing reliable performance when connection elements do not extend above the beam top flange. In some connections—for example, extended end-plate and cast

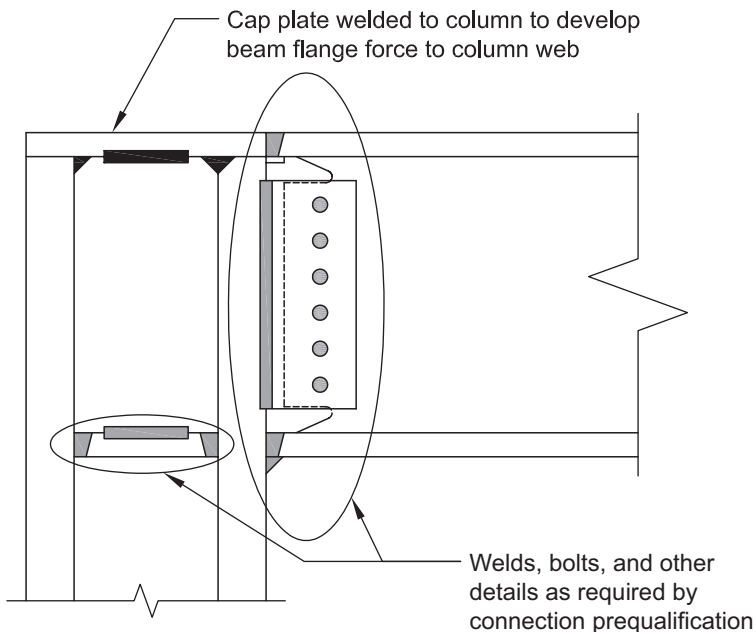


Fig. C-2.5. Example cap plate detail at column top for RBS connection.

bolted bracket connections, portions of the connection assembly extend above the column top flange. In such cases, the column should be extended to a sufficient height above the beam flange to accommodate attachment and landing of those connection elements. The continuity plates should be placed opposite the beam top flange, as is done at intermediate framing levels.

2.7. PANEL ZONES

Unless otherwise specified in the standard, panel zones for prequalified connections follow the requirements of the AISC *Seismic Provisions* (Section E2.6e for IMF and Section E3.6e for SMF). This includes determination of the required shear strength of the panel zone, panel-zone thickness limitations, and doubler-plate detailing requirements. Following these requirements may result in limited yielding of the panel zone, but the primary mechanism for developing inelastic deformation will be flexural yielding of the beam (for most prequalified connections) or yielding of connection elements (for the Simpson Strong Frame and the DuraFuse Frames connections).

2.8. COLUMN-BEAM MOMENT RATIOS

Unless otherwise specified in the standard, column-beam moment ratios for prequalified connections follow the requirements of AISC *Seismic Provisions* Section E3.4a for SMF. The AISC *Seismic Provisions* have no column-beam moment ratio requirements for IMF, and no additional requirements are imposed by this standard for prequalified connections used in IMF.

The column-beam moment ratio requirements for SMF in AISC *Seismic Provisions* Section E3.4a are primarily intended for planar frames, where there are no moment frame beams in orthogonal or multi-axial directions. For cases with moment frame beams in orthogonal or multi-axial directions, AISC *Seismic Provisions* Section E3.3 requires that the potential for beam yielding in both orthogonal directions simultaneously be considered when checking the column-beam moment ratio. Interpreting this requirement will require judgment that can be guided by AISC *Seismic Provisions* Section E3.3 Commentary.

The ConXtech ConXL connection is typically used in a biaxial configuration. This connection was tested in biaxial configurations and specific requirements for checking the column-beam moment ratio are provided in Chapter 10.

CHAPTER 3

WELDING REQUIREMENTS

3.3. BACKING AT BEAM-TO-COLUMN AND CONTINUITY PLATE-TO-COLUMN JOINTS

At the root of groove welds between beam flanges or continuity plates and column flanges, the inherent lack of a fusion plane between the left-in-place steel backing and the column flange creates a stress concentration and notch effect, even when the weld has uniform and sound fusion at the root. Further, when ultrasonic testing is performed, this left-in-place backing may mask significant flaws that may exist at the weld root. These flaws may create a more severe notch condition than that caused by the backing itself (Chi et al., 1997).

1. Steel Backing at Continuity Plates

The stress and strain level at the groove weld between a continuity plate and column flange is considerably different than that at the beam flange-to-column flange connection; therefore, it is not necessary to remove the backing. The addition of the fillet weld beneath the backing makes the inherent notch at the interface an internal notch rather than an external notch, reducing the notch effect. When backing is removed, the required reinforcing fillet weld reduces the stress concentration at the right-angle intersection of the continuity plate and the column flange.

2. Steel Backing at Beam Bottom Flange

The removal of backing, whether fusible or nonfusible, followed by backgouging to sound weld metal, is required so that potential root defects within the welded joint are detected and eliminated, and the stress concentration at the weld root is eliminated.

The influence of left-in-place steel backing is more severe on the bottom flange, as compared to the top flange, because at the bottom flange, the stress concentration from the backing occurs at the point of maximum applied and secondary tensile stresses in the groove weld, at the weld root, and at the outer fiber of the beam flange.

A reinforcing fillet weld with a $\frac{5}{16}$ in. (8 mm) leg on the column flange helps to reduce the stress concentration at the right-angle intersection of the beam flange and column flange and is placed at the location of maximum stress. The fillet weld horizontal leg may need to be larger than $\frac{5}{16}$ in. (8 mm) to completely cover the weld root area, eliminating the potential for multiple weld toes at the root that serve as small stress concentrations and potential fracture initiation points. When grinding the weld root and base metal area, previously deposited weld toe regions and their associated fracture initiation sites are removed, and the horizontal leg of the fillet weld need not be extended to the base metal.

3. Steel Backing at Beam Top Flange

Because of differences in the stress and strain conditions at the top and bottom flange connections, the stress/strain concentration and notch effect created by the backing/column interface at the top flange is at a lower level, compared to that at the bottom flange. Therefore, backing removal is not required. The addition of the reinforcing fillet weld makes the inherent notch at the interface an internal notch, rather than an external notch, further reducing the effect. Because backing removal, backgouging, and backwelding would be performed through an access hole beneath the top flange, these operations should be avoided whenever possible.

4. Prohibited Welds at Steel Backing

Tack welds for beam flange-to-column connections should be made within the weld groove. Tack welds or fillet welds to the underside of the beam at the backing would direct stress into the backing itself, increasing the notch effect at the backing/column flange interface. In addition, the weld toe of the tack weld or fillet weld on the beam flange would act as a stress concentration and a potential fracture initiation site.

Proper removal of these welds is necessary to remove the stress concentration and potential fracture initiation site. Any repair of gouges and notches by filling with weld metal must be made using filler metals with the required notch toughness.

5. Nonfusible Backing at Beam Flange-to-Column Joints

After nonfusible backing is removed, backgouging to sound metal removes potential root flaws within the welded joint. A reinforcing fillet weld with a $\frac{5}{16}$ in. (8 mm) leg on the column flange helps reduce the stress concentration at the right-angle intersection of the beam flange and column flange.

The fillet weld horizontal leg may need to be larger than $\frac{5}{16}$ in. (8 mm) to completely cover the weld root area, eliminating the potential for small stress concentrations and potential fracture initiation points. When grinding the weld root and base metal area, previously deposited weld toe regions and their associated fracture initiation sites are removed; therefore, the horizontal leg of the fillet weld need not be extended to base metal.

3.4. WELD TABS

Weld tabs are used to provide a location for initiation and termination of welds outside the final weld location, improving the quality of the final weld. The removal of weld tabs is performed to remove the weld discontinuities and defects that may be present at these start and stop locations. Because weld tabs are located at the ends of welds, any remaining weld defects at the weld-end removal areas may act as external notches and fracture initiation sites and are, therefore, removed. A smooth transition is needed between base metal and weld to minimize stress concentrations.

3.5. TACK WELDS

Tack welds outside weld joints may create unintended load paths and may create stress concentrations that become crack initiation sites when highly strained. By placing tack welds within the joint, the potential for surface notches and hard heat affected zones (HAZs) is minimized. When placed within the joint, the HAZ of a tack weld is tempered by the subsequent passes for the final weld.

Tack welds for beam flange-to-column connections are preferably made in the weld groove. Tack welds of backing to the underside of beam flanges would be unacceptable, and any tack welds between weld backing and beam flanges are to be removed in accordance with Section 3.3.4. Steel backing may be welded to the column under the beam flange, where a reinforcing fillet is typically placed.

When tack welds for the attachment of weld tabs are placed within the weld joint, they become part of the final weld.

3.6. CONTINUITY PLATES

The rotary straightening process used by steel rolling mills to straighten rolled sections cold works the webs of these shapes in and near the k -area. This cold working can result in an increase in hardness, yield strength, ultimate tensile strength, and yield-to-tensile ratio, and a decrease in notch toughness. In some instances, Charpy V-notch toughness has been recorded to be less than 2 ft-lb at 70°F [3 J at 20°C] (Barson and Korvink, 1998). These changes do not negatively influence the in-service behavior of uncracked shapes. However, the potential for post-fabrication k -area base metal cracking exists in highly restrained joints at the weld terminations for column continuity plates, web doublers, and thermal cut coped beams.

When the minimum clip dimensions are used along the member web, the available continuity plate length must be considered in the design and detailing of the welds to the web. For fillet welds, the fillet weld should be held back one to two weld sizes from each clip. For groove welds, weld tabs should not be used in the k -area because they could cause base metal fracture from the combination of weld shrinkage, the stress concentration/notch effect at the weld end, and the low notch-toughness web material.

When the maximum clip dimensions are used along the member flange, the width, hence the capacity, of the continuity plate is not reduced substantially. Care must be used in making quality weld terminations near the member radius, as the use of common weld tabs is difficult. If used, their removal in this region may damage the base metal, necessitating difficult repairs. The use of cascaded ends within the weld groove may be used within the dimensional limits stated. Because of the incomplete filling of the groove, the unusual configuration of the weld, and the relatively low level of demand placed upon the weld at this location, nondestructive testing of cascaded weld ends in groove welds at this location is not required.

3.7. QUALITY CONTROL AND QUALITY ASSURANCE

AISC *Seismic Provisions* Chapter J specifies the minimum requirements for a quality assurance plan for the seismic force-resisting system. It may be appropriate to supplement the Chapter J provisions with additional requirements for a particular project based on the qualifications of the contractor(s) involved and their demonstrated ability to produce quality work. Contract documents are to define the quality control (QC) and quality assurance (QA) requirements for the project.

QC includes those tasks to be performed by the contractor to ensure that their materials and workmanship meet the project's quality requirements. Routine welding QC items include personnel control, material control, preheat measurement, monitoring of welding procedures, and visual inspection.

QA includes those tasks to be performed by an agency or firm other than the contractor. QA includes monitoring of the performance of the contractor in implementing the contractor's QC program, ensuring that designated QC functions are performed properly by the contractor on a routine basis. QA may also include specific inspection tasks that are included in the contractor's QC plan, and may include nondestructive testing of completed joints.

CHAPTER 4

BOLTING REQUIREMENTS

4.1. FASTENER ASSEMBLIES

ASTM F3125/F3125M Grade F1852 twist-off type tension-control fastener assemblies are appropriate equivalents for ASTM F3125/F3125M Grade A325 or A325M bolts. ASTM F3125/F3125M Grade F2280 twist-off type tension control fastener assemblies are appropriate substitutes for ASTM F3125/F3125M Grade A490 or A490M bolts. Such assemblies are commonly produced and used and are addressed by the RCSC *Specification for Structural Joints Using High-Strength Bolts* (RCSC, 2020).

4.2. INSTALLATION REQUIREMENTS

AISC *Seismic Provisions* Section D2 requires all bolts in the seismic force-resisting system to be pretensioned, with the additional requirement that the joint's faying surfaces meet Class A conditions for slip-critical joints. Unless noted otherwise, however, bolted joints need not be designed as slip critical. Some bolted joints in prequalified connections waive the faying surface requirements of the AISC *Seismic Provisions*.

4.3. QUALITY CONTROL AND QUALITY ASSURANCE

See Commentary Section 3.7.

CHAPTER 5

REDUCED BEAM SECTION (RBS) MOMENT CONNECTION

5.1. GENERAL

In a reduced beam section (RBS) moment connection, portions of the beam flanges are selectively trimmed in the region adjacent to the beam-to-column connection. In an RBS connection, yielding and hinge formation are intended to occur primarily within the reduced section of the beam and thereby limit the moment and inelastic deformation demands developed at the face of the column.

A large number of RBS connections have been tested under a variety of conditions by different investigators at institutions throughout the world. A listing of relevant research is presented in the references at the end of this document. Review of available test data indicates that RBS specimens, when designed and constructed according to the limits and procedures presented herein, have developed interstory drift angles of at least 0.04 rad under cyclic loading on a consistent basis. Tests on RBS connections show that yielding is generally concentrated within the reduced section of the beam and may extend, to a limited extent, to the face of the column. Peak strength of specimens is typically achieved at an interstory drift angle of approximately 0.02 to 0.03 rad. Specimen strength then gradually reduces due to local and lateral-torsional buckling of the beam. Ultimate failure typically occurs at interstory drift angles of approximately 0.05 to 0.07 rad, by low cycle fatigue fracture a local flange buckles within the RBS or near the face of the column.

RBS connections have been tested using single-cantilever type specimens (one beam attached to a column) and double-sided specimens (specimen consisting of a single column with beams attached to both flanges). Tests have been conducted primarily on bare-steel specimens, although some testing is also reported on specimens with composite slabs. Tests with composite slabs have shown that the presence of the slab provides a beneficial effect by helping to maintain the stability of the beam at larger interstory drift angles.

Most RBS test specimens were tested pseudo-statically, using a loading protocol in which applied displacements are progressively increased, such as the loading protocol specified in ATC-24 (ATC, 1992) and the loading protocol developed in the FEMA/SAC program and adopted in Chapter K of the AISC *Seismic Provisions*. Two specimens were tested using a loading protocol intended to represent near-source ground motions that contain a large pulse. Several specimens were also tested dynamically. Radius-cut RBS specimens have performed well under all of these loading conditions.

5.2. SYSTEMS

Review of the research literature presented in the reference section at the end of this document and summarized in Commentary Section 5.1 indicates that the radius-cut RBS connection meets the prequalification requirements in AISC *Seismic Provisions* Section K1 for special and intermediate moment frames.

5.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A wide range of beam sizes has been tested with the radius-cut RBS. The maximum beam depth, maximum beam weight, and maximum beam flange thickness permitted for RBS connections has been increased compared to earlier editions of this standard, based on recent testing (Paquette et al., 2021).

Beam depth and beam span-to-depth ratio are significant in the inelastic behavior of beam-to-column connections. For the same induced curvature, deep beams will experience greater strains than shallower beams. Similarly, beams with a shorter span-to-depth ratio will have a sharper moment gradient across the beam span, resulting in reduced length of the beam participating in plastic hinging and increased strains under inelastic rotation demands. Most of the beam-to-column assemblies that have been tested had configurations approximating beam spans of about 25 ft (7.6 m) and beam depths varying from W30 (W760) to W36 (W920) so that beam span-to-depth ratios were typically in the range of 8 to 10 (FEMA, 2000e). Given the degree to which most specimens significantly exceeded the minimum interstory drift demands, it was judged reasonable to set the minimum span-to-depth ratio at 7 for SMF and 5 for IMF.

Local buckling requirements for members subjected to significant inelastic rotation are covered in the AISC *Seismic Provisions*. For the purposes of calculating the width-to-thickness ratio, it is permitted to take the flange width at the two-thirds point of the RBS cut. This provision recognizes that the plastic hinge of the beam forms within the length of the RBS cut, where the width of the flange is less than at the uncut section. This provision will result in a lower width-to-thickness ratio when taken at the RBS cut compared to that at the uncut section. Many of the RBS tests conducted as a part of the FEMA/SAC program used a W30×99 (W760×147) beam that does not quite satisfy the flange width-to-thickness ratio at the uncut section. Nevertheless, the tests were successful. For these reasons, it was judged reasonable to permit the calculation of the width-to-thickness ratio a reasonable distance into the RBS cut.

In developing this prequalification, the CPRP also reviewed lateral bracing requirements for beams with RBS connections. Some concerns were raised in the past that the presence of the RBS flange cuts might make the beam more prone to lateral-torsional buckling and that supplemental lateral bracing should be provided at the RBS. The issue of lateral bracing requirements for beams with RBS connections was subsequently investigated in both experimental and analytical studies (FEMA, 2000f; Yu et al., 2000). These studies indicated that for bare steel specimens (no composite

slab), interstory drift angles of 0.04 rad can be achieved without a supplemental lateral brace at the RBS, as long as the normal lateral bracing required for beams in SMF systems is provided in accordance with AISC *Seismic Provisions* Section D1.2b.

Studies also indicated that although supplemental bracing is not required at the RBS to achieve 0.04 rad interstory drift angles, the addition of a supplemental brace can result in improved performance. Tests on RBS specimens with composite slabs indicated that the presence of the slab provided a sufficient stabilizing effect that a supplemental brace at the RBS is not likely to provide significantly improved performance (FEMA, 2000f; Engelhardt, 1999; Tremblay et al., 1997). Based on the available data, beams with RBS connections that support a concrete structural slab are not required to have a supplemental brace at the RBS.

In cases where a supplemental brace is provided, the brace should not be connected within the reduced section (protected zone). Welded or bolted brace attachments in this highly strained region of the beam may serve as fracture initiation sites. Consequently, if a supplemental brace is provided, it should be located at or just beyond the end of the RBS that is farthest from the face of the column.

The protected zone is defined as shown in Figure 5.1 and extends from the face of the column to the end of the RBS farthest from the column. This definition is based on test observations that indicate yielding typically does not extend past the far end of the RBS cut.

6. Column Limitations

Nearly all tests of RBS connections have been performed with the beam flange welded to the column flange (i.e., strong-axis connections). The limited amount of weak-axis testing has shown acceptable performance. In the absence of more tests, the CPRP recommended limiting prequalification to strong-axis connections only.

The majority of RBS specimens were constructed with W14 (W360) columns. However, a number of tests have also been conducted using deeper columns, including W18, W27, and W36 (W460, W690, and W920) columns. Testing of deep-column RBS specimens under the FEMA/SAC program indicated that stability problems may occur when RBS connections are used with deep columns (FEMA, 2000f). In FEMA 350 (FEMA, 2000b), RBS connections were only prequalified for W12 (W310) and W14 (W360) columns.

The specimens in the FEMA/SAC tests conducted showed a considerable amount of column twisting (Gilton et al., 2000). However, two of the three specimens tested achieved 0.04 rad rotation, albeit with considerable strength degradation. The third specimen just fell short of 0.04 rad rotation and failed by fracture of the column web near the k -area. Subsequent study attributed this fracture to column twisting.

Subsequent to the FEMA/SAC tests, an analytical study (Shen et al., 2002) concluded that boundary conditions used in these tests may not be representative of what would be found in an actual building. Consequently, the large-column twisting (and, presumably, resultant k -area column fracture) seen in the FEMA/SAC tests would not

be expected in real buildings. The study also concluded that deep columns should not behave substantially different from W14 (W360) columns and that no special bracing is needed when a slab is present. This was followed by a more extensive analytical and large-scale experimental investigation on RBS connections with columns up to W36 (W920) in depth (Ricles et al., 2004). This investigation showed that good performance can be achieved with deep columns when a composite slab is present or when adequate lateral bracing is provided for the beam and/or column in the absence of a slab. More recent testing showed good performance using a W40 (W1000) column (Paquette et al., 2021). Consequently, RBS connections are prequalified for column depths up to W40 (W1000).

The behavior of RBS connections with cruciform columns is expected to be similar to that of a rolled wide-flange column because the beam flange frames into the column flange, the principal panel zone is oriented parallel to that of the beam, and the web of the cut wide-flange column is to be welded with a CJP groove weld to the continuous web 1 ft above and below the depth of the frame girder. Given these similarities and the lack of evidence suggesting behavioral limit states different from those associated with rolled wide-flange shapes, cruciform column depths are limited to those imposed on wide-flange shapes.

Successful tests have also been conducted on RBS connections with built-up box columns. The largest box column for which test data was available was 24 in. \times 24 in. (600 mm \times 600 mm). Consequently, RBS connections have been prequalified for use with built-up box columns up to 24 in. (600 mm). Limits on the width-to-thickness ratios for the walls of built-up box columns are specified in Section 2.3.2b(3) and were chosen to reasonably match the box columns that have been tested.

The use of box columns participating in orthogonal moment frames—that is, with RBS connections provided on orthogonal beams—is also prequalified. Although no data were available for test specimens with orthogonal beams, this condition should provide ostensibly the same performance as single-plane connections because the RBS does not rely on panel zone yielding for good performance and the column is expected to remain essentially elastic for the case of orthogonal connections.

Based on successful tests on wide-flange columns and on built-up box columns, boxed wide-flange columns would also be expected to provide acceptable performance. Consequently, RBS connections are prequalified for use with boxed wide-flange columns. When moment connections are made only to the flanges of the wide-flange portion of the boxed wide-flange, the column may be up to W40 (W1000) in depth. When the boxed wide-flange column participates in orthogonal moment frames, then neither the depth nor the width of the column is allowed to exceed 24 in. (600 mm), applying the same limits as for built-up boxes.

5.4. BEAM FLANGE-TO-COLUMN FLANGE WELD LIMITATIONS

Complete-joint-penetration (CJP) groove welds joining the beam flanges to the column flanges provided on the majority of RBS test specimens have been made by the self-shielded flux cored arc welding process (FCAW-S) using electrodes with

a minimum specified Charpy V-notch toughness. Three different electrode designations have commonly been used in these tests: E71T-8, E70TG-K2, and E70T-6. Further, for most specimens, the bottom flange backing was removed and a reinforcing fillet added, top flange backing was fillet welded to the column, and weld tabs were removed at both the top and bottom flanges.

Test specimens have employed a range of weld access-hole geometries, and results suggest that connection performance is not highly sensitive to the weld access-hole geometry. Consequently, prequalified RBS connections do not require a specific access-hole geometry such as that specified in AWS D1.8/D1.8M, clause 6.11.2 (AWS, 2021). Weld access holes should satisfy the requirements of AISC *Specification* Section J1.6. The commentary of Section J1.6 shows three alternatives for weld access hole geometry. Alternatives 1 or 2 are preferred for beams with RBS connections, whether the beam is a rolled shape or a built-up shape. Alternative 3 is for built-up shapes assembled after cutting the weld access hole. However, Alternative 3 is not intended for use with built-up shapes where the web is connected to the flanges using CJP groove welds, as required when built-up shapes are used for beams with RBS connections. Consequently, the Alternative 3 weld access hole geometry is not appropriate for RBS connections with built-up beams. Further guidance on weld access hole geometry is provided in AWS D1.1/D1.1M, clause 7.16 (AWS, 2020).

5.5. BEAM WEB-TO-COLUMN FLANGE CONNECTION LIMITATIONS

Two types of web connection details have been used for radius-cut RBS test specimens: a welded and a bolted detail. In the welded detail, the beam web is welded directly to the column flange using a CJP groove weld. For the bolted detail, pretensioned high-strength bolts are used. Specimens with both types of web connections have achieved at least 0.04 rad interstory drift angles, and consequently, both types of web connection details were permitted for RBS connections in FEMA 350 (2000b).

Previous test data (Engelhardt et al., 2000) indicate that beyond an interstory drift angle of 0.04 rad, specimens with bolted web connections show a higher incidence of fracture occurring near the beam-flange groove welds, as compared to specimens with welded web connections. Thus, while satisfactory performance is possible with a bolted web connection, previous test data indicate that a welded web is beneficial in reducing the vulnerability of RBS connections to fracture at the beam-flange groove welds.

Subsequent to the SAC/FEMA testing on RBS connections, a test program (Lee et al., 2004) was conducted that directly compared nominally identical RBS connections except for the web connection detail. The RBS specimens with welded web connections achieved a 0.04 rad interstory drift angle, whereas RBS specimens with bolted web connections failed to achieve 0.04 rad.

Thus, while past successful tests have been conducted on RBS connections with bolted web connections, other data have provided contradictory evidence, suggesting bolted web connections may not be suitable for RBS connections when used for SMF applications. Until further data are available, a welded web connection is required

for RBS connections prequalified for use in SMF. For IMF applications, bolted web connections are acceptable.

The beam web-to-column CJP groove weld is intended to extend the full distance between the weld access holes to minimize the potential for crack-initiation at the ends of the welds. Hence, the requirement for the plate to extend from one weld access hole to the other. All specimens were tested with the full-depth weld configuration.

5.6. FABRICATION OF FLANGE CUTS

Various shapes of flange cutouts are possible for RBS connections, including a constant cut, a tapered cut, and a radius cut. Experimental work has included successful tests on all of these types of RBS cuts. The radius cut avoids abrupt changes of cross section, reducing the chances of a premature fracture occurring within the reduced section. Further, the majority of tests reported in the literature used radius-cut RBS sections. Consequently, only the radius-cut RBS shape is prequalified.

An issue in the fabrication of RBS connections is the required surface finish and smoothness of the RBS flange cuts. No research data was found that specifically addressed this issue. Consequently, finish requirements for RBS cuts were chosen by the CPRP based on judgment and are consistent with those specified in FEMA 350 (2000b).

5.7. DESIGN PROCEDURE

Dimensions of the RBS cuts for the test specimens reported in the literature vary over a fairly small range. The distance from the face of the column to the start of the RBS radius cut (designated as a in Figure 5.1) ranged from 50 to 75% of the beam-flange width. The length of the cuts (designated as b in Figure 5.1) varied from approximately 75 to 85% of the beam depth. The amount of flange width removed at the minimum section of the RBS varied from about 38 to 55%. Flange removal for prequalified RBS connections is limited to a maximum of 50% to avoid excessive loss of strength or stiffness.

The design procedure presented herein for prequalified RBS connections is similar to that presented in FEMA 350 (2000b). The overall basis for sizing the RBS radius cut in this design procedure is to limit the maximum beam moment that can develop at the face of the column to the actual plastic moment (based on expected yield stress) of the beam when the minimum section of the RBS is fully yielded and strain hardened. Test data indicate that connecting the beam at the face of the column in accordance with the requirements herein allows the connection to resist this level of moment while minimizing the chance of fracture at the beam-flange groove welds.

Nearly all moment frame connection tests have been performed on single- or double-sided beam-column subassemblies with the beam perpendicular to the vertical axis of the column (i.e., a level beam). Nevertheless, sloping beams occur in most structures, such as at the roof. This standard does not contain provisions that explicitly address sloped beams because of the lack of systematic physical testing. Professional

judgment, therefore, is required to determine whether a proposed frame beam slope is appropriately covered by the prequalification limits in this standard.

Shallow slopes are not thought to significantly affect connection performance but identifying a threshold above which this is no longer true has not been determined. Testing of frame beams using reduced beam sections sloping at 28° (Ball et al., 2010) indicates that at this angle, the performance of the connection is adversely affected unless adjustments are made in the geometry of the reduced beam section cut. When the “ a ” dimension was equal at the top and bottom flanges (i.e., the centerline of the RBS is parallel to the vertical axis of the column), physical testing produced fracture of the beam flange CJP weld at the toe location in both specimens tested at lower than anticipated drift demands. Finite element analyses by Kim et al. (2010) showed that connections with the centerline of the RBS connection perpendicular to the beam flanges (i.e., using unequal a dimensions) reduces the demand on the beam flange weld at the toe location; see Figure C-5.1.

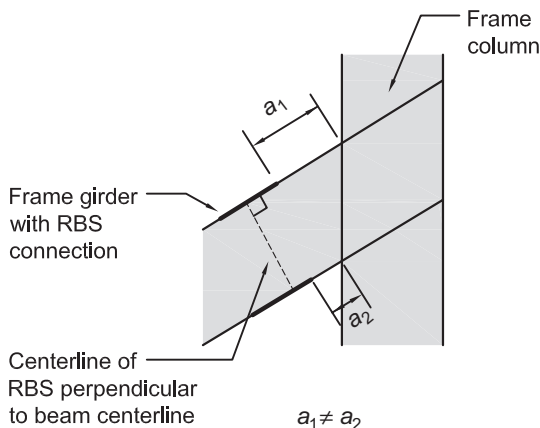


Fig. C-5.1. Analytical studies (Kim et al., 2010) suggest that a sloped frame beam with RBS centerline perpendicular to beam flanges perform better than if RBS centerline is parallel to column centerline.

CHAPTER 6

BOLTED UNSTIFFENED (BUEEP) AND STIFFENED EXTENDED END-PLATE (BSEEP) MOMENT CONNECTIONS

6.1. GENERAL

The three extended end-plate moment configurations currently addressed in this chapter are the most commonly used end-plate connection configurations in steel moment frames. AISC Design Guide 4, *Extended End-Plate Moment Connections, Seismic and Wind Applications* (Murray and Sumner, 2003) provides background, design procedures, and complete design examples for the three configurations. The guide was developed before this standard was written, and there are small differences between the design procedures in the guide and in Section 6.7. The primary differences are in the resistance factors. The standard supersedes the design guide in all instances.

Prequalification test results for the three extended end-plate moment connections are found in FEMA (1997); Meng (1996); Meng and Murray (1997); Ryan and Murray (1999); Sumner et al. (2000a, 2000b); and Sumner and Murray (2001, 2002). Results of similar testing but not used for prequalification are found in Adey et al. (1997, 1998, 2000); Castellani et al. (1998); Coons (1999); Ghobarah et al. (1990, 1992); Johnstone and Walpole (1981); Korol et al. (1990); Popov and Tsai (1989); and Tsai and Popov (1990).

The intent of the design procedure in Section 6.7 is to provide an end-plate moment connection with sufficient strength to develop the strength of the connected flexural member. The connection does not provide any contribution to inelastic rotation. All inelastic deformation for an end-plate connection is achieved by beam yielding and/or column panel zone deformation.

The design procedure in Section 6.7 is based on Borgsmiller and Murray (1995) and is similar to the “thick plate” procedure in AISC Design Guide 16, *Flush and Extended Multiple-Row Moment End-Plate Connections* (Murray and Shoemaker, 2002). The procedure is basically the same as that in FEMA 350 (2000b), but with much clarification. Applicable provisions in FEMA 353 (2000d) are incorporated into the procedure as well.

6.2. SYSTEMS

The three extended end-plate moment connections in Figure 6.1 are prequalified for use in IMF and SMF systems, except in SMF systems where the column is in direct contact with concrete structural slabs. The exception applies only when steel headed stud anchors are used to attach the concrete slab to the connected beam and is because

of the lack of test data. Prequalification testing has generally been performed with bare steel specimens. Sumner and Murray (2002) performed one test in which a slab was present. In this test, headed stud anchors were installed from near the end-plate moment connection to the end of the beam, and the concrete was in contact with the column flanges and web. The lower bolts failed prematurely by tension rupture because of the increase in the distance from the neutral axis due to the presence of the composite slab. In later testing, Murray repeated this test but placed a flexible material between the vertical face of the end plate and the slab to inhibit slab participation in transfer of load to the column. This specimen performed acceptably and resulted in provisions for using concrete structural slabs when such flexible material is placed between the slab and the end plate.

6.3. PREQUALIFICATION LIMITS

The parametric limitations in Table 6.1 were determined from reported test data in the prequalification references. Only connections that are within these limits are prequalified.

For tapered members, the depth of the beam at the connection is used to determine the limiting span-to-depth ratio.

1. Beam Limitations

The beam size limitations in Table 6.1 are directly related to connection testing. Because many of the tested beam sections were built-up members, the limitations are in cross-section dimensions instead of rolled-beam designations. There is no evidence that modest deviations from these dimensions will result in significantly different performance.

Similar to RBS testing, most of the tested beam-column assemblies had configurations approximating beam span-to-depth ratios in the range of 8 to 10. However, it was judged reasonable to set the minimum span-to-depth ratio at 7 for SMF and 5 for IMF.

The protected zone requirements are based on test observations.

2. Column Limitations

Extended end-plate moment connections may be used only with rolled or built-up I-shaped sections or flanged cruciform sections and must be flange connected. There are no other specific column requirements for extended end-plate moment connections.

6.4. CONTINUITY PLATES

The design procedure in Section 6.7 contains provisions specific to extended end-plate moment connections, and the procedure is discussed generally in AISC Design Guide 13, *Wide-Flange Column Stiffening at Moment Connections* (Carter, 1999).

6.5. BOLTS

Prequalification tests have been conducted with both pretensioned ASTM F3125/F3125M Grades A325, A325M, A490, and A490M bolts. Bolt length should be such that at least two complete threads are between the unthreaded portion of the shank and the face of the nut after the bolt is pretensioned. Faying surface requirements in AISC *Seismic Provisions* Section D2.2(d) are not required for end-plate moment connections.

6.6. CONNECTION DETAILING

Maximum gage—that is, the horizontal distance between outer bolt columns—is limited to the width of the beam flange to ensure a stiff load path. Monotonic tests have shown that the stiffness and strength of an end-plate moment connection are decreased when the bolt gage is wider than the beam flange.

Inner bolt pitch, the distance between the face of the beam flange and the first row of inside or outside bolts, must be sufficient to allow bolt tightening. The minimum pitch values specified have been found to be satisfactory. An increase in pitch distance can significantly increase the required end-plate thickness.

The end plate can be wider than the beam flange, but the width used in design calculations is limited to the beam flange width plus 1 in. (25 mm). This limitation is based on the CPRP's assessment of unpublished results of monotonic tests of end-plate connections.

The requirements for the length of beam flange-to-end-plate stiffeners are established to ensure a smooth load path. The 30° angle is the same as used for determining the Whitmore section width in other types of connections. The required 1 in. (25 mm) land is needed to ensure the quality of the vertical and horizontal weld terminations.

Tests have shown that the use of finger shims between the end plate and the column flange do not affect the performance of the connection (Sumner et al., 2000a).

Design procedures are not available for connections of beams with composite action at an extended end-plate moment connection. Therefore, careful composite slab detailing is necessary to prevent composite action that may increase tension forces in the lower bolts. Welded steel headed stud anchors are not permitted within 1½ times the beam depth, and compressible material is required between the concrete slab and the column face (Sumner and Murray, 2002; Yang et al., 2003).

Cyclic testing has shown that use of weld access holes can cause premature fracture of the beam flange at extended end-plate moment connections (Meng and Murray, 1997). Short to long weld access holes were investigated with similar results. Therefore, weld access holes are not permitted for extended end-plate moment connections.

Strain gage measurements have shown that the web plate material in the vicinity of the inside tension bolts generally reaches the yield strain (Murray and Kukreti, 1988). Consequently, it is required that the web-to-end-plate weld(s) in the vicinity of the inside bolts be sufficient to develop the strength of the beam web.

The beam flange-to-end plate and stiffener weld requirements equal or exceed the welding that was used to prequalify the three extended end-plate moment connections. Because weld access holes are not permitted, the beam flange-to-end-plate weld at the beam web is necessarily a PJP groove weld. The prequalification testing has shown that these conditions are not detrimental to the performance of the connection.

6.7. DESIGN PROCEDURE

The design procedure in this section, with some modification, was used to design the prequalification test specimens. The procedure is very similar to that in AISC Design Guide 4 (Murray and Sumner, 2003), except that different resistance factors are used. Example calculations are found in the design guide. Column stiffening example calculations are found in AISC Design Guide 13 (Carter, 1999).

CHAPTER 7

BOLTED FLANGE PLATE (BFP) MOMENT CONNECTION

7.1. GENERAL

The bolted flange plate (BFP) connection is a field-bolted connection. The fundamental seismic behaviors expected with the BFP moment connection include:

- (1) Initial yielding of the beam at the last bolt away from the face of the column.
- (2) Slip of the flange plate bolts, which occurs at similar resistance levels to the initial yielding in the beam flange, but the slip does not contribute greatly to the total deformation capacity of the connection.
- (3) Secondary yielding in the column panel zone, which occurs as the expected moment capacity and strain hardening occur.
- (4) Limited yielding of the flange plate, which may occur at the maximum deformations.

This sequence of yielding has resulted in very large inelastic deformation capacity for the BFP moment connection, but the design procedure is somewhat more complex than some other prequalified connections.

The flange plates and web shear plate are shop-welded to the column flange and field-bolted to the beam flanges and web, respectively. ASTM F3125/F3125M Grade A490 or A490M bolts with threads excluded from the shear plane are used for the beam flange connections because the higher shear strength of the Grade A490 or A490M bolts reduces the number of bolts required and reduces the length of the flange plate. The shorter flange plates that are, therefore, possible reduce the seismic inelastic deformation demands on the connection and simplify the balance of the resistances required for different failure modes in the design procedure. Flange plate connections with ASTM F3125/F3125M Grade A325 or A325M bolts may be possible but will be more difficult to accomplish because of the reduced bolt strength, greater number of bolts, and longer flange plates required. As a result, the connection is not prequalified for use with Grade A325 or A325M bolts.

Prequalification of the BFP moment connection is based upon 20 BFP moment connection tests under cyclic inelastic deformation (FEMA, 2000e; Schneider and Teeraparabwong, 1999; Sato et al., 2008). Additional evidence supporting prequalification is derived from bolted T-stub connection tests (FEMA, 2000e; Swanson et al., 2000) because the BFP moment connection shares many yield mechanisms, failure modes, and connection behaviors with the bolted T-stub connection. The tests were

performed under several deformation-controlled test protocols, but most use variations of the ATC-24 (ATC, 1992) or the SAC steel protocol (Krawinkler et al., 2000), which are both very similar to the prequalification test protocol of AISC *Seismic Provisions* Chapter K. The 20 BFP tests were performed on connections with beams ranging in depth from W8 (W200) to W36 (W920) sections, and the average total demonstrated ductility capacity exceeded 0.057 rad.

Most past tests have been conducted on specimens with single-sided connections, and the force-deflection behavior is somewhat pinched as shown in Figure C-7.1. Because plastic hinging at the end of the flange plate is the controlling yield mechanism, the expected plastic moment at this location dominates the connection design. The pinching is caused by a combination of bolt slip and the sequence of yielding and strain hardening encountered in the connection. Experiments have shown that the expected peak moment capacity at the plastic hinge is typically on the order of 1.15 times the expected M_p of the beam, as defined in the AISC *Seismic Provisions*, and the expected moment at the face of the column is on the order of 1.3 to 1.5 times the expected M_p of the beam, depending upon the span length, number of bolts, and length of the flange plate. The stiffness of this connection is usually slightly greater than 90% of that anticipated with a truly rigid, fully restrained (FR) connection. This reduced stiffness is expected to result in elastic deflection no more than 10% larger than computed with an FR connection, and thus elastic calculations with rigid connections are considered to be adequate for most practical design purposes.

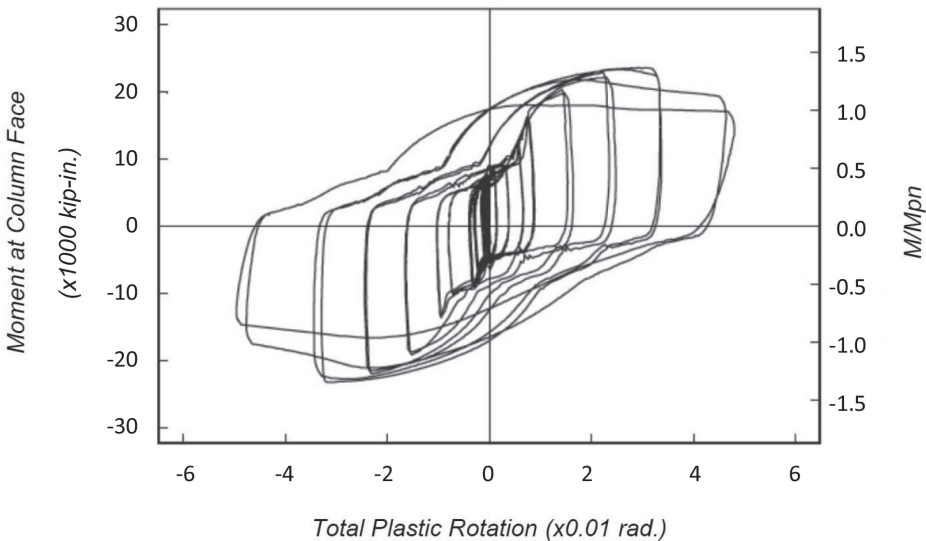


Fig. C-7.1. Moment at face of column versus total connection rotation for a BFP moment connection with a W30×108 (W760×161) beam and a W14×233 (W360×347) column.

7.2. SYSTEMS

Review of the research literature shows that BFP moment connections meet the qualifications and requirements of both SMF and IMF frames. However, no test data are available for BFP moment connections with composite slabs, and thus the BFP moment connection is not prequalified with reinforced concrete structural slabs that contact the face of the columns. Therefore, prequalification of the BFP moment connection is restricted to the case where the concrete structural slab has a minimum separation or isolation from the column as required in Section 2.3.4.

7.3. PREQUALIFICATION LIMITS

1. Beam Limitations

The SMF prequalification limits largely reflect the range of past testing of the BFP moment connection. Limits for IMF connections somewhat exceed these limits because 18 of the past 20 tests used to prequalify the connection developed plastic rotations larger than those required to qualify as an SMF connection, and all 20 tests greatly exceed the rotation required to qualify as an IMF connection.

BFP moment connections have been tested with beams as large as the W36×150 (W920×223) while achieving the ductility required for qualification as an SMF. Consequently, the W36 (W920) beam depth, 150 lb/ft weight limit (220 kg/m mass limit), and 1 in. (25 mm) flange thickness limits are adopted in this provision. Past tests have shown adequate inelastic rotation capacity to qualify as an SMF in tests with span-to-depth ratios less than 5 and greater than 16, so lower bound span-to-depth ratio limits of 7 and 9 are conservatively adopted for the IMF and SMF applications, respectively. Inelastic deformation is expected for approximately one beam depth beyond the end of the flange plate, and limited yielding is expected in the flange plate. As a result, the protected zone extends from the column face to a distance equal to the depth of the beam beyond the bolt farthest from the face of the column.

Primary plastic hinging of the BFP moment connection occurs well away from the face of the column, and lateral-torsional deformation will occur as extensive yielding develops in the connection. As a result, lateral bracing of the beam is required at the end of the protected zone. The bracing is required within the interval between 1 and 1.5 beam depths beyond the flange bolts farthest from the face of the column. This permits some variation in the placement of the lateral support to allow economical use of transverse framing for lateral support where possible. As with other moment frame connections, supplemental lateral bracing at the column flange connection can typically be accommodated by the stiffness of the diaphragm and transverse framing.

2. Column Limitations

BFP moment connections have been tested with wide-flange columns up to W14×233 (W360×347) sections. The SMF prequalification limits largely reflect the range of past testing of the BFP moment connection. All 20 tests were completed with strong-axis bending of the column, and the prequalification of the BFP moment connections is limited to connections made to the column flange.

As with most other prequalified connections, the BFP moment connection has not been tested with columns deeper than W14 (W360) sections or with built-up column sections. It was the judgment of the CPRP that the BFP moment connection places similar or perhaps smaller demands on the column than other prequalified connections. The demands may be smaller because of the somewhat smaller strain-hardening moment increase achieved with the BFP moment connection as compared to the welded web-welded flange and other FR connections. The location of yielding of the BFP moment connection is somewhat analogous to the RBS connection, and therefore, prequalification limits for the column are comparable to those used for the RBS connection.

7.4. CONNECTION DETAILING

The BFP moment connection requires plate steel for the flange plate, shear plate, and possibly panel zone doubler plates. Past tests have been performed with plates fabricated both from ASTM A36/A36M and A572/A572M Grade 50 (Grade 345) steels. Therefore, the prequalification extends to both plate types. The designer should be aware of potential pitfalls with the material selection for the flange plate design. The flange plate must develop tensile yield strength over the gross section and ultimate tensile fracture resistance over the effective net section. A36/A36M steel has greater separation of the nominal yield stress and the minimum tensile strength, and this may simplify the satisfaction of these dual requirements. However, variation in expected yield stress is larger for A36/A36M steel, and design calculations may more accurately approximate actual flange plate performance with A572/A572M steel.

The flange plate welds are shop welds, and these welds are subject to potential secondary yielding caused by strain hardening at the primary yield location in the beam. As a result, the welds are required to be demand-critical CJP groove welds. If backing is used, it must be removed, and the weld must be backgouged to sound material and backwelded to ensure that the weld can sustain yielding of the flange plate. Because the welds are shop welds, considerable latitude is possible in the selection of the weld process as long as the finished weld meets the demand-critical weld requirements stipulated in the AISC *Seismic Provisions*. In the test specimens used to prequalify this connection, electroslag, gas shielded metal arc, and flux cored arc welding have been used.

The BFP moment connection places somewhat less severe demands on the web connection than most FR connections because of the somewhat greater flexibility of the bolted flange connection. As a result, the shear plate may be welded with CJP groove welds, PJP groove welds, or fillet welds.

Bolts in the flange plate are limited to two rows of bolts, and the bolt holes must be made by drilling or subpunching and reaming. These requirements reflect testing used to prequalify the BFP moment connection, but they also reflect practical limitations in the connection design. Net section rupture is a clear possibility in the beam flange and flange plates, and it is very difficult to meet the net section rupture criteria if more than two rows of bolts are employed.

A single row of bolts causes severe eccentricity in the connection and would lead to an excessively long connection. Punched bolt holes without reaming and thermally cut bolt holes are not permitted because the surface roughness in the hole may initiate cracking of the net section under high tensile stress. As noted earlier, the connection is prequalified only for A490 or A490M bolts with threads excluded from the shear plane. Bolt diameter is limited to a maximum of 1½ in. (28 mm) because larger bolts are seldom used and the 1½ in. (28 mm) diameter is the maximum used in past BFP tests. The bolt diameter must be selected to ensure that flange yielding over the gross area exceeds the net section capacity of the beam flange.

Oversized bolt holes in the flange plates were included in some past tests because the oversized holes permit easier alignment of the bolts and erection of the connection and resulted in good performance of the connection. Consequently, oversize holes are permitted in the flange plates. When oversize holes are used, the bolted joint may be designed as for bearing type joints and need not be designed as slip critical. However, as required in the AISC *Seismic Provisions*, all bolts should be installed as pretensioned high strength bolts, and faying surfaces should be prepared to provide a Class A slip coefficient or higher.

The beam must fit between two welded flange plates with full consideration of rolling and fabrication tolerances. As a result, shims may be used to simplify erection while ensuring a tight connection fit.

7.5. DESIGN PROCEDURE

Step 1 of this procedure defines the maximum expected moment, M_{pr} , at the plastic hinge location, which is at the last bolt away from the face of the column in the flange plate. **Step 2** establishes the maximum bolt diameter that can meet this balanced criterion. Beginning with AISC 358-22, the approach used is the same as that used in the procedure for the double-tee connection found in Chapter 13. The approach that was previously used for bolted flange plate connections was developed from AISC *Specification* Chapter F and considers only the tension flange of the beam when determining whether gross yielding or net-section fracture will control. The current approach considers the entire cross section of the beam, which is less conservative and is substantiated by a study by Swanson (2016). Additional details are available in the Commentary to Section 13.5.

The shear strength of the flange bolts is the smallest strength permitted based on bolt shear with threads excluded from the shear plane, bolt bearing on the flange plate, and bolt bearing on the beam flange. **Step 3** provides this evaluation. **Step 4** is an approximate evaluation of the number of bolts needed to develop the BFP moment connection. The moment for the bolts is larger than M_{pr} because the centroid of the bolt group is at a different location than the primary hinge location. However, this moment cannot be accurately determined until the geometry of the flange plate and bolt spacing are established. The 1.25 factor is used as an empirical increase in this moment to provide this initial estimate for the number of bolts required. The bolts are tightened to meet slip-critical criteria, but the connection is not slip critical. The bolts are designed as bearing bolts.

CHAPTER 8

WELDED UNREINFORCED FLANGE-WELDED WEB (WUF-W) MOMENT CONNECTION

8.1. GENERAL

The welded unreinforced flange-welded web (WUF-W) moment connection is an all-welded moment connection, wherein the beam flanges and the beam web are welded directly to the column flange. A number of welded moment connections that came into use after the 1994 Northridge earthquake, such as the reduced beam section and connections provided with beam flange reinforcement, were designed to move the plastic hinge away from the face of the column. In the case of the WUF-W moment connection, the plastic hinge is not moved away from the face of the column. Rather, the WUF-W moment connection employs design and detailing features that are intended to permit the connection to achieve SMF performance criteria without fracture. Key features of the WUF-W moment connection that are intended to control fracture are as follows:

- The beam flanges are welded to the column flange using CJP groove welds that meet the requirements of demand critical welds in the AISC *Seismic Provisions*, along with the requirements for treatment of backing, weld tabs, and welding quality control and quality assurance requirements, as specified in Chapter 3.
- The beam web is welded directly to the column flange using a CJP groove weld that extends the full depth of the web—that is, from weld access hole to weld access hole. This is supplemented by a single-plate connection, wherein a single plate is welded to the column flange and is then fillet welded to the beam web. Thus, the beam web is attached to the column flange with both a CJP groove weld and a welded single-plate connection. The single-plate connection adds stiffness to the beam web connection, drawing stress toward the web connection and away from the beam flange-to-column flange connections. The single plate also serves as backing for the CJP groove weld connecting the beam web to the column flange.
- Instead of using a conventional weld access hole detail as specified in AISC *Specification* Section J1.6 (AISC, 2022b), the WUF-W moment connection employs a special seismic weld access hole with requirements on size, shape, and finish that reduce stress concentrations in the region around the access hole detailed in AWS D1.8/D1.8M (AWS, 2021).

Prequalification of the WUF-W moment connection is based on the results of two major research and testing programs. Both programs combined large-scale tests with extensive finite-element studies. Both are briefly described herein.

The first research program on the WUF-W moment connection was conducted at Lehigh University as part of the SAC-FEMA program. Results are reported in several publications (Ricles et al., 2000, 2002). This test program formed the basis of prequalification of the WUF-W moment connection in FEMA 350 (FEMA, 2000e). As part of the Lehigh program, tests were conducted on both interior and exterior type specimens. The exterior specimens consisted of one beam attached to a column. The interior specimens consisted of a column with beams attached to both flanges. One of the interior specimens included a composite floor slab. All specimens used W36×150 (W920×223) beams. Three different column sizes were used: W14×311, W14×398, and W27×258 (W360×463, W360×592, and W690×384). All WUF-W moment connection specimens tested in the Lehigh program satisfied the rotation criteria for SMF connections (± 0.04 rad total rotation). Most specimens significantly exceeded the qualification criteria. Considering that the interior type specimens included two WUF-W moment connections each, 12 successful WUF-W moment connections were tested in the Lehigh program. This research program included extensive finite-element studies that supported the development of the special seismic weld access hole and the details of the web connection.

The second major research program on the WUF-W moment connection was conducted at the University of Minnesota. The purpose of this research program was to examine alternative doubler plate details, continuity plate requirements, and effects of a weak panel zone. All test specimens used the WUF-W moment connection. Results are reported in several publications (Lee et al., 2002, 2005a, 2005b). Six interior type specimens were tested in the Minnesota program. All specimens used W24×94 (W610×140) beams. Three column sizes were used: W14×283, W14×176, and W14×145 (W360×421, W360×262, and W360×216). All specimens were designed with panel zones weaker than permitted by the AISC *Seismic Provisions*. Two of the test specimens, CR1 and CR4, were inadvertently welded with low-toughness weld metal. This resulted in premature weld failure in specimen CR4 (failure occurred at about 0.015 rad rotation). With the exception of CR4, all specimens achieved a total rotation of ± 0.04 rad, and sustained multiple cycles of loading at ± 0.04 rad prior to failure. All successful specimens exhibited substantial panel zone yielding, due to the weak panel zone design. This test program was also supported by extensive finite-element studies.

Considering the WUF-W moment connection research programs at both Lehigh and the University of Minnesota, WUF-W moment connection specimens have shown excellent performance in tests. There is only one reported failed test, due to the inadvertent use of low-toughness weld metal for beam flange CJP groove welds (Minnesota specimen CR4). Of all of the WUF-W moment connection specimens that showed good performance (achieved rotations of at least ± 0.04 rad), approximately one-half had panel zones weaker than permitted by the AISC *Seismic Provisions*. The other half satisfied the panel zone strength criteria of the AISC *Seismic Provisions*. This suggests that the WUF-W moment connection performs well for both strong and weak panel zones; therefore, the connection is not highly sensitive to panel zone strength.

The protected zone for the WUF-W moment connection is defined as the portion of the beam extending from the face of the column to a distance, d , from the face of the column, where d is the depth of the beam. Tests on WUF-W moment connection specimens show that yielding in the beam is concentrated near the face of the column but extends to some degree over a length of the beam approximately equal to its depth.

8.3. PREQUALIFICATION LIMITS

The WUF-W moment connection is prequalified for beams up to W36 (W920) in depth, up to 150 lb/ft in weight (220 kg/m mass limit), and up to a beam flange thickness of 1 in. (25 mm). This is based on the fact that a W36×150 (W920×223) is the deepest and heaviest beam tested with the WUF-W moment connection. The 1 in. (25 mm) flange thickness limitation represents a small extrapolation of the 0.94 in. (24 mm) flange thickness for the W36×150 (W920×223). Limits are also placed on span-to-depth ratio based on the span-to-depth ratios of the tested connections and based on judgment of the CPRP.

Beam lateral bracing requirements for the WUF-W moment connection are identical to those for the RBS moment connection. The effects of beam lateral bracing on cyclic loading performance have been investigated more extensively for the RBS moment connection than for the WUF-W moment connection. However, the available data for the WUF-W moment connection suggest that beams are less prone to lateral-torsional buckling than with the RBS moment connections. Consequently, it is believed that lateral bracing requirements established for the RBS moment connection are satisfactory, and perhaps somewhat conservative, for the WUF-W moment connection.

Column sections used in WUF-W moment connection test specimens were W14 (W360) and W27 (W690) sections. However, column limitations for the WUF-W moment connection are nearly the same as for the RBS moment connection, which includes wide-flange shapes up to W36 (W920) and box columns up to 24 in. by 24 in. (600 mm by 600 mm). A primary concern with deep columns in moment frames has been the potential for twisting and instability of the column driven by lateral-torsional buckling of the beam. Because beams with WUF-W moment connections are viewed as somewhat less prone to lateral-torsional buckling than beams with RBS moment connections, the column limitations established for the RBS moment connection were judged as appropriate for the WUF-W moment connection.

8.4. BEAM FLANGE-TO-COLUMN FLANGE WELDS

The welds must meet the requirements of demand critical welds in the AISC *Seismic Provisions*, as well as the detailing and quality control and quality assurance requirements specified in Chapter 3. These beam flange-to-column flange weld requirements reflect the practices used in the test specimens that form the basis for prequalification of the WUF-W moment connection and reflect what are believed to be best practices for beam flange groove welds for SMF and IMF applications.

A key feature of the WUF-W moment connection is the use of a special weld access hole. The special seismic weld access hole has specific requirements on the size, shape, and finish. This special access hole was developed in research on the WUF-W moment connection (Ricles et al., 2000, 2002) and is intended to reduce stress concentrations introduced by the presence of the weld access hole. The size, shape, and finish requirements for the special access hole are specified in AWS D1.8/D1.8M, clause 6.11.1.2 (AWS, 2021).

8.5. BEAM WEB-TO-COLUMN CONNECTION LIMITATIONS

The beam web is connected to the column flange with a full-depth (weld access hole-to-weld access hole) CJP groove weld, with a single plate serving as backing. The single plate is fillet welded to the beam web and also welded to the column flange. See Figure 8.2 for detail. The use of the CJP groove weld combined with the fillet-welded single plate is believed to increase the stiffness of the beam web connection. The stiffer beam web connection serves to draw stress away from the beam flanges and therefore reduces the demands on the beam flange groove welds.

Most of the details of the beam web-to-column connection are fully prescribed in Section 8.5; thus, few design calculations are needed for this connection. An exception to this is the connection of the single plate to the column. This connection must develop the shear strength of the single plate, as specified in Section 8.5(2). This can be accomplished by the use of CJP groove welds, PJP groove welds, fillets welds, or combinations of these welds. The choice of these welds is left to the discretion of the designer. In developing the connection between the single plate and the column flange, designers should consider the following issues:

- The use of a single-sided fillet weld between the single plate and the column flange should be avoided. If the single plate is inadvertently loaded or struck in the out-of-plane direction during erection, the fillet weld may break and may lead to erection safety concerns.
- The end of the beam web must be set back from the face of the column flange a specified amount to accommodate the web CJP root opening dimensional requirements. Consequently, the single plate-to-column weld that is placed in the web CJP root opening must be small enough to fit in that specified root opening. For example, if the CJP groove weld is detailed with a $\frac{1}{4}$ in. (6 mm) root opening, a fillet weld between the single plate and the column flange larger than $\frac{1}{4}$ in. (6 mm) will cause the root of the CJP groove weld to exceed $\frac{1}{4}$ in. (6 mm).
- Placement of the CJP groove weld connecting the beam web to the column flange will likely result in intermixing of weld metal, with the weld attaching the single plate to the column flange. Requirements for intermix of filler metals specified in AWS D1.8/D1.8M (AWS, 2021) should be followed in this case.

The CJP groove weld connecting the beam web to the column flange must meet the requirements of demand critical welds. Note that weld tabs are permitted, but not required, at the top and bottom ends of this weld. If weld tabs are used, they should

be removed after welding according to the requirements of Section 3.4. If weld tabs are not used, the CJP groove weld should be terminated in a manner that minimizes notches and stress concentrations, such as with the use of cascaded ends.

The fillet weld connecting the beam web to the single plate should be terminated a small distance from the weld access hole, as shown in Figure 8.3. This is to avoid introducing notches at the edge of the weld access hole.

8.6. DESIGN PROCEDURE

For the WUF-W moment connection, many of the details of the connection of the beam to the column flange are fully prescribed in Sections 8.4 and 8.5. Consequently, the design procedure for the WUF-W moment connection largely involves typical checks for continuity plates, panel zone shear strength, column-beam moment ratio, and beam shear strength.

With the WUF-W moment connection, yielding of the beam (i.e., plastic hinge formation) occurs over the portion of the beam extending from the face of the column to a distance of approximately one beam depth beyond the face of the column. For purposes of the design procedure, the location of the plastic hinge is taken to be at the face of the column. That is, $S_h = 0$ for the WUF-W moment connection. It should be noted that the location of the plastic hinge for design calculation purposes is somewhat arbitrary because the plastic hinge does not occur at a single point but, instead, occurs over some length of the beam. The use of $S_h = 0$ is selected to simplify the design calculations. The value of C_{pr} was calibrated so that when used with $S_h = 0$, the calculated moment at the column face reflects values measured in experiments. Note that the moment in the beam at the column face is the key parameter in checking panel zone strength, column-beam moment ratio, and beam shear strength.

The value of C_{pr} for the WUF-W moment connection is specified as 1.4, based on an evaluation of experimental data. Tests on WUF-W moment connections with strong panel zones (Ricles et al., 2000) showed maximum beam moments, measured at the face of the column, as high as $1.49M_p$, where M_p was based on measured values of F_y . The average maximum beam moment at the face of the column was $1.33M_p$. Consequently, strain hardening in the beam with a WUF-W moment connection is quite large. The value of C_{pr} of 1.4 was chosen to reflect this high degree of strain hardening. Combining the value of $C_{pr} = 1.4$ with $S_h = 0$ results in a moment at the face of the column, $M_f = M_{pr} = 1.4R_yF_yZ$, that reasonably reflects maximum column face moments measured in experiments.

CHAPTER 9

CAST BOLTED BRACKET (CBB) MOMENT CONNECTION

9.1. GENERAL

The cast bolted bracket (CBB) moment connection is designed to eliminate field welding and facilitate frame erection. Depending on fabrication preference, the brackets can be either fillet welded (W-series) or bolted (B-series) to the beam. The B-series can also be utilized to improve the strength of weak or damaged connections, although it is not prequalified for that purpose. Information on the cast steel and the process used to manufacture the brackets is provided in Appendix A.

Connection prequalification is based on 21 full-scale bolted bracket tests representing both new and repaired applications (Kasai and Bleiman, 1996; Gross et al., 1999; Newell and Uang, 2006; and Adan and Gibb, 2009). These tests were performed using beams ranging in depth from W16 to W36 (W410 to W920) and columns using W12, W14, and W27 (W310, W360, and W690) sections. Built-up box columns have also been tested. The test subassemblies have included both single cantilever and double-sided column configurations. Concrete slabs were not present in any tests. During testing, inelastic deformation was achieved primarily through the formation of a plastic hinge in the beam. Some secondary yielding was also achieved in the column panel zone. Peak strength typically occurred at an interstory drift angle between 0.025 and 0.045 rad. Specimen strength then gradually decreased with additional yielding and deformation. In the CBB testing reported by Adan and Gibb (2009), the average specimen maximum interstory drift angle exceeded 0.055 rad.

In earlier editions of this standard, the cast bolted bracket moment connection was called the Kaiser bolted bracket moment connection. The brackets were a proprietary design protected under a U.S. patent. The patent has since expired. This connection is therefore no longer proprietary and was subsequently renamed as the cast bolted bracket moment connection.

9.2. SYSTEMS

Review of the research literature and testing referenced in this document indicates that the CBB moment connection meets the prequalification requirement for special and intermediate moment frames.

The exception associated with concrete structural slab placement at the column and bracket flanges is based on testing conducted on the stiffened extended end-plate moment connection (Seek and Murray, 2008). While bolted bracket testing has been conducted primarily on bare-steel specimens, some limited testing has also been performed on specimens with a concrete structural slab. In these tests, the presence of

the slab provided a beneficial effect by maintaining the stability of the beam at larger interstory drift angles (Gross et al., 1999; Newell and Uang, 2006). However, in the absence of more comprehensive testing with a slab, the placement of the concrete is subject to the exception.

9.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A wide range of beam sizes was tested with bolted brackets. The lightest beam size reported in the literature was a W16×40 (W410×60). The heaviest beam reported was a W36×210 (W920×313). In the W36×210 (W920×313) test, the specimen met the requirements, but subsequently experienced an unexpected nonductile failure of the bolts connecting the bracket to the column. The next heaviest beams reported to have met the requirements were W33×130 and W36×150 (W840×193 and W920×223). Based on the judgment of the CPRP, the maximum beam depth and weight was limited to match that of the W33×130 (W840×193). The maximum flange thickness was established to match a modest increase above that of the W36×150 (W920×223).

The limitation associated with minimum beam flange width is required to accommodate fillet weld attachment of the W-series bracket and to prevent beam flange tensile rupture when using the B-series bracket.

Bolted bracket connection test assemblies used configurations approximating beam spans between 24 and 30 ft (7.3 and 9.1 m). The beam span-to-depth ratios were in the range of 8 to 20. Given the degree to which most specimens significantly exceeded the requirement, it was judged reasonable to set the minimum span-to-depth ratio at 9 for both SMF and IMF systems.

As with other prequalified connections, beams supporting a concrete structural slab are not required to have a supplemental brace near the expected plastic hinge. If no floor slab is present, then a supplemental brace is required. The brace may not be located within the protected zone.

2. Column Limitations

Bolted bracket connection tests were performed with the brackets bolted to the column flange (i.e., strong-axis connections). In the absence of additional testing with brackets bolted to the column web (weak-axis connections), the prequalification is limited to column flange connections.

Test specimen wide-flange column sizes ranged from W12×65 to W27×281 (W310×97 to W690×418). Testing performed by Ricles et al. (2004) of deep-column RBS connections demonstrated that deep columns do not behave substantially different from W14 (W360) columns when a slab is present or when adequate lateral bracing is provided for the beam and/or column in the absence of a slab. Based on the similarity in performance to that of the RBS connection, the CBB is prequalified to include column sizes up to W36 (W920).

The behavior of a flanged cruciform column in CBB connections is expected to be similar to that of a rolled wide-flange shape. Therefore, flanged cruciform columns are prequalified, subject to the limitations imposed on rolled wide-flange shapes.

Two of the tests were successfully conducted using a built-up box column. In the first box column test, connections were made on two opposing column faces. Then, in the second test, a connection was made to the orthogonal face of the same column. These two tests were intended to prequalify a box column participating in orthogonal moment frames. The tested box column was 15 $\frac{5}{8}$ in. (400 mm) square (Adan and Gibb, 2009). Consequently, bolted bracket connections are prequalified for use with built-up box columns up to 16 in. (400 mm) square.

Based on both successful wide-flange and built-up box column testing, acceptable performance would also be expected for boxed wide-flange columns. Therefore, the use of boxed wide-flange columns is also prequalified. When moment connections are made only to the flanges of the wide-flange portion of the boxed wide flange, subject to the bracing limitations mentioned previously, the column may be as deep as a W36 (W920). When the boxed wide-flange column participates in orthogonal moment frames, neither the depth nor the width of the column is allowed to exceed 16 in. (400 mm), applying the same limit as a built-up box.

3. Bracket Limitations

The ASTM cast steel material specification used to manufacture the brackets is based on recommendations from the Steel Founders' Society of America (SFSA).

The cast brackets are configured and proportioned to resist applied loads in accordance with the limit states outlined by Gross et al. (1999). These limit states include column flange local buckling; bolt prying action; combined bending and axial loading on the bracket; shear; and, additionally for the B-series, bolt bearing deformation and block shear rupture.

In tests representing new applications, the bracket column bolt holes were cast vertically short-slotted. The vertically slotted holes provide field installation tolerance. In tests representing a repair application, the holes were cast standard diameter. There has been no difference in connection performance using either type of cast hole (Adan and Gibb, 2009).

9.4. BRACKET-TO-COLUMN FLANGE CONNECTION LIMITATIONS

In the prequalification tests, fasteners joining the bracket to the column flange were pretensioned ASTM F3125/F3125M Grade A490 or A490M bolts. The column bolt head can be positioned on either the column or bracket side of the connection. Where possible, the column bolts are tightened prior to the bolts in the web single-plate shear connection.

When needed, finger shims between the bracket and column face allow for fit between the bracket and column contact surfaces. Tests indicated that the use of finger shims does not affect the performance of the connection.

Because the flanges of a box column are stiffened only at the corners, tightening of the column bolts can cause excessive local flange bending. Therefore, as shown in Figure C-9.1, a washer plate is required between the box column flange and the bracket.

As shown in Figure C-9.1, orthogonally connected beams framing into a box column are raised one-half of the column bolt spacing distance to avoid overlapping the column bolts.

9.5. BRACKET-TO-BEAM FLANGE CONNECTION LIMITATIONS

The cast steel brackets are not currently listed as a prequalified material in AWS D1.1/D1.1M (AWS, 2020). Therefore, the weld procedure specification (WPS) for the fillet weld joining the bracket to the beam flange is required to be qualified by test with the specific cast material.

Bolts joining the bracket to the beam flange in prequalification tests have been conducted with pretensioned ASTM F3125/F3125M Grade A490 or A490M bolts with the threads excluded from the shear plane. The beam bolt head can be positioned on either the beam or bracket side of the connection. Given the beam bolt pattern and hole size, it is necessary to use the bracket as a template when drilling the beam bolt holes. The holes must be aligned to permit insertion of the bolts without undue damage to the threads.

The brass washer plate prevents abrading of the beam and bracket contact surfaces. In the initial developmental stages of the connection, several specimens configured without the brass plate experienced flange net section rupture through the outermost bolt holes. Observation of the failed specimens indicated that fracture likely initiated at a notch created by the abrading contact surfaces near the hole. Furthermore, energy released through the beam-bracket slip-stick mechanism caused loud, intermittent bursts of noise, particularly at high levels of inelastic drift (Kasai and Bleiman, 1996). To overcome these problems, the brass plate was inserted between the bracket and the beam flange. The idea is based on the use of a brass plate as a special friction-based seismic energy dissipator (Grigorian et al., 1992). Although not intended to dissipate energy in the bolted bracket connection, the brass plate provides a smooth slip mechanism at the bracket-to-beam interface.

When bolting the bracket to a beam flange, a steel washer or clamp plate is positioned on the opposite side of the connected flange. The restraining force of the clamp plate prevents local flange buckling from occurring near the outermost bolt holes. In tests performed without the clamp plates, flange distortion increased the strains near the holes. The increased strain caused necking and fracture through the flange net area. In similar tests performed with the clamp plates, yielding and fracture occurred outside the connected region through the flange gross area (Kasai and Bleiman, 1996).

9.6. CONNECTION DETAILING

Both Figures 9.4 and 9.5 show the connection configured with continuity plates where required. The use of continuity plates is dictated by the need to satisfy prescribed limit states for the flange and web of the column. In a bolted connection, the configuration of the fasteners can impede the ability of the continuity plates to effectively address these limit states. The design intent for the CBB is to satisfy the prescribed limit states without continuity plates. In tests of wide-flange columns without continuity plates, the absence of the continuity plates did not appear to promote local flange bending or lead to other detrimental effects (Adan and Gibb, 2009). However, in the absence of additional tests, the need for continuity plates must be checked for all connections according to Step 17 of the design procedure.

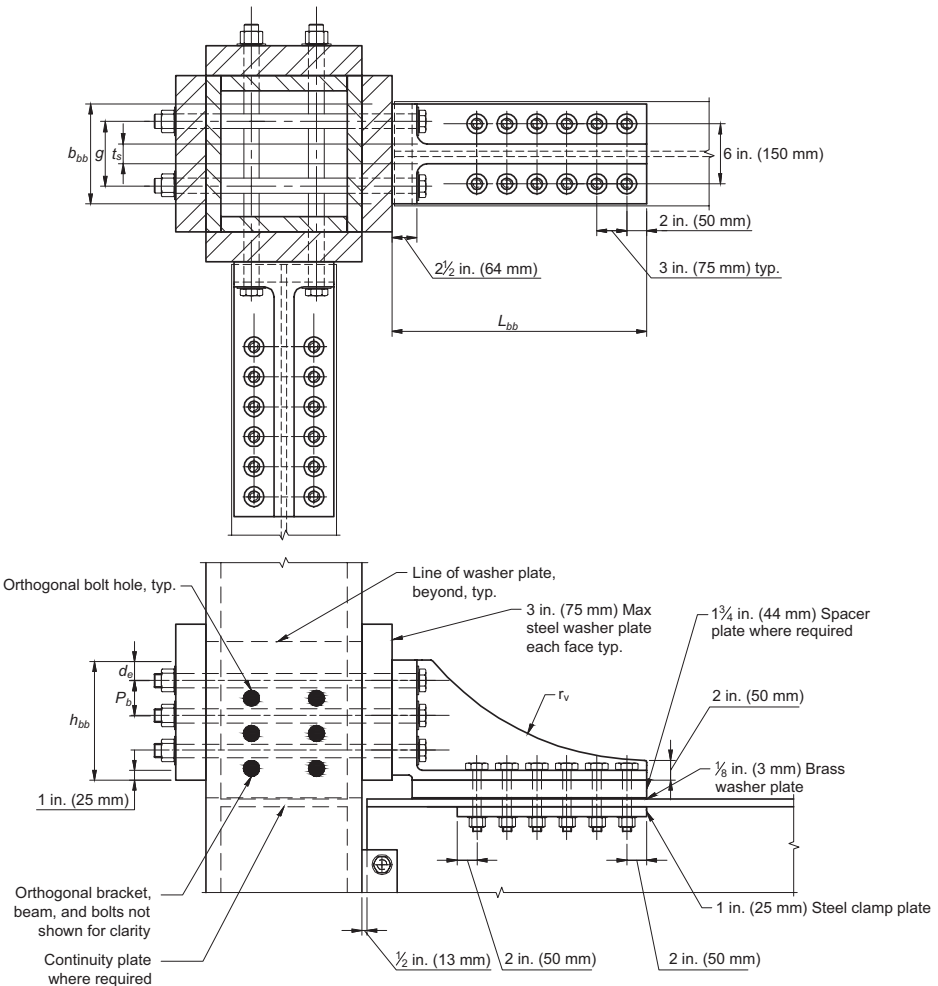


Fig. C-9.1. Box column connection detailing for CBB.

9.7. DESIGN PROCEDURE

The design procedure for prequalified CBB connections is intended to develop the probable maximum moment capacity of the connecting beam. Test data indicate that connecting the brackets to the column and beam in accordance with the requirements herein allows the connection to resist this level of moment.

Step 2 requires the selection of a trial bracket. Tables C-9.1, C-9.1M, C-9.2, and C-9.2M can be used as a guide in selecting trial bracket-beam combinations. The tables are based on beams that satisfy the limitations of Section 9.3.1 for ASTM A992/A992M or A572/A572M Grade 50 (Grade 345) wide-flange shapes.

Step 8 defines the minimum column flange width so that yield of the gross section of the column flange precedes tensile rupture. Because the column is subjected to combined bending and axial force (which may be tension or compression) and the magnitude of these force components when acting in combination are subject to considerable uncertainty, the approach used in this step was intended to be simple and conservative.

Step 9 was updated in ANSI/AISC 358-22 to be consistent with the approach used in Chapter 6. Satisfying Equation 9.7-4 checks column flange flexural yielding and eliminates bolt prying action.

Step 10 of the procedure applies to beams with flange holes and is intended to preclude beam net section fracture. Beginning with ANSI/AISC 358-22, the approach used is the same as that used in the procedure for the double-tee connection in Chapter 13 and the bolted flange plate connection in Chapter 7. Additional details are available in the Commentary to Section 13.5.

Step 17 requires checking continuity plate requirements. **Step 9** requires a column flange thickness that will adequately satisfy the column flange local bending limit state. However, the limit states of column web local yielding and web local crippling, as defined in AISC *Specification* Sections J10.2 and J10.3, respectively, must also be checked and may require continuity plates. When required, the plates are positioned at the same level as the beam flange as shown in Figures 9.4 and 9.5. Continuity plates may interfere with the placement and installation of the column flange bolts, and detailing of the continuity plates should consider this possible interference. It may be advantageous to choose a column section that does not require continuity plates to avoid this interference.

Step 20 of the procedure is supplemental if the column is a built-up box configuration. The procedure is based on the limit state of yielding (plastic moment) as defined in AISC *Specification* Section F11.1. The design assumes a simply supported condition with symmetrical point loads applied at the bolt locations.

TABLE C-9.1
Recommended W-Series
Bracket-Beam Combinations

Bracket Designation	Beam Designations
W1.0	W33×130, W30×124, W30×116, W24×131, W21×122, W21×111
W2.1	W30×108, W27×114, W27×102, W24×103, W21×93, W18×106, W18×97
W2.0	W27×94, W24×94, W24×84, W24×76, W21×83, W21×73, W21×68, W21×62, W18×86, W18×71, W18×65
W3.1	W24×62, W24×55, W21×57, W18×60, W18×55, W16×57
W3.0	W21×50, W21×44, W18×50, W18×46, W18×35, W16×50, W16×45, W16×40, W16×31

TABLE C-9.1M
Recommended W-Series
Bracket-Beam Combinations

Bracket Designation	Beam Designations
W1.0	W840×193, W760×185, W760×173, W610×195, W530×182, W530×165
W2.1	W760×161, W690×170, W690×152, W610×153, W530×138, W460×158, W460×144
W2.0	W690×140, W610×140, W610×125, W610×113, W530×123, W530×109, W530×101, W530×92, W460×128, W460×106, W460×97
W3.1	W610×92, W610×82, W530×85, W460×89, W460×82, W410×85
W3.0	W530×74, W530×66, W460×74, W460×68, W460×52, W410×75, W410×67, W410×60, W410×46.1

TABLE C-9.2
Recommended B-Series
Bracket-Beam Combinations

Bracket Designation	Beam Designations
B1.0	W33×130, W30×124, W30×116, W24×131, W21×122, W21×111
B2.1	W30×108, W27×114, W27×102, W27×94, W18×106, W18×97

TABLE C-9.2M
Recommended B-Series
Bracket-Beam Combinations

Bracket Designation	Beam Designations
B1.0	W840×193, W760×185, W760×173, W610×195, W530×182, W530×165
B2.1	W760×161, W690×170, W690×152, W690×140, W460×158, W460×144

CHAPTER 10

CONXTECH CONXL MOMENT CONNECTION

10.1. GENERAL

The ConXtech® ConXL™ moment connection is designed to provide robust, cost-effective moment framing while eliminating field welding and facilitating fast frame erection. The patented ConXL fabrication and manufacturing process utilizes forged parts, welding fixtures, and robotic welders to produce a standardized connection.

The collars and collar assemblies illustrated, and methodologies used in their fabrication and erection, are covered by one or more of the U.S. and foreign patents shown at the bottom of the first page of Chapter 10. Additional information on the ConXL connection can also be found at <http://www.conxtech.com>.

Prequalification of the ConXL moment connection is based on the 17 qualifying cyclic tests listed in Table C-10.1, as well as nonlinear finite-element modeling of the connection. The test database includes five biaxial moment connection tests. These unprecedented biaxial moment connection tests subjected the framing in the orthogonal plane to a constant shear creating a moment across the column-beam joint equivalent to that created by the probable maximum moment at the plastic hinge of the primary beams, while the framing in the primary plane was simultaneously subjected to the qualifying cyclic loading specified by ANSI/AISC 341-05 Appendix S (AISC, 2005) until failure occurred. Tests were conducted using a variety of column-to-beam strength ratios. Many tests were conducted with an intentionally reinforced column, consisting of a concrete-filled HSS with an embedded W12 (W310) inside the HSS, forcing all inelastic behavior out of the column. In one of the biaxial tests, simultaneous flexural yielding of the column was initiated during cycling. Typically, failures consist of low-cycle fatigue of a beam flange in the zone of plastic hinging, following extensive rotation and local buckling deformation.

The ConXL connection is a true biaxial moment connection capable of moment-connecting up to four beams to a column. All moment-connected columns require a full set of four collar flange top (CFT) pieces and four collar flange bottom (CFB) pieces at every beam-column moment connected joint, even if a column face has no beam present. Each column face with either a moment-connected beam or simply supported beam will have the full collar flange assembly [CFT, CFB, and collar web extension (CWX)] with the simply connected beam connected to the CWX with a standardized bolted connection.

Unlike more conventional moment frame design, which focuses on keeping the number of moment-resisting frames to a minimum for reasons of economy, the efficient ConXL system distributes the biaxial moment connection to nearly every beam-column-beam joint throughout the structure creating a distributed moment-resisting

TABLE C-10.1
Summary of ConXL Tests

Test No.	Test Condition	Column Size	Primary Axis Beam	Secondary Axis Beam	Rotation (rad)
1101	Planar	HSS 16×16× $\frac{5}{8}$ *	W18×76 RBS	N/A	0.05
1102	Planar	HSS 16×16× $\frac{5}{8}$ *	W18×119	N/A	0.05
1103	Planar	HSS 16×16× $\frac{5}{8}$ *	W24×84 RBS	N/A	0.06
1104	Planar	HSS 16×16× $\frac{5}{8}$ *	W24×104	N/A	0.05
1105	Planar	HSS 16×16× $\frac{5}{8}$ *	W24×117×6 [†]	N/A	0.04
1106	Planar	HSS 6×16× $\frac{5}{8}$ *	W24×117×9 [†]	N/A	0.04
1107	Planar	HSS 16×16× $\frac{5}{8}$ *	W21×62 RBS	N/A	0.04
1108	Planar	HSS 16×16× $\frac{5}{8}$ *	W21×62 RBS	N/A	0.06
1201	Planar	HSS 16×16× $\frac{5}{8}$ *	W30×108 RBS	N/A	0.05
1202	Planar	HSS 16×16× $\frac{5}{8}$ *	W30×108 RBS	N/A	0.05
1203	Planar	HSS 16×16× $\frac{5}{8}$ *	W30×90	N/A	0.04
1204	Planar	HSS 16×16× $\frac{5}{8}$ *	W30×90	N/A	0.04
2102	Biaxial	BU 6×16×1.25	W30×108 RBS	W30×148	0.05
2103	Biaxial	BU 16×16×1.25	W30×108 RBS	W30×148	0.06
2105	Biaxial	HSS 16×16× $\frac{1}{2}$	W21×55 RBS	W21×83	0.06
2106	Biaxial	BU 16×16×1.25**	W30×108 RBS	W30×148	0.05
2107	Biaxial	BU 16×16×1.25**	W30×108 RBS	W30×148	0.05
2111	Biaxial	BU 16×16×1.25***	W30×108 RBS	W30×148	0.047
2113	Biaxial	BU 16×16×1.25***	W30×108 RBS	W30×148	0.047

* Column consisted of HSS 16 with supplementary W12×136 housed within concrete fill.

** Built-up box fabricated using CJP groove welds.

*** Built-up box fabricated using PJP groove welds with groove weld size equal to $\frac{3}{4}$ of flange thickness.

† Beam flanges were trimmed to the indicated width in order to test the ability of the collar to withstand (a) narrow-flange beams [6 in. (150 mm) flange] and (b) maximum forces [9 in. (230 mm) flange].

BU indicates built-up box section columns.

space frame. Thus, instead of a less redundant structure with more concentrated lateral force resistance, all or almost all beam-column connections are moment-resisting creating extensive redundancy. The distribution of moment connections throughout the structure also allows for reduced framing sizes and provides excellent floor vibration performance due to fixed-fixed beam end conditions. The highly distributed lateral force resistance also provides for reduced foundation loads and an inherently robust resistance to progressive collapse.

Finite element models of tested beam-column assemblies confirm that the contribution of concrete column fill can be accounted for using the gross transformed properties of the column. Beams and columns should be modeled without rigid end offsets. Prescriptive reductions in beam stiffness to account for reduced beam section

(RBS) property reductions are conservative for ConXL framing because the RBS is located farther away from the column centerline than is typical of standard RBS connections. Therefore, modeling of ConXL assemblies employing RBS beams should model the reduced beam sections explicitly, rather than using prescriptive reductions in stiffness to account for the beam flange reduction.

Because ConXL systems have their lateral force resistance distributed throughout the structure, torsional resistance can be less than structures with required lateral force resistance concentrated on exterior lines. It is possible to minimize this effect by selecting stiffer members toward the building perimeter to increase the torsional inertia.

10.2. SYSTEMS

The ConXL moment connection is unique in that it meets the prequalification requirements for special and intermediate moment frames in orthogonal intersecting moment-resisting frames. It can also be used in more traditional plane frame applications. These requirements are met with a single standardized connection.

The exception associated with concrete structural slab placement at the column and collar assembly is based on testing conducted on the stiffened extended end-plate moment connection (Seek and Murray, 2005). Early testing by Murray of a bolted end-plate specimen with a concrete slab in place failed by tensile rupture of the bolts. This was postulated to be the result of composite action between the beam and slab, resulting in increased beam flexural strength and increased demands on the bolt relative to calculated demands neglecting composite effects. Later testing referenced previously demonstrated that placement of a flexible material in the slab adjacent to the column sufficiently reduced this composite action and protected the bolts. Although ConXL connections have not been tested with slabs present, it is believed that the same protective benefits of the flexible material apply to this connection.

ConXL's highly distributed lateral force resistance reduces the need for metal deck/concrete fill to act as a diaphragm and drag forces to a limited number of moment-resisting frames. Each moment-resisting column and connected beams resist a tributary lateral load and typically minimal concrete reinforcement or deck attachment is required.

10.3. PREQUALIFICATION LIMITS

1. Beam Limitations

Minimum beam depth is controlled by the collar dimensions and is 18 in. (450 mm). Maximum beam depth is controlled by strong-column/weak-beam considerations and is limited to 30 in. (750 mm) for practical purposes. The flange width and thickness requirements are limited by the ability of the collar flange to accommodate the beam flange weld and also by the strength of the bolts. A key ConXL requirement for allowable beam sections is limiting the force delivered by the beam to the bolts connecting the collar flange/beam to the collar corner assemblies/column so as to

not overcome the pretension load applied to the bolts. This requirement is covered in detail in Section 10.7.

ConXL connections have been successfully tested without reduced beam section reductions in flange width and are qualified for use without such reductions. However, RBS cuts in beam flanges can be a convenient way to achieve strong column weak beam limitations without increasing column weight.

Lateral bracing of beams is in accordance with the AISC *Seismic Provisions*. During the biaxial moment connection tests, the test beams (W30×108 with 50% RBS, W21×55 with 50% RBS) were not braced at the RBS and were braced at the beam ends, 10 ft (3 m) from the column center.

All moment-connected beams are required to meet seismic compaction requirements of the AISC *Seismic Provisions*, if RBS beams are used, the width-to-thickness ratio is taken within its reduced flange width as permitted for RBS connections [Section 5.3.1(6)].

2. Column Limitations

The key requirement for ConXL moment columns is a square sectional dimension of 16 in. (400 mm). Section type (built-up box or HSS) can vary, as can steel strength and wall thickness. All columns used in ConXL moment connections are concrete-filled with either normal or lightweight concrete, having minimum compressive strength of 3,000 psi (21 MPa). Columns are typically filled with concrete at the job site after erection and bolting is complete. The concrete is pumped to the top of column and allowed to free-fall the full height of column, using the column as a tremie. There are no obstructions, stiffener plates, etc., within the column; thus, the column is similar to a tremie-pipe allowing the concrete an unobstructed path to its placement with excellent consolidation (Suprenant, 2001).

Two biaxial beam-column tests (Tests 2111 and 2113, Table C-10.1) were performed with built-up box section columns fabricated using PJP groove welds to join the box section flange and web plates as illustrated in Figure 10.5. In each case, the column marginally met biaxial strong-column/weak-beam requirements, and some limited yielding of column flanges was observed in later stages of the tests. Both tests reached total rotation demands of 0.047 rad without failure or noticeable loss of load carrying ability. In addition, a single cantilever column test (Test 1120) was conducted to evaluate the inelastic behavior of box columns with PJP groove welds. In this test, a 16 in. square built-up box column with 1¼-in.- (31-mm-) thick plates joined using a PJP groove weld size of 15/16 in. (24 mm) was subjected to uniaxial ramped cyclic loading in a cantilever condition, while maintaining an applied axial load of 640 kips (2,800 kN) [approximately 9 ksi (62 MPa) axial stress]. The specimen was loaded to six cycles of displacement each to story drift ratios of 0.00375, 0.005, and 0.0075; four cycles at 0.010 rad; and two cycles each at 0.015, 0.020, 0.030, and 0.045 rad. Initial yielding was observed to occur at displacements of 0.0075 rad. Minor bowing of the flange plates occurred at 0.045 rad. The test was terminated without failure and while

still exhibiting positive strain hardening after loading to two cycles at 0.045 rad. No evidence of distress to the PJP welds in any of these tests was observed.

3. Collar Limitations

Appendix B describes the forged steel material specification used to manufacture the collars. The forging process produces an initial collar (blank collar) slightly larger than the final overall dimensions. The collar is then machined to their manufacturing dimensions within the required tolerances.

10.4. COLLAR CONNECTION LIMITATIONS

The collars are the key elements of the ConXL connection. They are standardized components, and no further design or sizing of these components is required. The same components are used for all beams and columns. The same is true for the collar bolts, where the specification, size, and number of bolts always remain the same. The design procedure ensures that column-beam combinations used in the ConXL connection fall within the code requirements of these standard connection components.

The bolts used in the ConXL connection are 1¼-in.-diameter ASTM A574 bolts. These bolts are similar in chemistry and mechanical properties to ASTM F3125/F3125M Grade A490 bolts but have socket heads to accommodate their use in this connection. Metric bolts conforming to ASTM A574M have not been tested and are not prequalified for use in this connection. Pretensioning is performed to the requirements for 1¼-in.-diameter ASTM F3125/F3125M Grade A490 bolts [102 kips (450 kN) in accordance with AISC *Specification* Table J3.1 (AISC, 2022b)].

10.5. BEAM WEB-TO-COLLAR CONNECTION LIMITATIONS

The collar web extension (CWX) is 1½ in. (38 mm) thick; thus, the minimum sized fillet weld between the CWX and beam web is a 5/16 in. (8 mm) fillet weld. This weld size for a two-sided fillet weld (each side of the web) should be sufficient for all allowable beams; this should be confirmed during the design procedure calculations.

10.6. BEAM FLANGE-TO-COLLAR FLANGE WELDING LIMITATIONS

Welding of the beam flange to the collar flange is performed in a proprietary ConX-tech beam weld fixture, which rotates the beam to allow access to the bottom flange for welding in the flat position. The beam weld fixture enables the manufacturing of the moment beam within ConXL tolerances.

10.7. DESIGN PROCEDURE

Step 3. The ConXL moment connection is a true biaxial moment connection; thus, the committee determined that columns must be sufficiently strong to permit simultaneous development of flexural hinging in all beams framing to a column, not just beams along a single plane. This biaxial column-beam moment evaluation is more conservative than current AISC *Seismic Provisions* requirements that consider plastic hinging of beams in a single plane only, even though columns supporting moment

frames in orthogonal directions are possible with other connections using built-up box sections or other built-up column sections. In calculating the ConXL biaxial column-beam moment ratio, it is permitted to take the actual yield strength of the column material in lieu of the specified minimum yield stress, F_y , and to consider the full composite behavior of the column for axial load and flexural action (interstory drift analysis). The default formula for column strength provided in the design procedure assumes that equal strength beams are present on all faces of the connection. When some beams framing to a column are stronger than others, it is permitted to use basic principles of structural mechanics to compute the actual required flexural strength.

The design procedure also considers the critical beam strength as it relates to the column strength at locations just above the beam top flange and just below the beam bottom flange, where flexural demand on the columns are greatest. Flexural demand on the column within the panel zone is less than at these locations.

Step 5. The available tensile strength for the bolts used in the ConXL connection is specified as the minimum bolt pretension load. The purpose of assigning the minimum pretension load as the available bolt tensile strength is to prevent overcoming of bolt pretension, at least up to the bolt loading subjected by the probable maximum moment. The minimum bolt pretension load is 102 kips (450 kN). Bolts are checked for tension only because the frictional force developed by the bolt pretension will resist beam shear (see **Steps 6 and 7**).

Step 6. Beam shear is resisted by the friction developed between the collar flanges and the collar corners. The collar flanges are clamped against the collar corner assemblies and column when the collar bolts are pretensioned. This pretension clamping force creates friction between the machined surfaces of the collar flanges and collar corners. The machined surfaces are classified as a Class B surface (unpainted blast-cleaned steel surfaces). From AISC *Specification* Section J3.9, the design frictional resistance per bolt is:

$$R_n = \mu D_u h_f T_b n_s \quad (\text{Spec. Eq. J3-4})$$

where

$$D_u = 1.13$$

$$T_b = 102 \text{ kips (450 kN)}$$

$$h_f = 1.0$$

$$n_s = 1$$

$$\mu = 0.50$$

$$\phi = 0.85 \text{ for oversized bolt holes}$$

$$\begin{aligned} \phi R_n &= 0.85(0.50)(1.13)(1.0)(102 \text{ kips})(1.0) \\ &= 49.0 \text{ kips/bolt (220 kN/bolt)} \end{aligned}$$

There are 16 bolts per beam end providing a total of 784 kips (3,500 kN) of frictional resistance against shear. This frictional force is significantly greater than any beam shear developed by an allowable beam.

Steps 8 and 9. The available length of weld for the collar web extension and collar corner assemblies allow for minimum-sized fillet welds to resist beam shear.

Step 11. The collar corner assemblies provide additional strength to the column walls to resist panel zone shear. Without taking into consideration the contribution of the concrete fill, the column section along with the collar corner assemblies should provide sufficient strength for anticipated panel zone shear; this should be confirmed during the design procedure calculations.

Step 12. The ConXL moment connection is a biaxial connection. Strong-column/weak-beam requirements specified by the AISC *Seismic Provisions* were formulated considering the typical planar framing prevalent in moment-frame construction following the 1994 Northridge earthquake. Because the ConXL connection is primarily used in intersecting moment frames, with biaxial behavior an inherent part of the design, the CPRP felt that it was imperative to require that columns have sufficient strength to develop expected simultaneous flexural hinging in beams framing into all column faces. The biaxial calculation considers all moment beams attached to the column.

CHAPTER 11

SIDEPLATE MOMENT CONNECTION

11.1. GENERAL

The SidePlate[®] moment connection is a post-Northridge connection system that uses a configuration of redundant interconnecting structural plates, fillet weld groups, and high-strength pretensioned bolts (as applicable), which act as positive and discrete load transfer mechanisms to resist and transfer applied moment, shear, and axial load from the connecting beam(s) to the column. This load transfer minimizes highly restrained conditions and triaxial strain concentrations that typically occur in flange-welded moment connection geometries. The connection system is used for both new and retrofit construction and for a multitude of design hazards such as earthquakes, extreme winds, and blast and progressive collapse mitigation.

The wide range of applications for SidePlate connection technology, including the methodologies used in the fabrication and erection shown herein, are protected by one or more U.S. and foreign patents identified at the bottom of the first page of Chapter 11. Information on the SidePlate moment connection can be found at www.sideplate.com. SidePlate moment connections not specifically designed by SidePlate Systems Inc. are considered unauthorized and not prequalified and should not be manufactured.

SidePlate moment connections are designed and detailed in two types:

- (1) Field-welded connection
- (2) Field-bolted connection

Both types are fully restrained connections of beams to columns conforming to Section 2.2. Figures 11.1 and 11.2 show the field-welded and field-bolted connections' various configurations, respectively. The field-bolted connection is available in three configurations as shown in Figure 11.3: configuration A (standard), configuration B (narrow), and configuration C (tuck). The field-bolted connection is also referred to as the SidePlate Plus Connection.

Moment frames that utilize the SidePlate connection system may be constructed using one of three methods. The most common construction method uses a full-length beam for erection—namely, SidePlate FRAME[®] configuration—as shown in Figure C-11.1(a) and (b). This method employs a full-length beam assembly consisting of the beam with shop-installed cover plates {B}/angles {H} (if required) and vertical shear elements (as applicable), which are either fillet-welded or bolted near the ends of the beam depending on the type of the connection.

Column assemblies are typically delivered to the job site with the horizontal shear plates {D} (as applicable) and side plates {A} shop welded to the column at the

proper floor framing locations. Where built-up box columns or HSS columns are used, horizontal shear plates {D} are not required nor applicable.

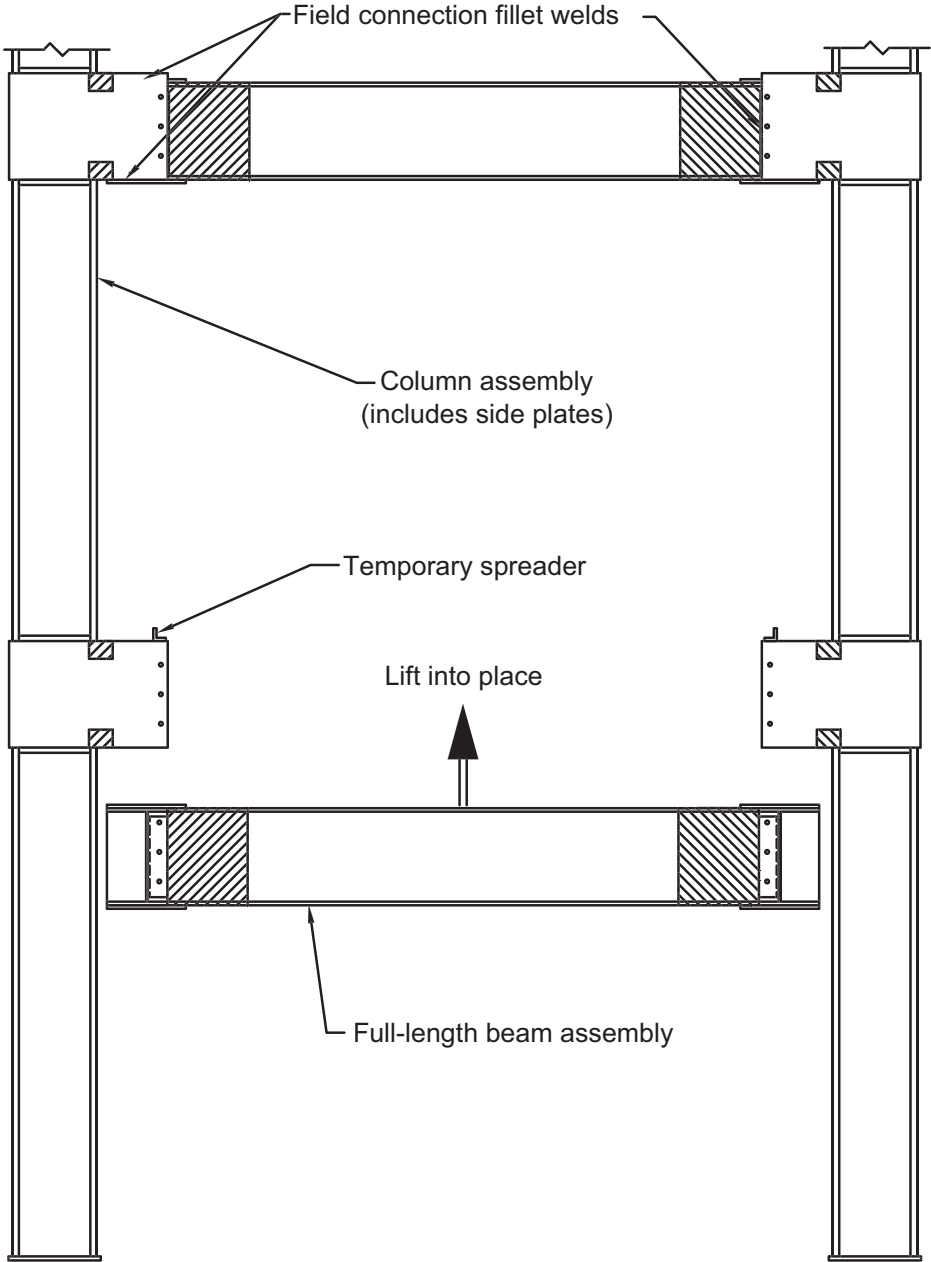
For the field-welded option: During frame erection, the full-length beam assemblies are lifted up in between the side plates {A} that are kept spread apart at the top edge of the side plates {A} with a temporary shop-installed spreader [Figure C-11.1(a)]. A few bolts connecting the beam's vertical shear plates {C} (shear elements as applicable) to adjacent free ends of the side plates {A} are initially inserted to provide temporary shoring of the full-length beam assembly, after which the temporary spreader is removed. The remaining erection bolts (as many as can be installed) are then inserted and installed to a snug-tight condition. These erection bolts can also act as a clamp to effectively close or minimize potential root gaps that might have existed between the interior face of the side plates {A} and the longitudinal edges of the top cover plate {B}, while bringing the top face of the wider bottom cover plate {B} into a snug fit with the bottom edges of the side plates {A}. To complete the field assembly, four horizontal fillet welds joining the side plates {A} to the cover plates {B} are then deposited in the horizontal welding position (position 2F per AWS D1.1/D1.1M), and, when applicable, two vertical single-pass field fillet welds joining the side plates {A} to the vertical shear elements (VSE) are deposited in the vertical welding position (position 3F per AWS D1.1/D1.1M). Alternately, this can be configured such that the width of bottom cover plate {B} is equal to the width of the top cover plate {B} (i.e., both cover plates {B} fit within the separation of the side plates {A}, which would also be slightly deeper in their lengths to accommodate), in lieu of the bottom cover plate {B} being wider than the distance between side plates {A}. Note that when this option is selected by the engineer, the two bottom fillet welds connecting the bottom cover plates {B} to the side plates {A} will be deposited in the overhead welding position (position 4F per AWS D1.1/D1.1M).

For the field-bolted option: During frame erection, the full-length beam assemblies are typically dropped down in between the side plates {A} that are kept spread apart at the bottom edge of the side plates {A} with a temporary shop-installed spreader [Figure C-11.1(b)]. A few bolts/fasteners assemblies connecting the beam's top cover plate {B} (or vertical shear plates {C} as applicable) to adjacent free ends of the longitudinal angles on the side plates {A} (or the side plates {A} themselves) are initially inserted to provide temporary shoring of the full-length beam assembly, after which the temporary spreader is removed. Shim plates may be installed between the side plates {A} and the cover plate {B} or longitudinal angles if required. The remaining bolts/fastener assemblies are then inserted to a snug-tight specification in a systematic assembly within the joint, progressing from the most rigid part of the joint until the connected plies are in as firm as contact as practicable. These bolts should clamp and effectively minimize any gaps that might have existed between the interior face of the side plates {A} and the longitudinal edges of the angles and that of the interface between the bottom face of the top cover plate {B} and the top longitudinal angles {G} on the exterior face of the side plates {A} (configuration A and configuration C). If the gaps are not closed during this process, it is acceptable to use a full length shim plate up to ¼ in. (6 mm) without penalty. Gaps that cannot be closed that are

greater than $\frac{1}{4}$ in. (6 mm) should be documented, and SidePlate Systems Inc. should be contacted for further consultation. Note that the standard and tuck configurations (configuration A and configuration C) have a pair of angles attached to the bottom flange of the beam, and the narrow configuration (configuration B) consists of pairs of angles attached to both the top and bottom flanges of the beam. To complete the field assembly, the second step of the pretensioning methodology is the subsequent systematic pretensioning of all bolt/fastener assemblies; they progress in a similar manner as was done for the snug-tight condition, from the most rigid part of the joint that will minimize relaxation of previously pretensioned bolts.

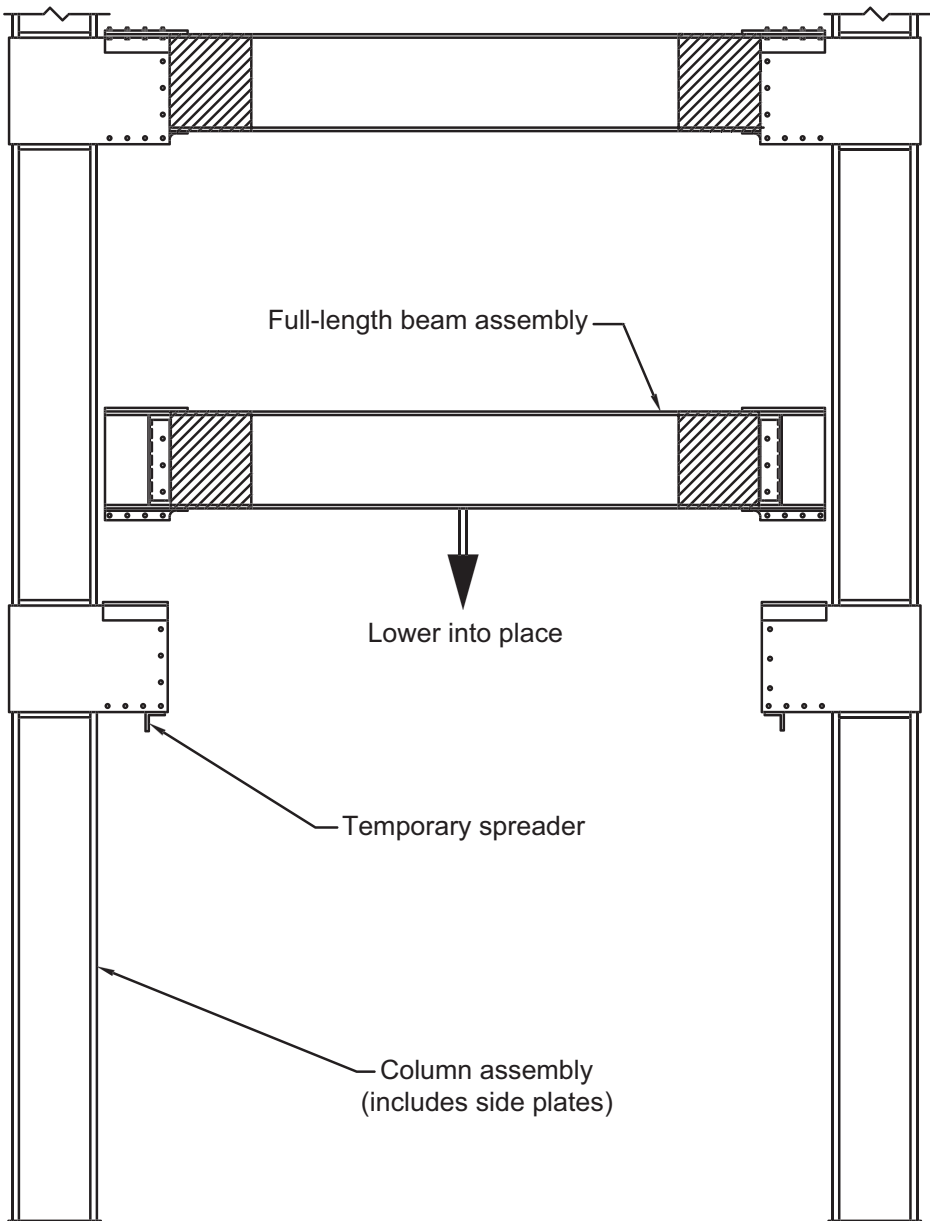
Where the full-length beam erection method (SidePlate FRAME configuration) is not used, the original SidePlate moment configuration may be used (second method). The original SidePlate moment configuration utilizes the link-beam erection method, which connects a link beam assembly to the beam stubs of two opposite column tree assemblies with field CJP groove welds [Figures C-11.1(c) and 11.1(d)]. As a third method, in cases where moment frames can be shop prefabricated and shipped to the site in one piece, no field bolting or welding is required [Figure C-11.1(e)].

The SidePlate moment connection is proportioned to develop the probable maximum moment of the connected beam. Beam flexural, axial, and shear forces are typically transferred to the top and bottom rectangular cover plates {B} via four shop horizontal fillet welds that connect the edges of the beam flange tips to the corresponding face of each cover plate {B} (two welds for each beam flange). When the U-shaped cover plates {B} or angles {H} are used, the same load transfer occurs via four shop horizontal fillet welds that connect the edge of the beam flange tips to the corresponding face of each cover plate {B}/angles {H} (two welds for each beam flange), as well as two shop horizontal fillet welds that connect the outside faces of the beams top and bottom flanges to the corresponding inside edge of each U-shaped cover plate {B} (for the conditions with pairs of angles {H}, there are two welds that will connect each angle to the corresponding beam flange face). These same forces are then transferred from the cover plates {B} or pairs of angles {H} to the side plates {A} via either four field horizontal fillet welds (in the field-welded connection) or four lines of bolts (in the field-bolted connection) that connect the cover plates {B} or pairs of angles {H} to the side plates {A}. The side plates {A} transfer all of the forces from the beam (including that portion of shear in the beam that is transferred from the web of the beam via vertical shear elements, as applicable, or via the cover plate {B} and pairs of angles {H}, as applicable) across the physical gap to the column via shop fillet welding (or flare bevel welding, as required) of the side plates {A} to the column flange tips (a total of four shop fillet welds; two for each side plate {A}), and to complete the weld group, there are two horizontally placed shop fillet welds at the top and bottom of each side plates {A}. These welds may attach directly to the face of a box or HSS column, or they may attach to horizontal shear plates {D} as applicable (a total of four shop fillet welds two for each side plate {A}). The horizontal shear plates {D} are, in turn, shop fillet welded to the column web and under certain conditions, also to the inside face of column flanges.



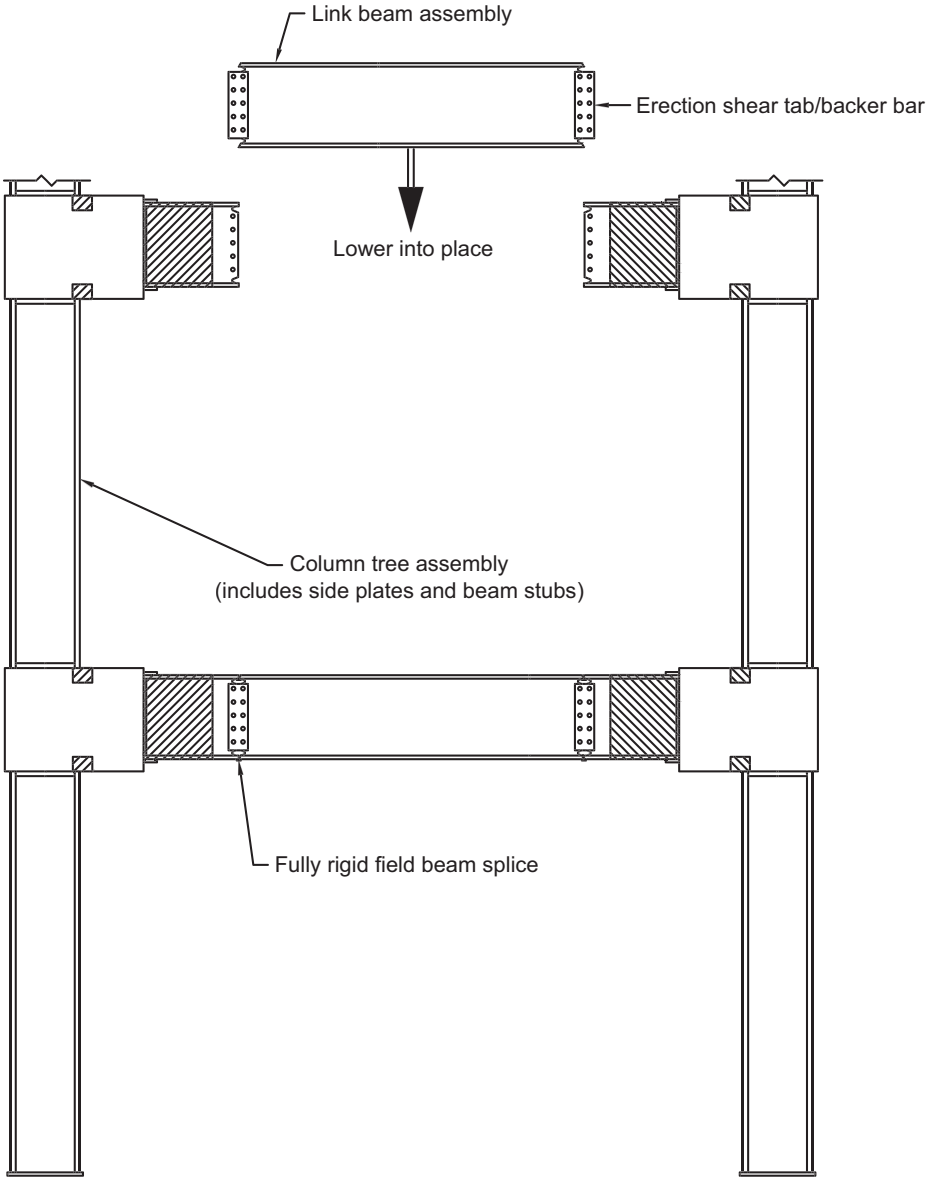
Full-length beam erection method (SidePlate FRAME configuration, field welded)

Fig. C-11.1(a). SidePlate moment connection construction methods.



Full-length beam erection method (SidePlate moment standard configuration, field bolted)

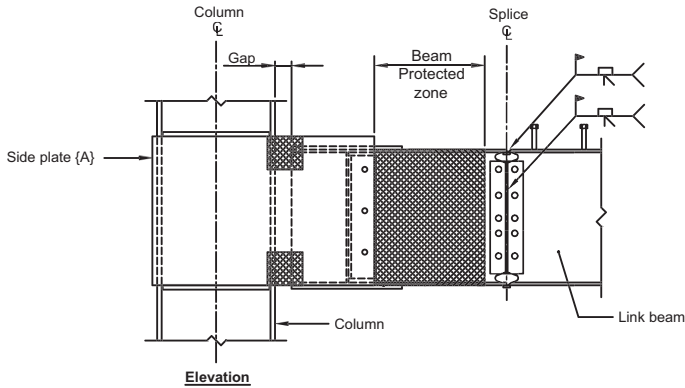
Fig. C-11.1(b). SidePlate moment connection construction methods.



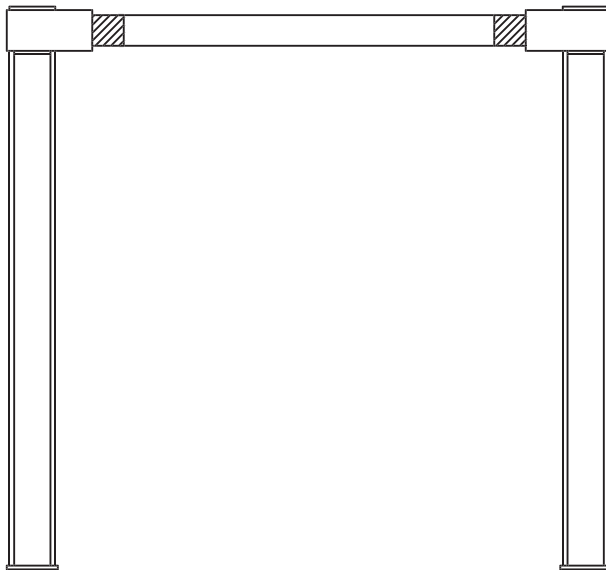
Link-beam erection method (original SidePlate moment configuration, field welded)

Fig. C-11.1(c). SidePlate moment connection construction methods.

SidePlate Systems developed, tested, and validated the SidePlate moment connection design methodology, design controls, critical design variables, and analysis procedures. The development of the SidePlate FRAME configuration that employs the full-length beam erection method builds off the research and testing history of its proven predecessor—the original configuration and its subsequent refinements. Moreover, in 2015–2017, the uniaxial field-bolted connection was developed and successfully tested and validated. In 2018, the biaxial version of the SidePlate connection with HSS and built-up box columns was developed and tested. It resulted



(d) Link beam-to-beam stub splice detail



(e) All shop-prefabricated single-story moment frame (no field welding); multistory frames dependent on transportation capabilities

Fig. C-11.1(d-e). SidePlate moment connection construction methods.

in further performance enhancements: optimizing the use of connection component materials with advanced analysis methods and maximizing the efficiency, simplicity, and quality control of its fabrication and erection processes. Following the guidance of the AISC *Seismic Provisions*, the validation of the field-welded and field-bolted SidePlate FRAME configuration consists of:

- (1) Analytical testing conducted by SidePlate Systems Inc. using nonlinear finite element analysis (FEA) for built-up and rolled shapes, plates, bolts, and welds and validated inelastic material properties by physical testing.
- (2) In addition to the tests conducted between 1994 and 2006 utilizing the original configuration, SidePlate Systems conducted physical validation testing with full-length beam assembly (SidePlate FRAME configuration) at the Lehigh University Center for Advanced Technology for Large Structural Systems (ATLSS) in 2010 (Hodgson et al., 2010a, 2010b, and 2010c; a total of six cyclic tests) and at the University of California, San Diego (UCSD), Charles Lee Powell Laboratories, in 2012 and 2013 (Minh Huynh and Uang, 2012; a total of two cyclic tests; and Minh Huynh and Uang, 2013; a total of one biaxial cyclic test). The biaxial moment connection test subjected the framing in the orthogonal plane to a constant shear, creating a moment across the column-beam joint equivalent to that created by the probable maximum moment at the plastic hinge of the primary beam, while the framing in the primary plane was simultaneously subjected to the qualifying cyclic loading specified by the AISC *Seismic Provisions* (AISC, 2016a). Also, a physical testing program was conducted at UCSD (Mashayekh and Uang, 2016; Reynolds and Uang, 2017) to validate the performance of the field-bolted SidePlate moment connection. A total of seven cyclic tests—two of which utilized HSS columns and one of which utilized a built-up box column—were conducted. The purpose of these tests was to confirm adequate global inelastic rotational behavior of either field-welded or field-bolted SidePlate moment connections with parametrically selected member sizes, corroborated by analytical testing, and to identify, confirm, and accurately quantify important limit state thresholds for critical connection components to objectively set critical design controls. The 2015–2017 testing program at UCSD additionally aimed to verify the satisfactory performance of HSS columns with a width-to-thickness ratio of up to 21 in SidePlate moment connections through the application of a significant axial load on the column in addition to the AISC *Seismic Provisions* loading protocol. The testing program also attempted to verify the satisfactory performance of SidePlate moment connections with built-up box columns without any internal horizontal shear plates {D} or stiffeners (continuity plates), where flange and web plates of built-up box columns are continuously connected by either fillet welds or PJP groove welds along the length of the column. It implies that no CJP welds will be required within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, flange, and web plates of boxed wide-flange columns in SidePlate moment connections. More recently, a full-scale testing program including four biaxial tests and one uniaxial test with HSS

and built-up box columns, was successfully conducted at UCSD (Reynolds and Uang, 2018). Two of the biaxial tests utilized the newly developed tuck configuration (configuration C) with a built-up wide-flange beam with no vertical shear element. The width-to-thickness ratio of the HSS column was 19.9, confirming the satisfactory performance of HSS columns with a width-to-thickness ratio of up to 21 once again.

- (3) Tests on SidePlate moment connections, both uniaxial and biaxial applications, show that yielding is generally concentrated within the beam section just outside the ends of the two side plates {A}. Peak strength of specimens is typically achieved at an interstory drift angle of approximately 0.03 to 0.05 rad. Specimen strength then gradually reduces due to local and lateral-torsional buckling of the beam. Ultimate failure typically occurs at interstory drift angles of approximately 0.04 to 0.06 rad for the field-welded and 0.06 to 0.08 rad for the field-bolted connection by low-cycle fatigue fracture from local buckling of the beam flanges and web.

To ensure predictable, reliable, and safe performance of the SidePlate FRAME configuration when subjected to severe load applications, the inelastic material properties, finite element modeling (FEM) techniques, and analysis methodologies that were used in its analytical testing were initially developed, corroborated, and honed based on nonlinear analysis of prior full-scale physical testing of the original SidePlate configuration. The finite element techniques and design methodologies have been further refined and polished as a result of the testing program with field-bolted connections at UCSD in 2015–2017.

The earliest physical testing of SidePlate connections consisted of a series of eight uniaxial cyclic tests, one biaxial cyclic test conducted at UCSD, and a separate series of large-scale arena blast tests. The blast tests consisted of an explosion followed by monotonic loading using the following configurations: two blast tests (one with and one without a concrete slab present), two blast-damaged progressive collapse tests, and one non-blast damaged test—all conducted by the Defense Threat Reduction Agency (DTRA) of the U.S. Department of Defense (DoD), at Kirtland Air Force Base, Albuquerque, N.M.

These extensive testing efforts have resulted in the ability of SidePlate Systems to:

- (1) Reliably replicate and predict the global behavior of the SidePlate FRAME configurations compared to actual tests.
- (2) Explore, evaluate, and determine the behavioral characteristics, redundancies, and critical limit state thresholds of its connection components.
- (3) Establish and calibrate design controls and critical design variables of the SidePlate FRAME configurations, as validated by physical testing.

Connection prequalification is based on the completion of several carefully prescribed validation testing programs, the development of a safe and reliable plastic capacity design methodology that is derived from ample performance data from

36 full-scale tests, of which six were biaxial, and the judgment of the CPRP. The connection prequalification objectives have been successfully completed; the rudiments are summarized in the following:

- (1) System-critical limit states have been identified and captured by physical full-scale cyclic testing and corroborated through nonlinear FEA.
- (2) The effectiveness of identified primary and secondary component redundancies of the connection system has been demonstrated and validated through parametric performance testing—both physical and analytical.
- (3) Critical behavioral characteristics and performance nuances of the connection system and its components have been identified, captured, and validated.
- (4) Material submodels of inelastic stress/strain behavior and fracture thresholds of weld consumables and base metals have been calibrated to simulate actual behavior.
- (5) Sufficient experimental and analytical data on the performance of the connection system have been collected and assessed to establish the likely yield mechanisms and failure modes.
- (6) Rational nonlinear FEA models for predicting the resistance associated with each mechanism and failure mode have been employed and validated through physical testing.
- (7) Based on the technical merit of the preceding accomplishments, a rational ultimate strength design procedure has been developed based on physical testing, providing confidence that sufficient critical design controls have been established to preclude the initiation of undesirable mechanisms and failure modes and to secure expected safe levels of cyclic rotational behavior and deformation capacity of the connection system for a given design condition.

11.2. SYSTEMS

The SidePlate moment connection meets the prequalification requirements for special and intermediate moment frames in both traditional in-plane frame applications (one or two beams framing into a column) as well as orthogonal intersecting moment-resisting frames (corner conditions with two beams orthogonal to one another, as well as three or four orthogonal beams framing into the same column).

The SidePlate moment connection has been used in moment-resisting frames with skewed and/or sloped beams with or without skewed side plates {A}, although such usage is outside of the scope of this standard.

The unique geometry of the SidePlate moment connection allows its use in other design applications where in-plane diagonal braces or diagonal dampers are attached to the side plates {A} at the same beam-to-column joint as the moment-resisting frame, while maintaining the intended SMF or IMF level of performance. When such dual systems are used, supplemental calculations must be provided to ensure that the connection elements (plates and welds) have not only been designed for the intended SMF or IMF connection in accordance with the prequalification limits set herein, but

also for the additional axial, shear, and moment demands due to the diagonal brace or damper.

11.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A wide range of beam sizes, including both rolled and built-up wide-flange and HSS beams, has been tested with the SidePlate moment connection, in both uniaxial and biaxial conditions. For the field-welded connection, the smallest beam size was a W18×35 (W460×52) and the heaviest a W40×297 (W1000×443). For the field-bolted connection, the smallest beam size was a W21×73 (W530×109), and the largest beam size was a W40×397 (W1000×591). The deepest beam tested was a W44×290 (W1100×433) with a depth of 43.6 in. (1110 mm). Beam compactness ratios have varied from that of a W18×35 (W460×52) with $b_f/2t_f = 7.06$ to a W40×294 (W1000×438) with $b_f/2t_f = 3.11$. For HSS beam members, tests have focused on small members such as the HSS7×4×½ (HSS177.8×101.6×12.7) having ratios of $b/t = 5.60$ and $h/t = 12.1$. As a result of these testing programs, critical ultimate strength design parameters for the design and detailing of the SidePlate moment connection systems have been developed for general project use. These requirements and design limits are the result of a detailed assessment of actual performance data coupled with independent physical validation testing and/or corroborative analytical testing of full-scale test specimens using nonlinear FEA. It was the judgment of the CPRP that the maximum beam depth and weight of the SidePlate moment connection would be limited to the nominal beam depth and approximate weight of the sections tested, as has been the case for most other connections.

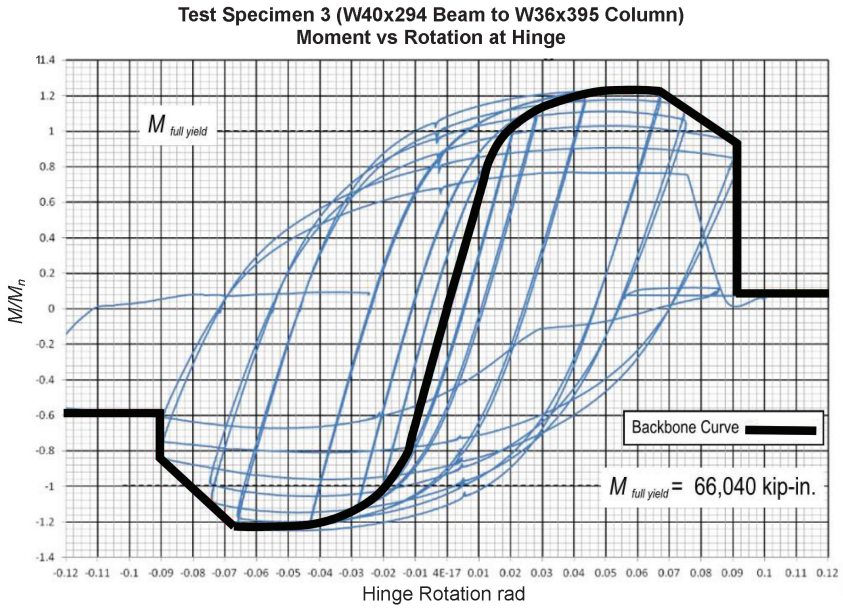
Because the behavior and overall ductility of the SidePlate moment connection systems is defined by the plastic rotational capacity of the beam, the limit state for the SidePlate moment connection system is ultimately the failure of the beam flange, away from the connection. Therefore, the limit of the hinge-to-hinge span-to-depth ratio of the beam, L_h/d , is based on the demonstrated rotational capacity of the beam.

As an example, for test specimen 3 tested at Lehigh University (Hodgson et al., 2010c), the W40×294 (W1000×438) beam connected to the W36×395 (W920×588) column reached two full cycles at 0.06 rad of rotation (measured at the centerline of the column), which is significantly higher than the performance threshold of one cycle at 0.04 rad of rotation required for successful qualification testing by the AISC *Seismic Provisions*. Most of the rotation at that amplitude came from the beam rotation at the plastic hinge. At this same 0.06 rad measured at the column centerline, the measured rotation at the beam hinge was between 0.085 and 0.09 rad [see Figure C-11.2(a)]. The tested half-span was 14.5 ft (4.4 m), which represents a frame span of 29 ft (8.8 m) and an L_h/d ratio of 5.5. Assuming that 100% of the frame system's rotation comes from the beam's hinge rotation (a conservative assumption because it ignores the rotational contributions of the column and connection elements), it is possible to calculate a minimum span at which the frame drift requirement of one cycle at 0.04 rad is maintained, while the beam reaches a maximum of 0.085 rad of

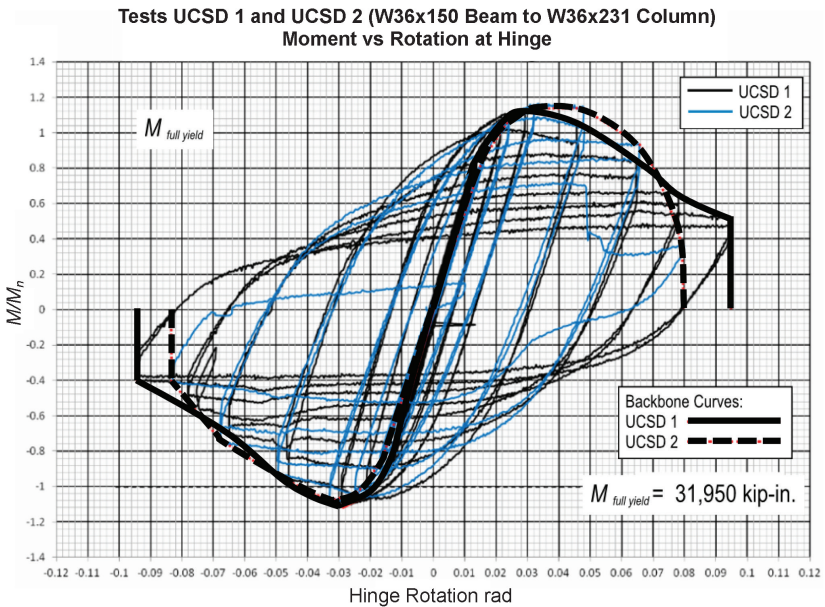
rotation. This calculation gives a minimum span of 20 ft (6.1 m) and an L_h/d ratio of 3. Making this same calculation for the tests of the W36×150 (W920×223) beam [Minh Huynh and Uang, 2012; Figure C-11.2(b)], using an average maximum beam rotation of 0.08 rad of rotation, gives a minimum span of 18 ft, 10 in. (5.7 m) and an L_h/d ratio of 3.2. Given that there will be variations in the performance of wide-flange beams due to local effects such as flange buckling, it is reasonable to set the lower bound L_h/d ratio for the SidePlate field-welded moment connection system at 4.5 for SMF and 3.0 for IMF, regardless of beam compactness. It should be noted that the minimum L_h/d ratio of 4.5 (where L_h is measured from the centerline of the beam's plastic hinges) typically equates to 6.7 as measured from the face of column to face of column when the typical side plate {A} extension (shown as "Side plate {A} extension" in Figure 11.9) from face of column is used. The 6.7 ratio, which is slightly less than the 7.0 for other SMF moment connections, allows the potential for a deeper beam to be used in a shorter bay than other SMF moment connections. The field-bolted testing program at UCSD (Mashayekh and Uang, 2016; Reynolds and Uang, 2017) showed that the field-bolted connections sustained approximately 2% more story drift, so it is reasonable to set the lower bound L_h/d ratio for the SidePlate field-bolted moment connection at 4.0 for SMF and 3 for IMF regardless of beam compactness [see Figure C-11.2(c) for the measured rotation of the field-bolted W40×211 (W1000×314) beam and Figure C-11.2(d) for the measured rotation of the field-bolted W40×397 (W1000×591) beam at the hinge location]. All moment-connected beams are required to satisfy the width-to-thickness requirements of AISC *Seismic Provisions* Sections E2 and E3.

Required lateral bracing of the beam follows the AISC *Seismic Provisions*. However, due to the significant lateral and torsional restraint provided by the side plates {A} as observed in all full-scale tests, for calculation purposes, the unbraced length of the beam is taken as the distance between the respective ends of each side plate {A} extension (see Figures 11.14 through 11.21 for depictions of the alphabetical designations). As determined by the full-scale tests, no additional lateral bracing is required at or near the plastic beam hinge location.

Lateral bracing of columns in accordance with AISC *Seismic Provisions* Section E3.4c.1 is not a requirement if the beam is sufficiently braced at the top beam flange (e.g., with a deck or slab). Sufficient bracing of the beam can be attained by either continuous bracing of the beam top flange by slab on deck or by meeting the requirements of AISC *Seismic Provisions* Section D1.2b. The substantiation for not using direct bracing is twofold. One is that none of the qualifying tests had any additional direct column bracing at the level of the top or bottom beam flanges, and the column did not exhibit any sign of torsional twisting, even for deep columns. This was due to the significant lateral and torsional restraint provided by side plates {A} that indirectly braced the column. This is in accordance with AISC *Seismic Provisions* Section E3.4c.1, where the indirect stability bracing is permitted if substantiated. Second, an internal numerical study was conducted by SidePlate Systems to demonstrate that the side plates {A} provide adequate indirect stability bracing and that the connection does not need additional lateral bracing for column stability.



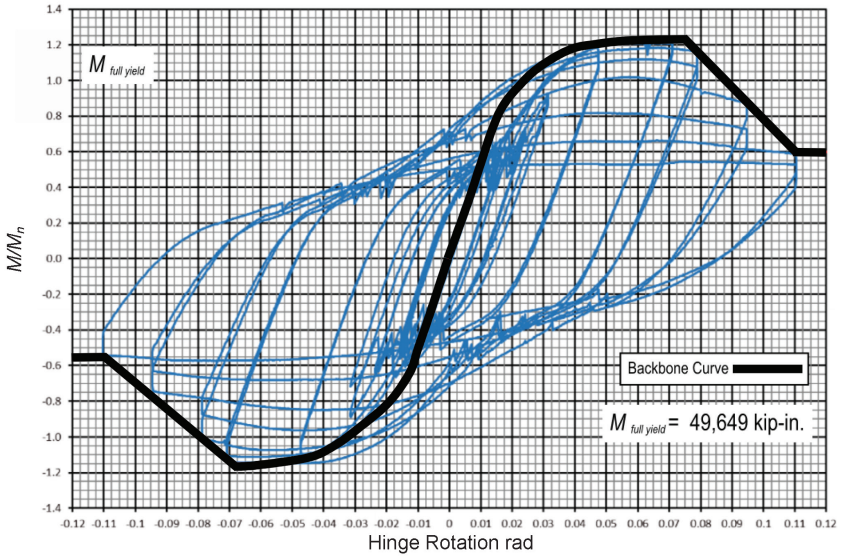
(a) W40x294 (W1000x438) beam (field welded)



(b) W36x150 (W920x223) beam (field welded)

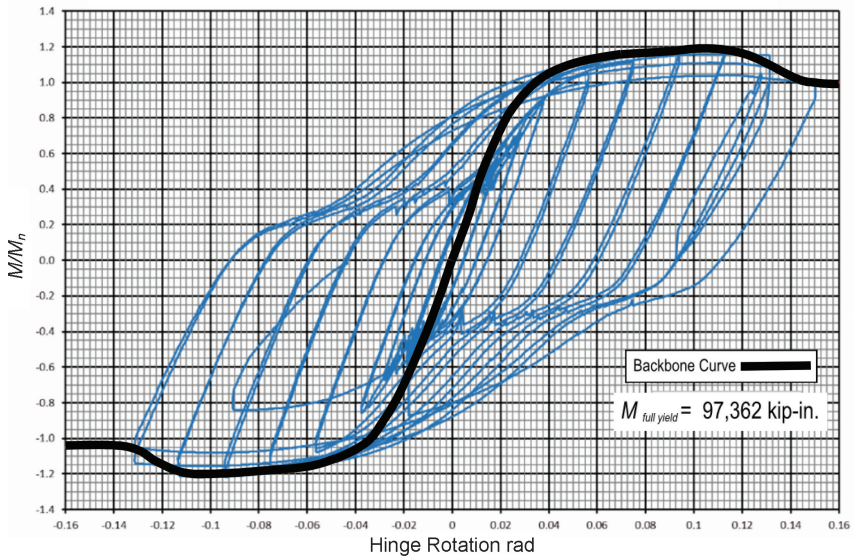
Fig. C-11.2. (a and b) SidePlate moment frame tests—backbone curves. (Measured at the beam hinge location.)

**Test Specimen U1 (W40x211 Beam to W36x282 Column Config. A)
Moment vs Rotation at Hinge**



(c) W40x211 (W1000x314) beam (field bolted)

**Test Specimen U5 (W40x397 Beam to BU30x30x2 Column Config. A)
Moment vs Rotation at Hinge**



(d) W40x397 beam (field-bolted)

Fig. C-11.2. (c and d) SidePlate moment frame tests—
backbone curves. (Measured at the beam hinge location.)

The protected zone is defined as shown in Figures 11.10 and 11.11 and extends from the end of the side plate {A} to half the beam depth beyond the plastic hinge location, which is located at one-third the beam depth in the field-welded connection and one-sixth the beam depth in the field-bolted connection beyond the end of the side plate {A} due to the cover plate {B} or angle {H} extensions. This definition is based on test observations that indicate yielding typically does not extend past 83% and 67% of the depth of the beam from the end of the side plate {A} in the field-welded and field-bolted connections, respectively.

2. Column Limitations

SidePlate[®] moment connections have been tested with W14 (W360); W16 (W410); W30 (W760); W33 (W840) built-up I-sections; W36 (W840); built-up box sections of 30×30×2 (750×750×50) and 27×27×2½ (680×680×65); and hollow structural sections (HSS), including HSS14×14×⅞ (HSS355.6×355.6×22), HSS18×18×¾ (HSS457.2×457.2×19), and HSS20×20×⅞ (HSS508×508×22). Note that when using HSS and built-up box columns, the side plates {A} transfer the loads to the column in the same way as with wide-flange columns. The only difference is that the horizontal shear component at the top and bottom of the side plates {A} now transfer that horizontal shear directly into the faces of the HSS and built-up box column using a shop fillet weld; thus, an internal horizontal shear plate {D} or stiffener plate is not required. This was verified with the execution of various tests, including a test with a W40×397 (W1000×591) beam and a 30×30×2 (760×760×50) built-up box column, as well as a W27×102 (W690×152) beam and HSS20×20×⅞ (HSS508×508×22) column, both without internal horizontal shear plates {D} or stiffeners (continuity plates). As such, built-up box columns are prequalified as long as they meet all applicable requirements of the AISC *Seismic Provisions*, with the exceptions mentioned here. There are no internal stiffener or continuity plates within the column, and there are no requirements that the columns be filled with concrete for either SMF or IMF applications. Also no CJP groove welds will be required within a zone extending from 12 in. (300 mm) above the upper beam flange to 12 in. (300 mm) below the lower beam flange, flange, and web plates of boxed wide-flange columns in SidePlate moment connections with built-up box columns. Note: In some blast or other seismic loading applications, there may be advantages to filling the HSS or built-box columns with concrete to strengthen the column.

In 2015, SidePlate Systems conducted two tests with HSS columns as part of the testing program for expanding its prequalification to field-bolted connections (Mashayekh and Uang, 2016). This configuration is also referred to as the SidePlate PLUS Connection. The secondary purpose of these tests was the inclusion of HSS columns with the width-to-thickness ratio of up to 21 in SidePlate moment connections. It was believed that the width-to-thickness ratio of the walls of HSS columns is a function of local buckling of the walls of the HSS shape in addition to the connection itself. Therefore, it was decided to apply a substantial axial load on the columns (40% nominal axial strength of the column) to test and relax the width-to-thickness limit for SidePlate moment connections. The columns performed very well, and there

was no yielding/buckling on the face of HSS columns. As a result of two full-scale physical tests and numerous numerical studies, it was confirmed that the width-to-thickness limit of HSS columns in SidePlate moment connections can be increased to 21 as long as the axial load in the column stays below 40% of the nominal axial strength of the column—that is, $0.40A_gF_y$. The HSS column in the tests complied with ASTM A500/A500M Grade C. The columns performed very well; there were no issues regarding the performance of the column. However, it was decided to limit the HSS column to ASTM A1085/A1085M per the CPRP's recommendation.

The behavior of SidePlate moment connections with cruciform columns is similar to uniaxial one- and two-sided moment connection configurations because the ultimate failure mechanism remains in the beam. Successful tests have been conducted on SidePlate moment connections with cruciform columns using W36 (W920) shapes with rolled or built-up structural tees.

In 2018, cyclic testing of five full-scale field-bolted SidePlate steel moment connections was conducted at UCSD (Reynolds and Uang, 2018) to evaluate their performance. Four of the specimens (B1, B2, B3, and B4) were biaxially loaded, while Specimen B1a was uniaxially loaded. Each of the biaxially loaded specimens underwent two stages of loading. The first stage of loading was the monotonic loading of the transverse beams to a predefined load to develop the probable maximum moment at the column face in these beams. The second stage of loading consisted of the standard cyclic loading sequence as specified in the 2016 AISC *Seismic Provisions* applied to the primary beam. The uniaxially loaded specimen was only subjected to the AISC *Seismic Provisions* cyclic loading protocol. Specimen B2 was the first specimen tested and consisted of a built-up box column $27 \times 27 \times 2\frac{1}{2}$ (680 \times 680 \times 65) with W36 \times 282 (W920 \times 420) transverse beams and a W36 \times 210 (W920 \times 313) primary beam. Specimen B1 was the second specimen tested and consisted of an HSS column $20 \times 20 \times \frac{7}{8}$ (510 \times 510 \times 22) with W27 \times 146 (W690 \times 217) transverse beams and a W27 \times 102 (W690 \times 152) primary beam. The remaining tests reused the same side plates {A}, column, and transverse beams from Specimen B1. Specimens B3 and B4 used a built-up I section 33 in. (840 mm) deep with a nominal weight of 105 lb/ft (160 kg/m). The newly developed tuck configuration (configuration C) was used for Tests B3 and B4 where the bottom angles were inverted in comparison with the standard configuration and connected to the interior face of the beam bottom flange. Specimens B1a, B3, and B4 did not use vertical shear elements. These specimens utilized ASTM A992/A992M, ASTM A572/A572M Grade 50 (Grade 345), and ASTM A500/A500M Grade C steel for the W-shapes, plates, and HSS, respectively. ASTM F3148 and ASTM F3125/F3125M Grade F2280 high-strength bolts were used in the connections. Specimen B2 ultimately failed by fracture of the beam bottom flange after completing the 5% drift cycles. Testing of Specimens B1 and B1a was stopped after achieving 7% and 6% drift, respectively, to preserve the column for future tests. The primary beams of Specimens B1 and B1a were identical except that Specimen B1a did not utilize vertical shear elements. The performance of the two beams was comparable and no degradation in the performance was observed due to the elimination of the vertical shear element.

For SMF systems, the column bracing requirements of AISC *Seismic Provisions* Section E3.4c.1 are satisfied when a lateral brace is located at or near the intersection of the frame beams and the column. Note: Full-scale tests have demonstrated that without any additional lateral bracing the full-depth side plates {A} provide the required indirect lateral bracing of the column flanges through the side plate {A}-to-column flange welds and the connection elements that connect the column web to the side plates {A}. Therefore, no additional direct lateral bracing of the column flanges is required.

3. Connection Limitations

All test specimens have used ASTM A572/A572M Grade 50 (Grade 345) plate material, and connection plates are required to comply with this material standard. There are two exceptions. The first exception is for thick plates used in the side plate interlock assembly, where material is conservatively specified to provide improved levels of notch-toughness to accommodate potentially higher toughness demands associated with the re-entrant corners in the slots. The second exception is for the vertical shear element, where ASTM A36/A36M material is permitted, based on the low stresses in this element.

Nonlinear finite element parametric modeling of side plate {A} extensions in the range of $0.65d$ to $1.7d$ have demonstrated similar overall connection and beam behavior when compared to the results of full-scale tests.

Because there is a controlled level of plasticity within the design of the two side plates {A}, the side plate {A} protected zones have been designated based upon test observations as indicated in Figures 11.10 and 11.11, respectively. It should be noted that a more conservative design methodology is used for the design of the side plates {A} of the field-bolted configuration, which results in even less yielding in the critical section of the side plates {A}. However, it was decided for consistency to assign similar protected zones for both the field-welded and the field-bolted connections.

11.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

See Figures 11.14 through 11.21 for depictions of the alphabetical and numerical designations. The beams and columns selected must satisfy physical geometric compatibility requirements between the beam flange and column flange to allow sufficient lateral space for depositing fillet welds {5} along the longitudinal edges of the beam flanges that connect to the top and bottom cover plates {B}. Equations 11.4-1a/11.4-1aM and 11.4-1b/11.4-1bM assist designers in selecting appropriate final beam and column size combinations prior to the SidePlate moment connection actually being designed for a specific project. Note: One of the field-bolted connection tests utilized a PJP weld for weld {5}, which allows for a tighter tolerance in the geometric compatibility checks. The test performed similar to others with fillet welds for weld {5}; thus, weld {5} may be deposited as a PJP weld or fillet weld as needed.

Unlike more conventional moment frame designs that typically rely on the deformation of the column panel zone to achieve the required rotational capacity, SidePlate

moment connection technology instead stiffens and strengthens the column panel zone by providing a minimum of three panel zones (the column web plus the two full-depth side plates {A}). This configuration forces the vast majority of plastic deformation to occur through flange local buckling of the beam.

The column web must be capable of resisting the panel zone shear loads transferred from the horizontal shear plates {D} through the pair of shop fillet welds {3}. The strength of the column web is thereby calculated and compared to the ultimate strength of the welds {3} on both sides of the web. To be acceptable, the panel zone shear strength of the column must be greater than the strength of the two welds. This ensures that the limit state will be failure of the welds as opposed to failure of the column web. The two side plates {A} may be used as doubler plates to check the overall panel zone strength. The following calculation and check is built into the SidePlate moment connection design software:

$$\frac{R_u}{R_n} < 1.0 \quad (\text{C-11.4-1})$$

where

R_n = nominal strength of column web panel zone in accordance with AISC Specification Section J10.6(b), kips (N)

$$= 0.60F_y d_c t_{cw} \left(1 + \frac{3b_{fc} t_{fc}^2}{d_{sp} d_c t_{cw}} \right) \quad (\text{from Spec. Eq. J10-11})$$

R_u = ultimate strength of fillet welds {3} from horizontal shear plates {D} to column web, kips (N)

where

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

d_c = depth of column, in. (mm)

d_{sp} = depth of side plate {A}, in. (mm)

t_{cw} = thickness of column web, in. (mm)

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

In determining the SMF column-beam moment ratio to satisfy strong column/weak beam design criteria, the beam-imposed moment, M_{pb}^* , is calculated at the column centerline using statics (i.e., accounting for the increase in moment due to shear amplification from the location of the plastic hinge to the center of the column as a result of the development of the probable maximum moment of the beam, M_{pr} , at the plastic hinge location), and then linearly decreased to one-quarter the column depth above and below the extreme top and bottom fibers of the side plates {A}. This location is used for determination of the column strength because the column is unlikely to form a hinge within the panel zone due to the presence and strengthening effects of the two side plates {A}.

In calculating the biaxial column-beam moment ratio, it is permitted to take the actual yield strength of the column material as the specified yield strength in lieu of the specified minimum yield stress, F_y , and to consider the full composite behavior of

the column for axial and flexural loading action (story drift analysis) if it is filled with concrete. The column strength formula, Equation 11.4-6, assumes equal column properties about both axes. For column sections with unequal properties about both axes, interaction equations based on rational analysis should be used. Guidance for checking columns subject to biaxial bending and axial force is provided in AISC *Seismic Provisions* Section E3 Commentary.

This requirement need not apply if any of the exceptions articulated in AISC *Seismic Provisions* Section E3.4a are satisfied. The calculation and check are included in the SidePlate connection design software.

11.5. CONNECTION WELDING LIMITATIONS

Fillet welds joining the connection plates to the beam and column provided on all of the SidePlate test specimens have been made by either of the self-shielded flux cored arc welding processes (FCAW-S or FCAW-G), with a few specimens using the submerged arc welding process (SAW) for certain shop fillet welds. Other than the original three prototype tests in 1994 and 1995 that used a non-notch-tough weld electrode, tested electrodes satisfy minimum Charpy V-notch toughness as required by the 2010 AISC *Seismic Provisions*. Also, it should be noted that typically the test specimens were fitted and tacked together using an E7018 stick electrode and then welded with an FCAW process (implying that the intermixing of FCAW and E7018 has been tested and is not of concern). Test specimens that included either a field CJP groove-welded beam-to-beam splice or field fillet welds specifically utilized E70T-6 for the horizontal position and E71T-8 for the vertical position.

11.6. CONNECTION DETAILING

Figures 11.14 through 11.16 show typical one- and two-sided moment connection details used for shop fabrication of the column with fillet welds. Tests have shown that the horizontal shear plate {D} need not be welded to the column flanges for successful performance of the connection. However, if there are orthogonal forces being transferred through the connection from collector, chord, or cantilever beams, then fillet welds connecting the horizontal shear plates {D} and the column flanges may be required.

In the field-welded connection, tests have shown that the use of oversized bolt holes in the side plates {A}, located near their free end (see Figure C-11.3), does not affect the performance of the connection because beam moments and shears are transferred through fillet welds. Bolts from the side plate {A} to the vertical shear element are only required for erection of the full-length beam assembly prior to field welding of the connection and may be removed, at the contractor's discretion after the field fillet welds have been applied (also implying that if all the erection bolts cannot be placed, it is acceptable as it relates to the performance of the connection).

Figures 11.17 and 11.18 show the typical full-length beam detail used for shop fabrication of the beam with fillet welds. Multiple options can be used to create the

vertical shear element (if needed), such as a combination of angles and plates or simply bent plates.

Figures 11.19(a) and 11.19(b) show the typical full-length beam-to-side plate {A} detail used for field erection of the beam with fillet welds and bolts, respectively. In the field-bolted connection, either longitudinal angles {G} (rolled or built-up) or horizontal plates {T} that are welded to the side plates {A} may be used to transfer the load from the beam to the side plates {A}, as shown in Figure 11.19(b).

Figures 11.20 and 11.21 show shop weld {9} connecting side plate {A} to column face as well as shop weld {10} that connects the intersecting orthogonal side plates to construct the side plate interlock assembly in biaxial connections.

11.7. DESIGN PROCEDURE

The design procedure for the SidePlate moment connection system is based on results from both physical testing and detailed nonlinear FEM. The procedure uses an ultimate strength design approach to size the plates and welds in the connection, incorporating strength, plasticity, and fracture limits. For welds, an ultimate strength analysis incorporating the instantaneous center of rotation is used (as described in AISC *Steel Construction Manual* Part 8). For bolts, an ultimate strength analysis incorporating eccentric bolt group design methodology and instantaneous center of rotation is used (as described in AISC *Specification* Section J2.4b). Overall, the design process is consistent with the expected seismic behavior of an SMF system: Lateral drifts due to seismic loads induce moments and shear forces in the columns and beams. Where these moments exceed the yield strength of the beam, a plastic hinge will form. While

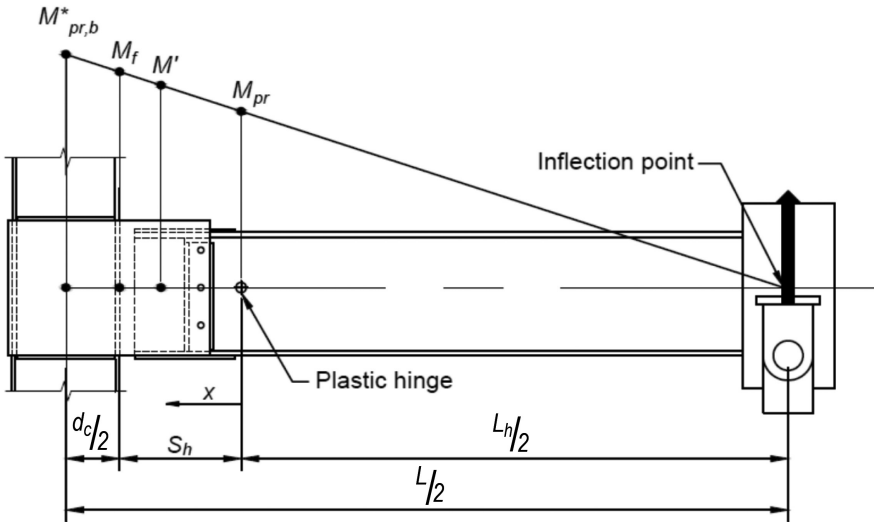


Fig. C-11.3. Amplification of maximum probable plastic hinge moment, M_{pr} , to the column face.

the primary yield mechanism is plastic bending in the beam, in the field-welded connection, a balanced design approach allows for secondary plastic bending to occur within the side plates {A} (hence the reasoning for the protected zones on the side plates {A} for this option). In the field-bolted connection, more conservative side plate {A} design methodology has been developed so that secondary plastic hinging within the side plates {A} does not occur (hence the protected zones on the side plates {A} in this option are not required). Ultimately, the location of the hinge in the beam directly affects the amplification of load (i.e., moment and shear from both seismic and gravity loads) that is resisted by the components of the connection, the column panel zone, and the column, as shown in Figure C-11.3. Each connection component can then be designed to resist its respective load demands induced by the seismic drift (including any increases due to shear amplification as measured from the plastic hinge location in the beam).

For the SidePlate moment connection, all of the connection details—including the sizing of connection plates, angles, fillet welds, and bolts—are designed and provided by engineers at SidePlate Systems. The design of these details is based on basic engineering principles, plastic capacities validated by full-scale testing, and nonlinear FEA. A description of the design methods is presented in **Step 7**. The initial design procedure for the engineer of record in designing a project with SidePlate moment connections largely involves:

Sizing the frame's beams and columns, shown in **Steps 1** and **2**.

Checking applicable building code requirements and performing a preliminary compliance check with all prequalification limitations, shown in **Steps 3** and **4**.

Verifying that the SidePlate moment connections have been designed with the correct project data as outlined in **Step 5** and are compliant with all prequalification limits, including final column-beam relationship limitations as shown in **Steps 6, 7, and 8**.

Step 1. Equations 11.4-1a/11.4-1aM and 11.4-1b/11.4-1bM should be used as a guide in selecting beam and column section combinations during design iterations.

Satisfying these equations minimizes the possibility of incompatible beam and column combinations that cannot be fabricated and erected or that may not ultimately satisfy column-beam moment ratio requirements.

Step 2. The SidePlate moment connection design forces a plastic hinge to form in the beam beyond the extension of the side plates {A} from the face of the column (side plate {A} extension in Figure 11.9). Because inelastic behavior is forced into the beam at the hinge, the effective span of the beam is reduced, thus increasing the lateral stiffness and strength of the frame (see Figure C-11.4). This increase in stiffness and strength provided by the two parallel side plates {A} must be simulated when creating elastic models of the steel frame. Many commercial structural analysis software programs have a built-in feature for modeling the stiffness and strength of the SidePlate moment connection.

Step 5. Some structural engineers design moment-frame buildings with a lateral-only computer analysis. The results are then superimposed with results from additional lateral and vertical load analysis to check beam and column stresses. Because these additional lateral and vertical loads can affect the design of the SidePlate moment connection, they must also be submitted with the lateral-only model forces. Such additional lateral and vertical loads include drag and chord forces, factored shear loads at the plastic hinge location due to gravity loads on the moment frame beam itself, loads from gravity beams framing into the face of the side plates {A}, and gravity loads from cantilever beams (including vertical loads due to earthquakes) framing into the face of the side plates {A}.

There are instances where an in-plane lateral drag or chord axial force needs to transfer through the SidePlate moment connection, as well as instances where it is necessary to transfer lateral drag or chord axial forces from the orthogonal direction through the SidePlate moment connection. In such instances, these loads must be submitted in order to properly design the SidePlate moment connection for these conditions.

Step 6 of the procedure requires SidePlate Systems to review the information received from the structural engineer, including the assumptions used in the generation of final beam and column sizes to ensure compliance with all applicable building code requirements and prequalification limitations contained herein. Upon reaching concurrence with the structural engineer of record that beam and column sizes are acceptable and final, SidePlate Systems creates a load matrix of the entire structure with these member sizes, including all submitted applicable loads and forces, and designs and details all of the SidePlate moment connections for a specific project in accordance with **Step 7**. Any changes in member sizes, loads, or forces need to be coordinated with SidePlate Systems because such changes will typically require this step to be repeated.

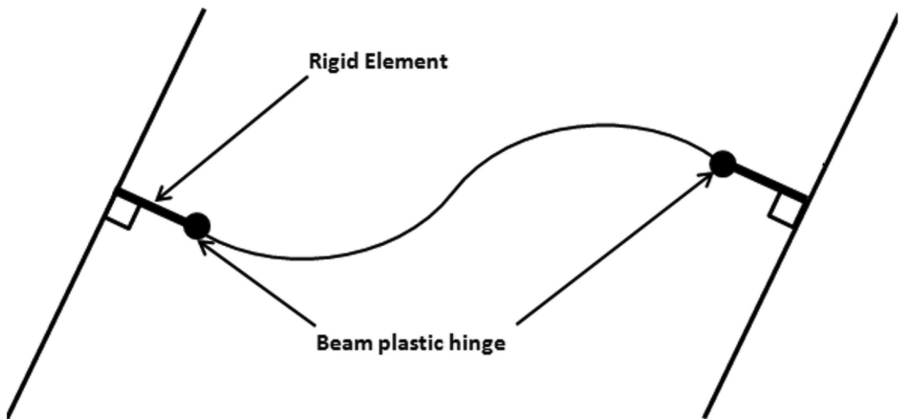


Fig. C-11.4. Increased frame stiffness with reduction in effective span of the beam.

The SidePlate moment connection design procedure is based on the idealized primary behavior of an SMF system—the formation of a plastic hinge in the beam outside of the connection. In the field-welded connection, although the primary yield mechanism is the development of a plastic hinge in the beam near the end of the side plate {A}, secondary plastic behavior (plastic moment capacity) is developed within the side plates {A} themselves, at the face of the column (this is not the case for the field-bolted connections). Overall, a balanced design is used for the connection components to ensure that the plastic hinge will form at the predetermined location. The demands on the connection components are a function of the strain-hardened flexural strength of the beam, the gravity loads carried by the beam, and the relative locations of each component and the beam plastic hinge. Connection components closer to the column centerline are subjected to increased moment amplification compared to components located closer to the beam plastic hinge as illustrated in Figure C-11.3.

Step 7 of the process requires that SidePlate Systems design and detail the connection components for the actions and loads determined in **Step 6**. The procedure uses an ultimate strength design approach to size plates, bolts, and welds, thus incorporating strength, plasticity, and fracture limits. For welds, an ultimate strength analysis incorporating the instantaneous center of rotation is used (as described in AISC *Steel Construction Manual* Part 8). For bolts, an ultimate strength analysis incorporating eccentric bolt group design methodology and instantaneous center of rotation is used (as described in AISC *Specification* Section J2.4b). Overall, the design process is consistent with the expected seismic behavior of an SMF system as described previously.

The SidePlate moment connection components are divided into four distinct design groups:

- (1) Load transfer out of the beam
- (2) Load transfer into the side plates {A}
- (3) Design of the side plates {A} at the column face
- (4) Load transfer into the column

The transfer of load out of the beam is achieved through welds {4} and {5}. The loads are in turn transferred through the vertical shear elements {E} and cover plates {B} into the side plates {A} by either welds {6} and {7} (field-welded) or bolt group (field-bolted). The load at the column face (gap region) is resisted solely by the side plates {A}, which transfers the load directly into the column through weld {2} and weld {1} in a box or HSS. In a wide-flange column, the load is transferred through weld {2} and indirectly through weld {3} through the combination of weld {1} and the horizontal shear plates {D}. At each of the four design locations, the elements are designed for the combination of moment, M_{group} , and shear, V_u .

Connection Design

Side Plate {A}, Field-Welded. To achieve the balanced design for the connection—the primary yield mechanism developing in the beam outside of the connection with secondary plastic behavior within the side plates {A}—the required minimum

thickness of the side plate {A} is calculated using an effective side plate {A} plastic section modulus, Z_{eff} , generated from actual side plate {A} behavior obtained from stress and strain profiles along the depth of the side plate {A}, as recorded in test data and nonlinear analysis (see Figure C-11.5). The flexural strength of the plates, $M_{n,sp}$, is then calculated using the simplified Z_{eff} and an effective plastic stress, F_{ye} , of the plate. Allowing for yielding of the plate as observed in testing and analyses (see Figure C-11.6) and comparing to the design demand, M_{group} , calculated at the face of column gives:

$$\frac{M_{group}}{M_{n,sp}} \leq 1.0 \tag{C-11.7-1}$$

where

$$M_{n,sp} = F_{ye} Z_{eff}$$

Side Plate {A}, Field-Bolted. The required minimum thickness of the side plate {A} is calculated based on the engineering principals of the fully yielded section at either the column face or at the location of the first bolt as shown in Figures C-11.7(a) and C-11.7(b). The section of the side plate {A} at the column face has larger design demand in comparison with that of the net section at the location of the first bolt so the required minimum thickness will be the greater of the two design checks.

To ensure the proper behavior of the side plate {A} and to preclude undesirable limit states, such as buckling or rupture of the side plate {A}, the ratio of the gap distance between the end of the beam and the face of the column to the side plate

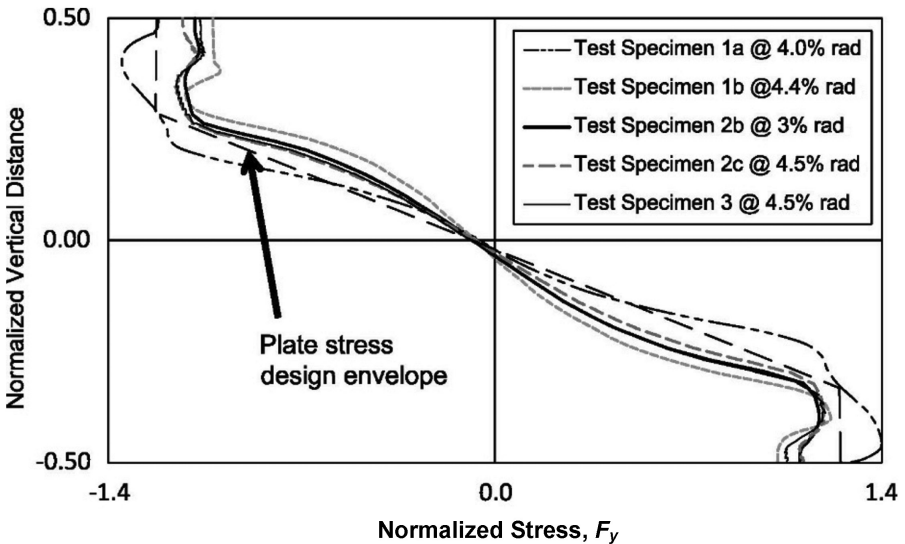


Fig. C-11.5. Stress profile along depth of side plate {A} at the column face at maximum load cycle.

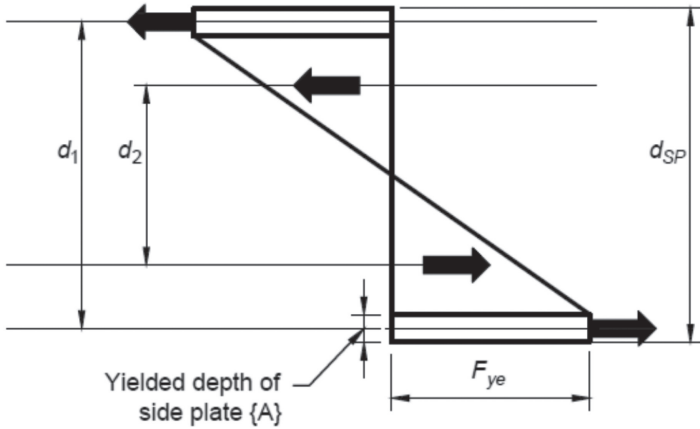
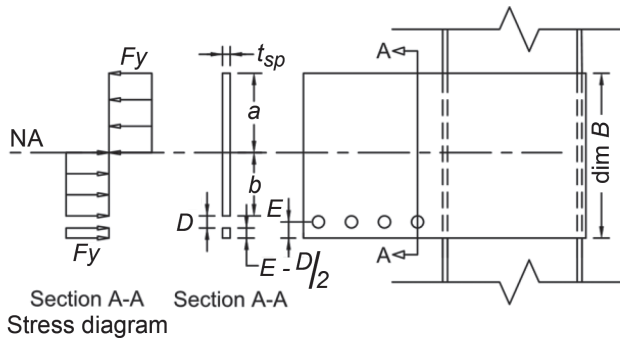
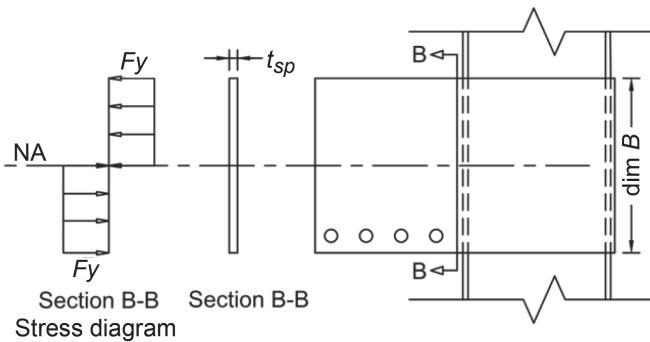


Fig. C-11.6. Idealized plastic stress distribution for computation of the effective plastic modulus, Z_{eff} , of the side plate.



(a) Side plate {A} elevation view and stress diagram at the net section



(b) Side plate {A} elevation view and stress diagram at the column face

Fig. C-11.7. Side plate {A} elevation view and stress diagrams for configuration A (standard).

{A} thickness is kept within a range for all connection designs. The optimum gap-to-thickness ratio has been derived based upon the results of full-scale testing and parametric nonlinear analysis.

Cover Plate {B}. The thickness of the cover plates {B} is determined by calculating the resultant shear force demand, R_u , from the beam moment couple as:

$$R_u = M_{group}/d \quad (C-11.7-2)$$

and by calculating the vertical shear loads, resisted through the critical shear plane of the cover plates {B}.

The critical shear plane for the field-welded connection is defined as a section cut through the cover plate {B} adjacent to the boundary of weld {7}, as shown in Figure C-11.8(a). Hence, the thickness, t_{cp} , of the cover plate {B} is:

$$t_{cp} = \frac{R_u}{2(0.6)F_y L_{crit}} \quad (C-11.7-3)$$

where

L_{crit} = length of critical shear plane through cover plate {B} as shown in Figure C-11.8(a), in. (mm)

The top cover plate {B} in the field-bolted connection (standard configuration) is designed based on the block shear check in the critical shear plane which is defined as a section cut through the cover plate {B} through the bolt holes, as shown in Figure C-11.8(b).

Vertical Shear Element (VSE). The thickness of the VSE, if applicable (which may include angles {E} and/or bent plates {C}, as shown in Figures 11.14–11.19), is determined as the thickness required to transfer the vertical shear demand from the beam web into the side plates {A}. The vertical shear force demand, V_u , at this load transfer comes from the combination of the capacities of the cover plates {B} and the VSE. The minimum thickness of the VSE, t_{vse} , to resist the vertical shear force is computed as follows:

$$t_{vse} = \frac{V'_u}{2(0.6)F_y d_{pl}} \quad (C-11.7-4)$$

where

V'_u = calculated vertical shear demand resisted by VSE, kips (N)

d_{pl} = depth of vertical shear element, in. (mm)

Horizontal Shear Plate (HSP) {D}. The thickness of the HSP {D}, if applicable (see Figures 11.14–11.19), is determined as the thickness required to transfer the horizontal shear demand from the top (or bottom) of the side plates {A} into the column web. The shear demand on the HSP is calculated as the design load developed through the fillet weld connecting the top (or bottom) edge of the side plates {A} to the HSP (weld {1}). The demand force is determined using an ultimate strength

analysis of the weld group at the column (weld {1} and weld {2}) as described in the following section.

$$t_{hsp} = \frac{V_u''}{0.6F_y l_{pl}} \tag{C-11.7-5}$$

where

V_u'' = calculated horizontal shear demand delivered by weld {1} to the HSP, kips (N)

l_{pl} = effective length of horizontal shear plate {D}, in. (mm)

Welds. Welds are categorized into three weld groups and sized using an ultimate strength analysis.

The weld groups are categorized as follows (see Figures 11.14-11.21 and Figure C-11.9):

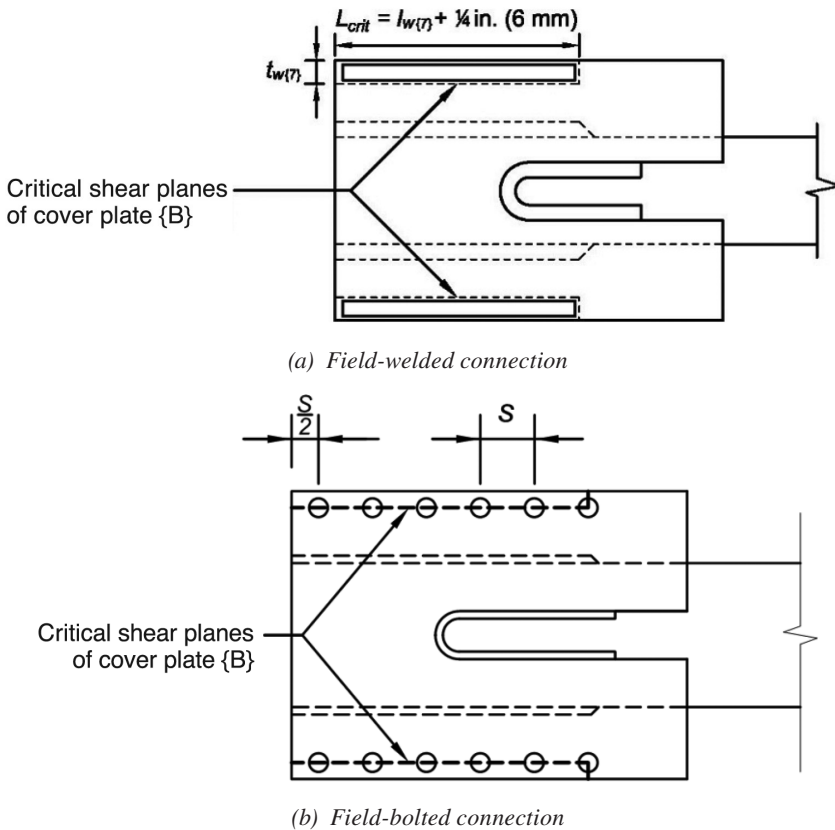


Fig. C-11.8. Critical shear plane of cover plate {B}.

Weld Group 1—Fillet welds from the beam flange to the cover plate {B}/angles {H} (weld {5} and weld {5a}) and the fillet weld from the beam web to the VSE (weld {4}).

Weld Group 2—Fillet welds from the cover plate {B} to the side plate {A} (weld {7}) and fillet welds from the VSE to the side plate {A} (weld {6}) (only for field-welded connections).

Weld Group 3—Fillet welds from the side plate {A} to the HSP {D} (weld {1}), fillet welds from the side plate {A} to the column flange tips (weld {2}), and fillet welds from the HSP {D} to the column web (weld {3}).

The ultimate strength design approach for the welds incorporates an instantaneous center of rotation method as shown in Figure C-11.10 and described in AISC *Steel Construction Manual* Part 8.

At each calculation iteration, the nominal shear strength, R_n , of each weld group for a determined eccentricity, e , is compared to the demand from the amplified moment to the instantaneous center of the group, V_{pre} . The process is continued until equilibrium is achieved. Because the process is iterative, SidePlate Systems engineers use design calculation software to compute the weld sizes required to achieve the flexural and shear strength needed for each weld group to resist the amplified flexural and vertical shear demand, M_{group} and V_u , respectively.

Bolts (Field-Bolted Connection Only). The ultimate strength analysis incorporating eccentric bolt group design methodology and instantaneous center of rotation as shown in Figure C-11.11 and described in AISC *Specification* Section J2.4(b) is

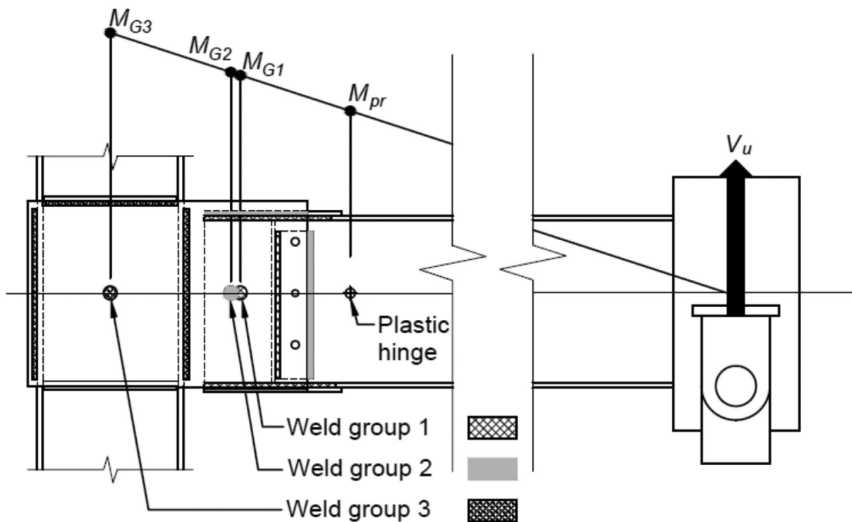


Fig. C-11.9. Location of design weld groups and associated moment demand, $M_{G\#}$.

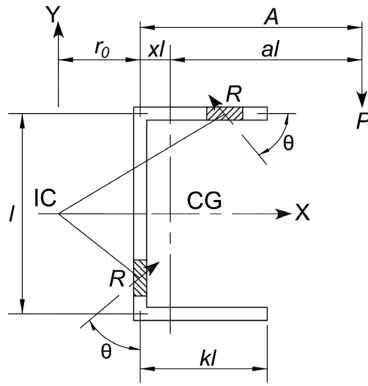


Fig. C-11.10. Instantaneous center of rotation of a sample weld group.

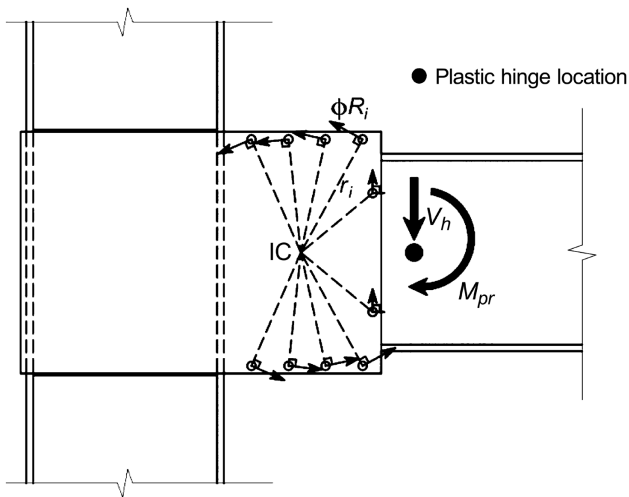


Fig. C-11.11. Instantaneous center of rotation of a sample bolt group.

used to design the number of required bolts. An iterative process is required to find the solution. At each calculation iteration, the nominal shear strength, R_n , of the bolt group (comprising horizontal and vertical rows of bolts), for a determined eccentricity, e , is compared to the demand from the amplified moment and shear to the instantaneous center of the group, V_{pre} . The process is continued until equilibrium is achieved.

Step 8 requires that the engineer of record review calculations and drawings supplied by SidePlate Systems engineers to ensure that all project-specific moment connection designs have been appropriately completed and that all applicable project-specific design loads, building code requirements, building geometry, and beam-to-column combinations have been satisfactorily addressed.

The CPRP has prequalified the SidePlate moment connection after reviewing the proprietary connection design procedure contained in the SidePlate moment connection design software (version 16 for welded and version 17 for bolted), as summarized here. In the event that SidePlate moment connection designs use a later software version to accommodate minor format changes in the software's user input summary and output summary, the SidePlate moment connection designs will be accompanied by a SidePlate moment connection validation report that demonstrates that the design dimensions, lengths, and sizes of all plates and welds generated using the CPRP-reviewed connection design procedure remain unchanged from that obtained using the later version connection design software. Representative beam sizes to be included in the validation report are W36×150 (W920×223) and W40×294 (W1000×438) for the field-welded and W36×150 (W920×223), W40×211 (W1000×314), and W40×397 (W1000×591) for the field-bolted connection.

CHAPTER 12

SIMPSON STRONG-TIE STRONG FRAME MOMENT CONNECTION

12.1. GENERAL

The Simpson Strong-Tie® Strong Frame® moment connection uses patented Yield-Link® structural fuse technology to create a field-bolted, partially restrained (PR) moment connection for strong-axis wide-flange beam-to-column connections. The Yield-Links are either configured as separate T-stub elements connected to each beam flange, or they are connected to a common end plate for shallow beam connections. During seismic events, inelastic demand is absorbed in the Yield-Link elements of the beam-to-column connection instead of requiring the formation of a plastic hinge in the beam adjacent to the column. The connection eliminates field welding, and the frame behavior afforded by the connection enables frames to be designed without the need for flange bracing on the beams. This is particularly useful in structures where providing flange bracing can be difficult (such as when integrated into wood structures) or is an undesirable architectural intrusion. Connection testing qualified the use of snug-tight bolts for typical field-installed bolts, simplifying bolt installation, inspection, and frame erection.

The connection centers around the Yield-Link (Link) structural fuse performance and a capacity-based design procedure that, under lateral loading, pushes inelastic demand into the Links rather than the members. Unlike other prequalified special moment frame (SMF) connections, little if any inelastic behavior is expected in the members. Whether configured as a modified T-stub or connected to a common end plate, the Link serves to transfer moment from the beam to the column. The connection bolts to the column flange with four snug-tight ASTM F3125/F3125M Grade A325 or A325M bolts at each link (pretensioned ASTM F3125/F3125M Grade A325, A325M, or F1852 bolt assemblies are also permitted). The stem of the Link bolts to the beam flange with pretensioned ASTM F3125/F3125M Grade A325, A325M, A490, A490M, F1852, or F2280 bolt assemblies. In between the connection to the beam and column, the stem of the Link is elongated and contains a section with reduced area that defines the location of yielding in the Link. This reduced area controls the axial strength of the Link and provides for very reliable estimates of the yield and ultimate moment strength of the beam-to-column connection. To prevent buckling of the yielding section of the Link when in compression, a buckling restraint plate (BRP) is placed over the Link and bolted to the beam flange on either side of the reduced-area section of the Link. The BRP uses snug tight ASTM F3125/F3125M Grade A325 or A325M bolts that pass through a spacer plate that fills the gap between the bottom of the BRP and the near surface of the beam flange. The web of the beam connects to either the column or the common end plate via a single-plate shear

connection. The connection uses an arrangement of bolts that permit transfer of shear and axial forces between the beam and column, while at the same time limiting the transfer of moment. This is accomplished by having a central pivot point defined by a central bolt passing through standard holes in both the beam web and the shear plate and by having the remaining bolts in the shear plate pass through either horizontal or vertical slots in the shear plate and standard holes in the beam web. This arrangement creates a hinge in the beam web-to-column flange connection and defines the effective rotation point for the plastic hinge. Shear-plate bolts are permitted to be snug-tight ASTM F3125/F3125M Grade A325 or A325M or pretensioned ASTM F3125/F3125M Grade A325, A325M, or F1852. The hinge is used to transfer net axial force from the beam to the column, so in addition to shear- and moment-related design provisions found in other prequalified moment connections, this connection also contains design provisions for axial load transfer.

Initial qualification testing consisted of a series of nine reversed cyclic tests according to the 2010 AISC *Seismic Provisions* Section E3.6c (AISC, 2010a) covering three configurations, each with three replications. Each test consisted of a single-story, single-bay frame with lateral loads (in-plane shear) introduced into the top flange of the beam through a wood nailer connected to the beam flange. Only one end of the beam used the Strong Frame connection, and the remaining beam-to-column and column-to-test bed connections were pinned. This configuration was chosen for testing the connection over the typical cantilever beam configuration for two primary reasons: It allowed beam axial loads to be driven through the joint to enable verification of both the axial and moment related design provisions, and it permitted observation of the beam flange response when flange bracing was omitted. The testing resulted in all frames reaching a drift level of 0.05 rad without a loss of strength greater than 20% of the nominal plastic moment strength, M_p , satisfying the requirements of the 2010 AISC *Seismic Provisions* Section E3.6b. For this connection, M_p is calculated using the yielding area of the links and the connection geometry rather than the beam properties.

At the current time there are no other PR connections that have been prequalified as an SMF connection, and PR SMF connections were not directly addressed in the 2010 AISC *Seismic Provisions*. The 2016 AISC *Seismic Provisions* (AISC, 2016a) address the potential use of PR SMF connections in Section E3.6b.(b). Accordingly, even though the initial testing met the SMF connection performance requirements of the 2010 AISC *Seismic Provisions*, additional requirements were created to demonstrate the suitability of the connection and the design procedure for use as SMF or IMF connections in high-seismic applications.

The first additional requirement was to assess the connection performance through a component equivalency evaluation using the procedures found in FEMA P-795, *Quantification of Building Seismic Performance Factors: Component Equivalency Methodology* (FEMA, 2011). An independent study was commissioned to perform the FEMA P-795 evaluation, comparing the Strong Frame connection to the reduced beam section (RBS) connection, resulting in two changes to the design procedure. The Link flange-to-stem weld was required to develop the full strength of the

unreduced portion of the stem at the column side (it had been previously designed for the probable maximum tensile strength of the reduced yielding area); and a single thickness of stem material, 1/2 in. (13 mm), was selected (initially different thicknesses were considered). Six additional tests similar to those described previously (three reversed cyclic tests according to the 2010 AISC *Seismic Provisions* and three monotonic tests) were then conducted to verify the performance with the amended design and detailing procedure.

Although not required by the 2010 AISC *Seismic Provisions*, the monotonic tests were conducted to satisfy FEMA P-795 requirements. The purpose of the monotonic testing is to better understand the collapse behavior of buildings using the connection by investigating the interstory drift capacity afforded by the connection. The results of the cyclic tests again showed that the connection meets the performance requirements of the 2010 AISC *Seismic Provisions* and that the ultimate limit state was as predicted: a net section fracture in the reduced portion of the Links. The results of the monotonic tests showed that the connection has tremendous displacement capacity, the tests being stopped at 9.5% interstory drift without failure or decreasing from peak capacity. The conclusion from the FEMA P-795 study was that the Strong Frame connection is equivalent to the prequalified RBS connection. It should be noted that for all the tests—initial and secondary, cyclic and monotonic—yielding initiated from about 0.01 to 0.015 rad interstory drift angle as is typically expected of frames with SMF connections.

Even with the successful FEMA P-795 evaluation, a second additional requirement was added to look more at system behavior rather than the individual connection behavior as was the focus of the FEMA P-795 evaluation. To address this additional requirement, a series of nonlinear response history analyses were performed using a suite of ground motions and a suite of archetype buildings to compare the seismic response of buildings using the Strong Frame connection to buildings using a prequalified connection. The connection chosen for comparison was again the RBS connection. As before, an independent study was commissioned, with designs for both systems minimized to the extent allowed by the respective design procedures. The study included the development of archetype designs for representative steel moment frames for a two-story, four-story, and six-story building using ASCE/SEI 7-10 (ASCE, 2010), the 2010 AISC *Specification* (AISC, 2010b), and the 2010 AISC *Seismic Provisions* and was evaluated using seven scaled ground motion pairs. The study demonstrated that the performances of the Strong Frame and comparable RBS structures were very similar and within acceptable levels. No collapses were predicted by the analysis. The most severe response was recorded for the two-story RBS archetype, which exhibited a maximum story drift ratio for one record of nearly 5%. Mean story drift response for both the Strong Frame and RBS structures averaged approximately 2.3%, and the mean plus one standard deviation response averaged 3% for the Strong Frame structures and 2.8% for the RBS structures.

In addition to the cyclic and monotonic testing specifically used to qualify the Strong Frame connection, other large-scale shake table test programs have employed the connection. Steel frames using the Strong-Frame connection were part of the 2009

NEESWood Capstone tests at Japan's E-Defense facility in Miki, Japan (van de Lindt et al., 2009). The full-scale seven-story structure consisted of first-story steel framing using the Strong Frame connection, which supported a six-story, wood, light-frame structure on top and had a plan dimension of 40 ft by 60 ft (12 m by 18 m). More recently, steel frames using the Strong Frame connection were employed as retrofit elements in the first story of a four-story, full-scale, light-frame, wood building built to simulate a typical San Francisco-style wood structure with a soft/weak first story due to ground-level parking. Known as the NEESSoft project (Bahmani et al., 2016; Pryor et al., 2014; van de Lindt et al., 2016), the building was successfully tested at the NHERI at San Diego (formerly NEES at UCSD) outdoor shake table under a variety of different ground motions.

In 2015, six additional cyclic tests (three each of two configurations) were conducted on small frames employing shallow beams and the end-plate Yield-Link connection, where the single-plate shear connection and the yielding stem of the Yield-Link are all welded to a common end plate, which is then bolted to the column flange. As before, single-story, single-bay frames were tested. Column bases were hinged, and both beam-to-column connections were moment resisting. The smaller of the two frame specimens consisted of 6-in.- (150-mm-) deep welded H-section columns and an 8.5-in.- (220-mm-) deep welded H-section beam in a frame approximately 8 ft (2.4 m) tall by 9.5 ft (2.9 m) wide (to member centerlines). The larger of the frame specimens similarly consisted of 18-in.- (460-mm-) deep welded columns and an 8.5-in.- (220-mm-) deep welded beam in a frame approximately 8 ft (2.4 m) tall by 12 ft (3.7 m) wide. As before, in-plane lateral load was delivered through a wood nailer connected to the top flange of the beam, and the bottom flange remained unbraced. Bolts connecting the yielding stem of the Link to the beam were pretensioned, whereas the remaining bolts at the beam web-to-shear plate, the end plate-to-column flange, and the BRP bolts were all installed snug-tight. There were no observed failures in the smaller frame, and testing was stopped after two cycles of 0.08 rad interstory drift angle. For the larger frame, failure occurred in the Links, with fracture of the yielding area of the Link occurring in the first cycle of 0.08 rad interstory drift angle for one test and in the second cycle of 0.08 rad for the other two tests. All tests met the connection performance requirements in AISC *Seismic Provisions* Section E3.6b. For the smaller frames, yielding initiated at approximately 0.0125 rad interstory drift angle, whereas it was approximately 0.01 rad for the larger frame.

In 2017, three additional cyclic tests and one monotonic test were conducted to support an expansion of both Yield-Link and member sizes. As before, single-story, single-bay frames were tested with one beam-to-column connection designed as moment resisting using T-stub Yield-Links, with the other beam-to-column connection and both column base connections designed as pins. In-plane lateral load was again delivered to the frame using a wood nailer connected to the top flange of the beam, and also as before, the bottom flange was unbraced. Columns were braced out of plane at the cap plate (slightly above the level of the top flange of the beam) and the column base. Column sections were W36×210 (W920×313), and the beam was W36×170 (W920×253). The center-to-center distance between the two columns

was 35 ft (11 m), and the beam centerline was 21.5 ft (6.6 m) above the column base hinges. T-stub Yield-Links were designed for the new maximum size consisting of 1-in.- (25-mm-) thick stem material, with a 6-in.- (150-mm-) wide yield width in the reduced section, which for comparison corresponds to the remaining flange area of a 50% RBS cut on a W36×160 (W920×238). As expected, fracture of the yielding section of the Link was the failure mode in all three cyclic tests. Two of the test specimens completed one cycle of 0.05 rad interstory drift prior to failure, and one completed two cycles of 0.05 rad interstory drift before failure. No failures occurred in the monotonic test, and it was stopped at approximately 0.09 rad interstory drift due to actuator limitations. For both cyclic and monotonic tests, initiation of yielding occurred at approximately 0.01 rad interstory drift angle. The overall tested assembly is shown in Figure C-12.1, and a plot of moment at the face of the column versus story drift is shown in Figure C-12.2.

In accordance with the design procedure, the length of the yielding section of the Links must increase with increasing beam depth. This leads to the need for more than one bolt on each side of the yielding area of the Link securing the BRP to the beam flange, and thicker Links mean more stress on the BRP, their bolts, and the beam flange. Whereas previously the BRP components were based on the empirical results of early testing, the buckling restraint assembly for the W36 (W920) tests previously discussed was designed according to a new set of requirements that add specific design provisions to the empirical minimum requirements.

The inclusion of beams and columns of up to W36 (W920) profiles and Yield-Links of up to 6 in.² yielding area opened up the connection to use in design space that had



Fig. C-12.1. Testing new largest Link size with 6 in.² yield area.

not been studied, specifically taller buildings. In accordance with the 2010 AISC *Seismic Provisions*, Section E3.6b(b), additional analysis work complying with ASCE/SEI 7-16, Section 12.2.1.1 (ASCE, 2016), was requested by the CPRP committee. To meet this request, a FEMA P-695 (FEMA, 2009) study was conducted to compare the performance of a steel SMF using both RBS connections and Strong-Frame connections. The basis of this comparison came from a previous FEMA P-695 study on 4-, 8-, 12-, and 20-story steel SMF using RBS connections and documented in a report by NIST (NIST, 2010). The NIST RBS frames were reanalyzed to verify the new model was achieving similar results, and then the frames were redesigned for use with the Strong-Frame connection and evaluated using FEMA P-695 procedures. The results showed that for all building heights the buildings with the Strong-Frame connection had better resistance to collapse, and the calculated overstrength ranged from 3.00 to 3.63 for the Strong Frame buildings. Table C-12.1 shows the comparison between the computed collapse margin ratios (CMR) and adjusted collapse margin ratios (ACMR) for both systems. Additional information on the large Link testing, buckling restraint design procedure verification, and FEMA P-695 analysis results is given in Pryor et al. (2018).

12.2. SYSTEMS

Requirements regarding separation of concrete slabs and the column/connection have been simplified by referencing Section 2.3.4. In addition, measures should be taken to prevent concrete from entering the space on either side of the spacer plates adjacent to the yielding core of the Yield-Link connection.

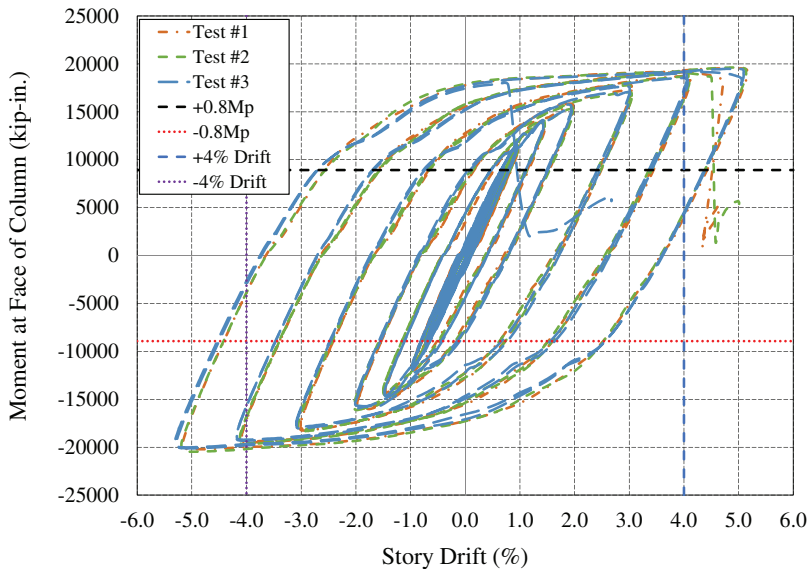


Fig. C-12.2. Moment at face of column versus story drift.

TABLE C-12.1
Incremental Dynamic Analysis Results

Building	SMF	Incremental Dynamic Analyses		
		S_{MT} (g)	CMR	ACMR
4-story	RBS	0.94	1.80	2.52
	Strong Frame	0.94	1.83	2.58
8-story	RBS	0.55	1.39	1.87
	Strong Frame	0.55	2.55	4.08
12-story	RBS	0.40	1.41	1.84
	Strong Frame	0.40	2.63	4.18
20-story	RBS	0.08	1.95	2.23
	Strong Frame	0.08	5.53	8.81

12.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A number of different beam sizes were used in the frame tests, with the largest being W36 (W920) profiles and the smallest being W8 (W200) profiles. End-plate Yield-Links are qualified for use on smaller beams in the W8 (W200) to W12 (W310) range (or equivalent built-up sections). Because the capacity-based design procedure forces inelastic behavior into the connection rather than the beam, in general, the width-to-thickness requirements of the 2022 AISC *Specification* (AISC, 2022b) apply. However, because the connection does rely on the beam flange and web to form part of the buckling restraint assembly for the yielding portion of the Link, the beam flange thickness is required to be checked according to the requirements in Section 12.9, Step 10, and also have a minimum thickness of 0.40 in. (10 mm). Furthermore, the width-to-thickness value cannot exceed λ_r in AISC *Specification* Table B4.1a. Additionally, the capacity-based design procedure and connection performance (no plastic hinging in the beam) allows the beam stability bracing to be designed in accordance with the AISC *Specification*. The protected zone encompasses the shear connection and yielding portions of the connection, specifically the Yield-Links, and elements of the connection in contact with both.

2. Column Limitations

A number of different column sizes were used in the frame tests, with the largest being W36 (W920) profiles and the smallest being W6 (W150) profiles. Because only strong-axis connections were tested, beams are required to connect to column flanges. Where frames are detailed to create plastic hinging at the column base, the width-to-thickness requirements for highly ductile members apply in the first story.

Otherwise, the requirements of the AISC *Specification* apply. Column lateral bracing requirements in the 2022 AISC *Seismic Provisions* (AISC, 2022a) are to be satisfied. An exception is provided to allow bracing the column at the level of the top flange of the beam only if additional limits are placed on the column flexural design strength provisions of the AISC *Specification* to ensure the columns remain elastic outside the panel zones. The limits are noted in Step 13.2 of the Section 12.9 Design Procedure requirements.

3. Bolting Limitations

The connection testing specifically prequalified a number of bolts in the connection to be installed snug-tight. These include the Link flange-to-column flange bolts, end plate-to-column flange bolts, and the shear-plate bolts. These same bolts may also be installed pretensioned if desired. The buckling restraint plate bolts are required to be installed only snug-tight. The Link stem-to-beam bolts are required to be installed pretensioned to prevent slip that would occur under design loads. In the prequalification testing, slip would typically start between 2 to 3% interstory drift, at which point the bolts went into bearing. No special preparation of either the Links or the beam flange surfaces in the test frames was done. The only prequalification requirement is that faying surfaces not be painted.

12.4. COLUMN-BEAM RELATIONSHIP LIMITATIONS

The requirements for the Strong Frame connection are similar to those of other prequalified SMF connections. M_{pr} , however, is calculated based on the probable maximum tensile strength of the Links, $M_{pr} = P_{r-link}(d+t_{stem})$, where P_{r-link} is the probable maximum tensile strength of the Link calculated as the product of the yield area, the specified minimum tensile strength, F_u , and the ratio of the expected tensile strength to the specified minimum tensile strength, R_t . When Links are fabricated from ASTM A572/A572M Grade 50 (Grade 345) plate material, this approach results in a 23% higher estimate of demand than what would be calculated if an approach equivalent to that of other SMF connections was used (Equation 2.4-1). Basing connection demand on the section properties and the expected tensile strength is used in numerous places in the design procedure and produces similarly higher demands when compared to typical SMF requirements. This is consistent with the overall goal of keeping nearly all inelastic demand in the replaceable Yield-Link elements and creating little if any inelastic demand in the members. Using this higher demand also applies to the evaluation of panel zone strength, which for the Strong Frame connection is done in accordance with the AISC *Specification* rather than the AISC *Seismic Provisions*. One effect of this requirement is the use of the AISC *Specification* $\phi = 0.90$ rather than the AISC *Seismic Provisions* $\phi_v = 1.00$ (AISC *Seismic Provisions* Section E3.6e), in conjunction with nominal resistance, R_n , calculated in accordance with AISC *Specification* Section J10.6. Adding to the differences in how M_{pr} is calculated results in panel zone shear demands approximately 26% higher than would be calculated if typical SMF design methodologies were used.

12.5. CONTINUITY PLATES

The need for continuity plates is determined in the design procedure by basing demand on the expected tensile strength of the Links as discussed in Commentary Section 12.4 and design strength as determined by the AISC *Specification*. As was used successfully in the qualification testing, fillet welds are permitted at the web and flanges of the column.

12.6. YIELD-LINK FLANGE-TO-STEM WELD LIMITATIONS

As discussed previously, initially the design demand for this weld was based on the expected tensile strength of the reduced portion of the Link. While this did permit the qualification testing to successfully meet the performance requirements of the AISC *Seismic Provisions*, the ultimate limit state for some of the tests was failure of this weld rather than the more desirable failure in the yielding area of the Link. As a result of the additional requirement to pass the FEMA P-795 component equivalency evaluation, which compared the Strong Frame connection performance to that of an RBS connection, this weld was changed to require either CJP groove welds or double-sided fillet welds that develop the tensile strength of the unreduced portion of the Link.

12.7. FABRICATION OF YIELD-LINK CUTS

The fabrication requirements reflect production quality necessary to ensure the proper performance of the links.

12.8. CONNECTION DETAILING

The requirements of this section reflect the tested conditions and common allowances where appropriate. The connection is detailed to accommodate up to 0.07 rad of rotation, which, along with frame flexibility, will accommodate the expected interstory drift without affecting any connection element other than the Yield-Links. Shear plate connection welds are required to develop the strength of the shear plate, and Yield-Link material thickness may vary from nominally ½ in. (13 mm) to 1 in. (25 mm) thick and is fabricated from one of the three permitted steel grades. The previous specification of Link thickness tolerance was changed to reference ASTM A6/A6M.

The stems of the pair of Yield-Links at each connection must be fabricated from the same heat of material to ensure minimum variability in actual F_y and F_u for the pair of Links in a connection. This is because imbalance of the Link strengths can drive additional force into the central pivot bolt of the connection. This force is parallel to and can be cumulative with the net axial connection force in the beam, which is also resisted by the central pivot bolt. Rather than include an explicit design procedure to accommodate unbalanced Link strength, it was decided at this time to simply use material from the same heat for the stems of each pair of Links at a given connection.

In general, the topic of the potential adverse effects of unequal strength in the Links or flanges of a moment connection is not limited to just the Strong Frame connection. While the central pivot design of the Strong Frame connection in essence attempts to

maintain the location of the plastic neutral axis at the centerline of the beam even if the Links are of different strengths—and thus create relatively even strain demands in each link for a given connection rotation—the same is not true for traditional built-up shapes that may have different flange strengths and form plastic hinges in the beam cross section. The neutral axis of the plastic section would shift toward the flange with higher strength, and uneven strain demands in the flanges would result. However, the effect on inelastic performance for this condition has not been studied, and currently there are no requirements to control flange strength in SMF connections using built-up sections subject to plastic hinging.

When using separate T-stub Yield-Links, beam shear is transmitted directly to the column through the single-plate shear connections and not through the Yield-Link, which affords the opportunity to use other than standard holes at the Link flange-to-column connection. When end-plate Yield-Links are used, shear is transferred from the beam to the column through the bolts that connect the end plate to the column. Because these bolts may be snug-tight, standard holes must be used in both the column and the end plate. Specific methods of fabricating bolt holes and the specification of surface roughness were removed because any approved method is acceptable, and surface roughness must already conform to the *AISC Specification*.

Changes to the requirements for the buckling restraint assembly reflect the addition of specific design requirements in Step 10 of the design procedure. The maximum BRP bolt diameter has also been removed because the bolts are now specifically designed in Step 10.

12.9. DESIGN PROCEDURE

The design procedure for the Strong Frame connection parallels the design concepts for frames with other moment connections but is adapted to the specific configuration of the connection. Connection flexural strength is controlled by the strength of the Yield-Links, and shear strength is controlled by the strength of the shear-plate connection. This allows beams to be designed, if desired, to be unbraced yet stable under the combined effects of expected ultimate connection flexural strength, gravity loads, and axial load resulting from lateral loading. Unlike some historical PR moment connections, the Strong Frame connection is proportioned to remain elastic under the combined effects of design lateral and vertical loads, with the Yield-Links only experiencing inelastic behavior during seismic events in which the real seismic forces are expected to exceed the unamplified design seismic forces (Rex and Goverdhan, 2000). This permits the use of typical elastic analysis procedures similar to other SMF connections. However, like some historical PR moment connections, the beams are designed as simple span for gravity loads (Geschwindner and Disque, 2005). This facilitates post-earthquake repairs, should they be needed, by ensuring the beam is proportioned to support its design gravity loads even if the Links are removed during replacement. In addition to the various strength checks for frame members and elements of the connection, the PR nature of the Strong Frame connection requires a detailed stiffness check using actual connection stiffness to ensure lateral drift limits are met. This means that the lateral stiffness-to-mass and lateral

yield strength-to-mass ratios are required to be the same as any other frame using SMF connections. As such, the code equations for base shear and period estimation are equally applicable to frames using the Strong Frame connection as they are to frames using other SMF connections. This was verified as part of the nonlinear response history study comparing Strong Frame and RBS connections discussed previously. For each of the archetype structures, the periods of the RBS frames and Strong Frame frames were virtually identical.

The design process can be iterative, and **Step 1** begins with suggestions on how to create trial values for sizes of the frame members and provides an initial estimate of story drift which is explicitly checked later in the design procedure. In addition to designing the beam as simply supported, **Step 2** also suggests a deflection limit on the beam to limit member end rotations that would affect the connection.

Step 5 determines the width of the yielding portion of the Link based on the permitted thicknesses ranging from ½ in. (13 mm) to 1 in. (25 mm) and subject to limitations that include a maximum width of 6 in. (150 mm), which corresponds to the strongest Yield-Link that has been qualified. Testing showed that for the approved steel grades, if the length of the straight portion of the yielding section of the Link is proportioned such that the strain demand in that section does not exceed 0.085 when the connection is subjected to a rotation of 0.05 rad, the Link will possess sufficient toughness to enable the connection to meet the cyclic test performance requirements of the AISC *Seismic Provisions*, and this is reflected in **Step 6**.

In **Step 7**, the Link expected yield strength and probable maximum tensile strength are computed. The value of R_t is specified as 1.2 to reflect the proper value from the 2016 AISC *Seismic Provisions* Table A3.1 for ASTM A572/A572M Grade 50 (Grade 345) plates, strips, and sheets. If the Link is fabricated from hot-rolled structural shapes of ASTM A992/A992M or ASTM A913/A913M Grade 50 (Grade 345) as permitted, the tabulated value of $R_t = 1.1$ is used.

In **Step 8**, the Link-to-beam flange connection is designed. Both here and in the web shear-plate connection, bolt bearing is required to be designed using bearing values that limit deformation at the bolt hole. The purpose of this is to again drive the inelastic response into the reduced portion of the Link and to keep other areas of the connection outside of the link essentially damage free to facilitate Link replacement should it be desired after a seismic event.

In **Step 9**, the Yield-Link connection to the column flange is designed. **Step 9.1** determines bolt tension demand from either T-stub Yield-Links or end-plate Yield-Links, as appropriate, for design of the bolts. While the end-plate Yield-Link is similar to the prequalified 4-bolt extended, unstiffened end plate (4E), the yield line mechanism in the end-plate Yield-Link is slightly different. Conservatively, the T-section two-yield line model reflected in Equation 12.9-10 is used to design the end-plate Yield-Link. In Equation 12.9-9, the second term is additional tensile force assigned to the two bolts directly adjacent to the single-plate shear connection due to the moment that results from the eccentric application of shear to the single-plate shear connection. Shear

transfer from the end-plate Yield-Link to the column flange is accomplished through the bolts at the compression flange location of the end plate, which are checked in **Step 9.1a**. In **Step 9.2**, the required Yield-Link flange thickness, for a no prying action condition with a force limited by the probable maximum tensile strength of the Link as reflected in the calculation of r_t in **Step 9.1**, is determined.

Step 10 has been expanded to include specific design provisions for checking the elements of the buckling restraint assembly, which include BRP thickness, beam flange thickness, and BRP bolts. **Step 10.1** determines the required minimum thickness of the BRP. L_{cant} is the lever arm from the start of the reduced region of the Yield-Link to the edge of the first spacer plate bolt hole, plus plate stretch length due to 0.05 rad of joint rotation. It represents the bending leverage on the BRP from the angled reinsertion contact of a link after a large tension deformation has stretched the link (0.05 rad joint rotation) and then the link begins to be reinserted under compression. This was shown through detailed nonlinear finite element analysis (FEA) to be the controlling action for determining BRP thickness. In **Step 10.2**, the beam flange is checked for prying action from BRP bolt forces developed during full compression insertion into the buckling restraint assembly. Equation 12.9-14 is the same as the AISC *Steel Construction Manual* (AISC, 2017) prying equation (Equation 9-17a). The derivation of T_{ux} is based on inelastic column buckling theory using the tangent modulus of elasticity. In comparison to FEA analysis, results predicted the yield lines from Dowsell (2011) provided the best correlation with the design procedure yielding conservative results. In **Step 10.3**, bolt size is determined from consideration of tension induced from constrained weak-axis buckling plus shear due to friction associated with the constrained weak-axis buckling, or from shear determined from in-plane buckling due to strong-axis buckling only.

Step 11 is a procedure for calculating the actual connection stiffness for use in checking frame drift and connection behavior. The Link stiffness is calculated as three springs in series, where the springs represent the stiffnesses of the Link flange in bending, the yielding portion of the link stem under axial load, and the nonyielding portion of the Link stem under axial load. Once the axial stiffness of the Links is computed, the connection can either be modeled with appropriate geometry using discrete axial elements to represent the top and bottom links at a connection, or an equivalent rotational spring can be determined and used in the modeling. As seen in Figure C-12.1, this approach has been shown to be very effective for modeling both the elastic and inelastic behavior of the connection (Pryor and Murray, 2013).

Step 11.2 requires that the frame, using the actual Strong Frame connection properties, meets the required drift limit and that the connection response is elastic under design load combinations (not including amplified seismic load combinations). The calculation of required shear in **Step 12** is analogous to that used in designing RBS connections. Because a plastic hinge is not formed in the beam in Strong Frame connections, the value of L_h is the distances between the rotational points in the shear-plate connections rather than between the centers of plastic hinges. See Chapter 5 Commentary for additional information.

Required member checks are in **Step 13**. **Step 13.1** requires the beams to be checked using the 2022 AISC *Specification* under combined demand that consists of the maximum probable end moments, axial forces considering either the maximum that the system can deliver, or amplified seismic loads and gravity loads. If the designer chooses, beam size can be selected to meet the requirements of the AISC *Specification* under this combined loading without lateral bracing. In **Step 13.2**, column design demand is determined from load combinations that include seismic effects derived from either the maximum that the system can deliver or the overstrength seismic loads for both axial force and moment [the exception in AISC *Seismic Provisions* Section D1.4a(b) allowing one to ignore overstrength level moments when checking the column is not permitted for designs using the Strong Frame connection]. The design strength of the column outside the panel zone is not permitted to exceed $\phi_b F_y S_x$, where $\phi_b = 0.90$ even if otherwise permitted by AISC *Specification* Section F2 when column bracing is only provided at the level of the top flange of the beam.

In **Step 15**, the shear plate and beam web are designed in accordance with the AISC *Specification* to permit hinging about a central rotation point while resisting the beam shear and axial force demand determined from capacity-based design principles. In **Step 15.1**, note that the bolt shear demand is controlled by the shear force on the central bolt in the connection because it takes its portion of the vertical shear reaction in combination with its portion of the axial loads being transferred from the beam to the column, combined using the square root of the sum of the squares (vector sum) rule.

Analogous to a beam flange force, in **Step 16** the maximum probable axial strength of the Yield-Link is used to calculate panel-zone shear demand. As is the case for typical connections, Link strengths are summed for double-sided connections.

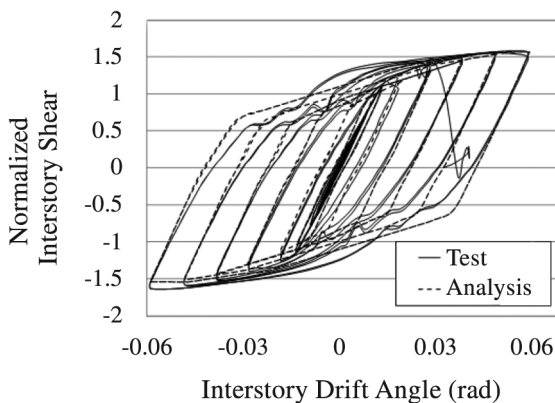


Fig. C-12.3 Testing versus FEA analysis for frame modeled with all material nonlinearity in the Yield-Link elements.

Borrowing from the bolted unstiffened and stiffened extended end-plate moment connection provisions in Chapter 6, **Step 18** provides an analogous design procedure for checking the column flanges for flexural yielding based on the maximum probable tensile strength of the Yield-Link.

If the design strength of the column web or flange without continuity plates or stiffeners is insufficient to support the maximum probable tensile strength of the Yield-Links, the design requirements for the stiffeners or continuity plates are in **Step 19**. Fillet welds are permitted at both column web and flange connections to the continuity plates or stiffeners.

CHAPTER 13

DOUBLE-TEE MOMENT CONNECTION

13.1. GENERAL

The double-tee provisions were written primarily based on testing performed at the Georgia Institute of Technology. The series of six connection tests used a W14×145 (W360×216) column and either a W21×44 (W530×66) or W24×55 (W610×82) beam (Swanson, 1999). The tests are summarized in Table C-13.1. This series of six full-scale assembly tests was supplemented by a series of 48 tests on T-stub components conducted at the Georgia Institute of Technology (Swanson, 1999; Swanson and Leon, 2000). None of the tested configurations included a concrete slab, and all of the tested configurations included a single-plate shear connection between the web of the beam and the column flange.

Research work conducted at the University of Wyoming (McManus and Pucket, 2010) was also considered in development of these double-tee provisions. Testing associated with this program included 22 T-stub component tests with very wide gages between the two rows of bolts connecting the T-flange to the column flange and T-stubs with very thick shims between the T-flange and column flange. These shims were arranged to provide a gap between the T-flange and column flange such that inelastic bending of the T-flange could occur not only when the T-stub was subjected to tension, but also when it was subjected to compression.

Research conducted at the University of Cincinnati on T-stubs built-up from thicker plates was also considered in the development of these provisions (Hantouche et al., 2013; Hantouche et al., 2015). The intent of this work was to allow for the use of built-up T-stub components in the double-tee connections. However, because the testing that was performed at the University of Cincinnati did not include full-scale assemblies, the CPRP elected to not permit built-up T-stubs in this edition of the provisions.

A series of five full-scale beam-column subassemblies was tested at the University of Texas at Austin in 1996 (Ulloa Barbaran, 1996; Larson, 1996). These assemblies consisted of cantilever W36×150 (W920×223) beams connected to pinned-pinned W14×426 (W360×634) columns using a single-plate shear connection to resist shear and, in most cases, fully bolted T-stubs to resist moment. The first specimen was designed with a shear-only connection—with a single-plate shear connection but without T-stubs—to investigate the contribution of the single-plate shear connection and beam web to the moment strength of the connection. The second and third specimens were designed both with a single-plate shear connection and T-stubs, but the T-stubs were configured to provide only a partial strength beam connection. The fourth and fifth specimens were designed with both a single-plate shear connection and T-stubs with the tees in the fourth specimen proportioned to transmit 100% of

TABLE C-13.1.
Summary of Prequalification Tests on
Double-Tee Connections

Test	Beam	Column	T-Stub	Bolts
FS-03	W21×44	W14×145	W16×45	$\frac{7}{8}$ in. A490
FS-04	W21×44	W14×145	W16×45	1 in. A490
FS-05	W24×55	W14×145*	W16×100	$\frac{7}{8}$ in. A490
FS-06	W24×55	W14×145*	W16×100	1 in. A490
FS-07	W24×55	W14×145*	W21×93	$\frac{7}{8}$ in. A490
FS-08	W24×55	W14×145*	W21×93	1 in. A490

* Column was stiffened with continuity and doubler plates.

the beam moment to the column and the tees in the fifth specimen proportioned to transmit approximately 125% of the beam's plastic moment (Ulloa Barbaran, 1996). The connections were loaded by applying a displacement to the end of the cantilever beam. The fourth and fifth specimens are most germane to the topic of prequalification of double-tee connections. The fourth specimen failed when the bolts connecting the T-stubs to the column flange fractured. After disassembling the connection, small fractures near the bolt holes in the flange of the beam were noted. Testing of the fifth specimen was stopped when a fracture of the beam flange was noticed. None of the specimens performed adequately to be considered for use in SMF or IMF systems.

Research on bolted flange plate (BFP) connections conducted at the University of Illinois (Schneider and Teeraparbwong, 1999) and at the University of California at San Diego (Sato et al., 2008) was also considered because the connection of the flange plate to the beam flange in a BFP connection is nearly identical to the connection of the T-stem to the beam flange in a double-tee connection.

13.2. SYSTEMS

None of the tested configurations included a concrete slab; thus, the slab must be isolated from the column to avoid developing unintended and unproven mechanism in the connection as built during a seismic event. Further, no shear studs were present in the tested configuration; therefore, as a result, shear studs are precluded in the as-built connections in a conservative but consistent decision aimed at preventing potential fractures from initiating at the stud welds in regions expected to undergo significant inelastic strains. Given that the distance from the face of the column to one beam depth beyond the shear bolts farthest from the column may be a large distance, the requirement that studs be omitted in this region may create difficulties for composite beam systems. However, the requirement is felt necessary to preserve the anticipated performance of the connection. It is speculated that the presence of the bolts and nuts connecting the beam flange to the T-stem (the shear bolts) will transfer some shear between the steel and concrete without compromising the seismic performance of the connection.

Double-tee connections tested at Georgia Tech ranged in stiffness from connections that were clearly fully restrained (FR) to nearly partially restrained (PR). The AISC *Seismic Provisions* explicitly permit PR connections in OMFs but are silent on the use of PR connections in IMFs and SMFs. This standard precludes the use of PR double-tee connections until the issue can be considered in more detail. **Step 13** of the double-tee design procedure includes a check of the connection stiffness to ensure that it satisfies the FR criteria.

13.3. PREQUALIFICATION LIMITS

Prequalification limits for the beam are based on the configurations successfully tested at Georgia Tech. Limitations for the column are based on limitation for other prequalified connection types.

13.4 CONNECTION DETAILING

1. T-Stub Material Specifications

T-stubs tested at Georgia Tech were cut from W-shapes rolled from either ASTM A572/A572M Grade 50 (Grade 345) or dual grade ASTM A36/A572 Grade 50 steel. In all cases, the yield and tensile strengths of the steel used for the T-stubs were consistent with ASTM A572/A572M Grade 50 (Grade 345) or ASTM A992/A992M material. Details are available in Swanson (1999).

2. Continuity Plates

All of the experiments upon which these provisions are based were conducted with continuity plates present. As a result, continuity plates are required in the prequalified double-tee connections. The continuity plates are required to control column-flange bending that may result in column-flange prying forces in the tension bolts between the T-flange and column flange. While T-flange prying is considered explicitly in the design procedure, column-flange prying is not. Further, when two rows of four tension bolts are used, it is speculated that the absence of continuity plates may result in column-flange bending that could produce larger forces in the interior tension bolts than the exterior tension bolts. More research is needed before the requirement of including continuity plates can be relaxed.

4. Bolts

All of the tests used as a basis for prequalification employed two identical T-stubs and bolt patterns that were arranged symmetrically with respect to the axes of the beam and column (with the exception of the web bolts, which are slightly off center with respect to the plane of the beam web and column web). Further, all prequalification tests used two rows of bolts between the T-stem and beam flange and between the T-flange and column flange. Significant questions remain regarding whether additional rows of bolts between the T-flange and column flange would participate in carrying load without the presence of stiffeners on the T-stub.

All of the six full-scale assemblies tested at Georgia Tech made use of ASTM F3125/F3125M Grade A490 bolts, but a number of the T-stub component tests included ASTM F3125/F3125M Grade A325 fasteners. As a result, Grade A325, A325M, A490, or A490M bolts, or their tension control equivalents, are permitted in the double-tee connections. While all of the tested configurations used the same diameter and grade of fasteners throughout the connections, there is no requirement that all bolts in the prequalified connection be the same diameter or grade. It is expected that all bolts will likely be Grade A490 or A490M, but there may be advantages to using larger-diameter tension bolts between the T-flange and column flange and smaller-diameter shear bolts between the T-stem and beam flange.

Oversize holes are not permitted in the beam flange, because if they are used, it is difficult to achieve adequate moment capacity in the beam at the row of shear bolts farthest from the column face.

Of the 22 experiments conducted at the University of Wyoming, 18 components failed with a fracture of the flange of the T-stub. Of the 60 T-stub components tested at Georgia Tech (including the 12 T-stubs in the full-scale assembly tests), no fractures in the flanges were observed. An analysis of the 82 experiments revealed that the T-stubs that failed via flange fracture all had a ratio of tension bolt gage to T-flange thickness, g_{tb}/t_{ft} , that was greater than 10. No flange fractures were observed in T-stub components with a ratio of g_{tb}/t_{ft} of less than 9. After considering the data, the CPRP elected to limit the ratio of g_{tb}/t_{ft} to be not greater than 7.0, a value that includes all of the components tested at Georgia Tech.

All bolts are required to be installed as pretensioned fasteners and faying surfaces are required to be prepared as they would be in slip-critical connections. The connections are not intended to be designed as slip critical, however. The required pretension and faying surface preparations are intended to provide a reasonable level of friction to prevent slip at service loads and provide slip resistance conducive to energy dissipation at design loads. Prevention of slip at design loads is not considered desirable as this would limit a robust energy dissipation mechanism of the connection.

5. T-Stub Shims

The thickness of shims between the T-flange and column flange is limited to a maximum of ¼ in. (6 mm) to preclude the use of shims that would be similar in configuration to the shimmed configuration in the University of Wyoming experiments where the T-flange was permitted to flex both when the T-stub was in tension and when it was in compression. Further, the use of shims that extend the full width and breadth of the T-flange is encouraged.

13.5. DESIGN PROCEDURE

Step 2. This step checks that beam flange tensile rupture does not occur before the full plastic moment of the beam is developed at the shear bolts farthest from the column face. The same check is used for the bolted flange plate and cast bolted bracket connections. This check differs from that which appears in AISC *Specification*

Section F13 (AISC, 2022b) which compares the gross yielding strength of the flange to the net section rupture strength of the flange. The check in Step 2 is predicated on the assumption that there are two bolts in standard holes in the tension flange of the beam, and the approach is substantiated by a study led by Swanson (2016) wherein 18 relevant experiments on bolted flange plate and double-tee connections were identified. In 17 of those experiments, the beam was able to develop at least the full plastic moment strength, and in the 18th experiment, the beam reached approximately 97% of the full plastic moment strength before the beam flange fractured. Equation 13.5-1 predicted that the strength of the beams in all 18 experiments would be governed by gross yielding, whereas the check in AISC *Specification* Section F13 predicted that gross yielding would govern in only half of the experiments and that net section fracture would govern in the other half. Based on this study, it was determined that the check in AISC *Specification* Section F13 was overly conservative.

In the event that the check in Step 2 is not satisfied, reinforcement of the beam could be provided. It should be noted, however, that none of the experiments upon which these provisions are based included any reinforcement of the beam flanges at the critical net section. Thus, reinforcing the beam flange is not within the scope of this prequalified connection design procedure.

Step 3. Bolt shear will govern in most cases using typical bolt spacing and end distances. The bearing and tearout strength of the T-stem and beam flange will be checked in **Step 18** after additional dimensions of the connection are determined. A simple calculation can be performed at this stage to determine bolt spacing and end distance that will guarantee that shear of the bolt will govern, which will maximize the strength of the shear bolts.

Step 4. Equation 13.5-4 is used to estimate the number of shear bolts that will be required to connect the T-stem to the beam flange. The quantity $1.25M_{pr}$ in the numerator is in place as a preliminary estimate of the moment at the face of column, M_f , which cannot be determined until the number of shear bolts is known.

Step 5. The location of the plastic hinge relative to the column face is determined in this design step. This is based on spacing and end distance of the shear bolts and the setback distance of the end of the beam relative to the column face, which will be much larger than the $\frac{1}{2}$ in. (13 mm) that is typically used for bolted flange plate connections and simple shear connections to allow space for the T-flanges between the end of the beam and column face.

Step 8. The maximum probable flange force is determined by dividing the moment at the column face, M_f , by the distance between centerlines of the stems of the top and bottom T-stubs. Because the thickness of the T-stems has not yet been determined, this distance is estimated as $1.05d_b$. The expected flange force will be computed using the actual distance between the T-stem in **Step 14** after the actual stem thickness is known.

Step 9. The size of the T-stem is determined based on gross yielding and net fracture in tension, and gross yielding or buckling in compression. The gross section of the

T-stem at the first row of shear bolts will be determined by the width of the T-stub and the thickness of the T-stem. The width of the T-stub will be governed by the spacing of the bolts between the T-flange and column flange. As a general rule of thumb, assume that the T-stub will be slightly narrower than the width of the column flange. The lesser of the actual T-stub width and the Whitmore width is used in calculating section properties. If the T-stem is not tapered, the actual width is simply the T-stub width, W_T . If the T-stem is tapered, then the actual width will depend on the geometry of the taper.

The typical gages for beam sections published in the AISC *Steel Construction Manual* can generally be used for the shear bolts. In some cases, it may be advantageous to use a wider gage as long as edge distance requirements are met, which will increase the Whitmore width of the stem and allow more entering and tightening clearance, but this may lower the block shear strength of the beam flange.

The strength of the T-stem will be backchecked in **Step 16** after the section from which the T-stubs are to be cut is selected in **Step 12**.

Step 10. Equation 13.5-12 provides a lower bound estimate of the required diameter of the tension bolts based on the assumption that there is no prying in the T-flange. Larger bolts may be required depending on the level of prying present in the T-flange and may be desirable to allow a ductile prying mechanism to form in the T-flange (Swanson, 2002).

Step 11. The provisions in this step are based on the prying model developed by Swanson (2002). The strength of the tension bolts and T-flange will be confirmed in **Step 17** after the final T-stub dimensions are determined in **Step 12**. In general, the smallest gage possible for the tension bolts is desirable from a strength point of view; however, larger gages can provide additional ductility in the T-flange at the expense of higher prying forces. The gage of the tension bolts will be governed by the entering and tightening clearances of the tension bolts relative to the beam flange, T-stem, and shear bolts. Care must also be taken to ensure that there is sufficient space between the top and bottom T-stubs for the single-plate shear connection.

The thickness of the T-flange is also a critical parameter and influences the strength of the T-stub greatly. A solution envelope for a typical T-stub is shown in Figure C-13.1 and is the result of plotting the T-stub flange capacity as a function of the flange thickness. The bold line OABC defines the capacity of the flange and tension bolts, and the region below this line represents an adequate design. Segment OA defines the flange mechanism strength and is calculated as ϕT_1 using Equation 13.5-43. Segment AB, computed as ϕT_2 using Equation 13.5-44, is referred to as a mixed-mode failure because yielding of the T-flange and bolt failure are both expected. Segment BC represents the conventional strength of the bolts without prying and is computed as ϕT_3 using Equation 13.5-45. Point A in the chart is generally considered to represent a balanced failure because the full strength of the flange is exhausted at the same time that the bolt forces, including prying, become critical. The flange thickness associated with the point B is often referred to as the critical thickness, t_{fcrit} , because a

T-stub with a flange thickness greater than that will have negligible prying and will develop the full tensile strength of the bolts.

The initial tension bolt diameter determined in **Step 10** using Equation 13.5-12 is based on setting ϕT_3 equal to F_{pr}/n_{tb} and solving for d_{tb} . The initial flange thickness determined in **Step 11** using Equation 13.5-17 corresponds to Segment AB of Figure C-13.1 and is found by setting ϕT_2 equal to F_{pr}/n_{tb} and solving for t_{fi} . The flange thickness associated with a balanced failure, point A in Figure C-13.1, can be found by setting ϕT_1 equal to ϕT_2 and solving for t_{fi} , as is shown in Equation 13.5-21. The strength of flanges thinner than the thickness given in Equation 13.5-21 will be governed by a plastic flange mechanism. The critical flange thickness associated with point B in Figure C-13.1 can be found by setting the flange strength for a mixed-mode failure, ϕT_2 , equal to the flange strength in the absence of prying, ϕT_3 , and solving for t_{fi} , as shown in Equation 13.5-23.

The dimension a is estimated in **Step 11** as shown in Equation 13.5-14. After the final T-stub dimensions are determined in **Step 12**, the T-flange and tension bolt strength is back-checked in **Step 17**, using a different value of a as calculated in Equation 13.5-47. When determining the gage of the tension bolts, g_{tb} , keep in mind entering and tightening clearances for the tension bolts, particularly in the side of the T-stem adjacent to the beam where the flange of the beam can interfere with tightening of the tension bolts. Using a smaller tension bolt gage reduces prying forces in the bolts, but if the gage is too small, it may be very challenging, if not impossible, to pretension the tension bolts.

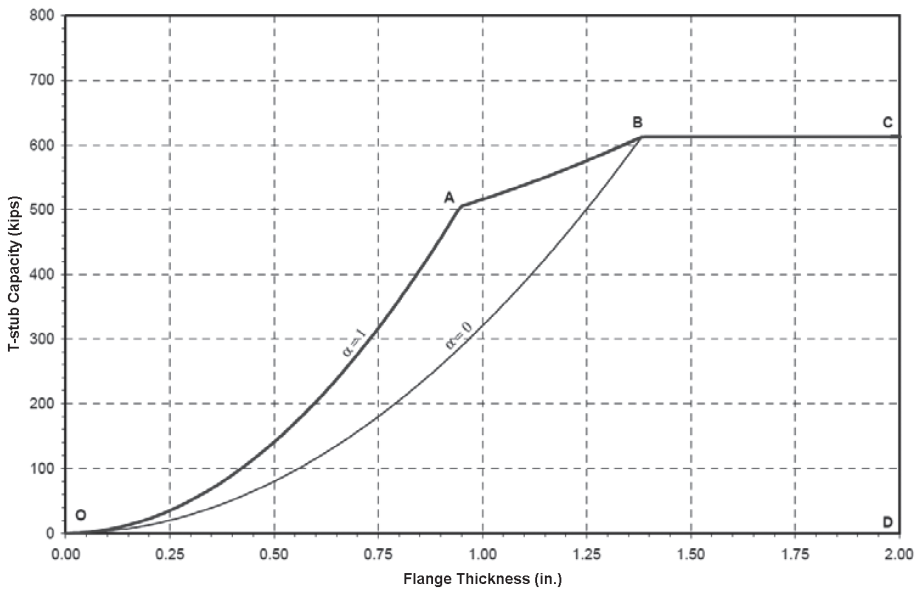


Fig. C-13.1. Relationship of T-stub capacity to flange thickness.

Step 12. Select a W-shape that the T-stubs can be cut from. The depth of the W-shape must be sufficiently large to accommodate the distance from the face of the column to the end of the T-stems, which is S_h plus the end distance of the shear bolts in the T-stem. The web thickness and flange thickness of the W-shape must be large enough to accommodate the values of t_{st} and t_{fl} computed in **Steps 9** and **11**, respectively. Finally, the flange width of the W-shape must be large enough to accommodate the gage of the tension bolts and the required edge distance.

Step 13. The initial stiffness of the connection is computed based on the stiffness model proposed by Swanson (1999) and is compared to a minimum stiffness of $18EI/L$ that is required for a FR connection. It is expected that most connections designed on the basis of developing the full plastic moment of the beam will be stiff enough to qualify as FR. Still, this check is required for the double-tee connection.

Step 14. After the final dimensions of the T-stubs and connection are determined, the expected flange force, F_f , is computed based on actual dimensions instead of estimated dimensions.

Step 15. After the expected flange force, F_f , has been determined and the actual dimensions of the connection are known, the strength of the shear bolts is checked to ensure that they are adequate.

Step 16. After the expected flange force, F_f , has been determined and the actual dimensions of the connection are known, the strength of the T-stem is checked to ensure that it is adequate.

Step 17. After the expected flange force, F_f , has been determined and the actual dimensions of the connection are known, the strength of the T-flange and tension bolts is checked to ensure that they are adequate. Shear and tension interaction in the tension bolts need not be considered. Because the shear plate will undoubtedly be much stiffer with respect to vertical forces than the T-stem, it is safely assumed that the web bolts carry all of the beam shear force to the column while the tension bolts carry only tension resulting from the beam moment. This can be demonstrated by considering the ratio of the stiffness of the shear plate to the stiffness of the T-stems on the top and bottom flanges of the beam.

$$\frac{K_{shear\ plate}}{K_{T-stems}} = \frac{12 L_{sp}^3 t_{sp}}{(2)(12) W_T t_{st}^3}$$

If it is assumed that $L_{sp} \approx W_T$, $t_{st} \approx 2t_{sp}$, and $L_{sp} \approx 20 t_{sp}$, then this ratio reduces to 25 indicating that the shear connection is 25 times stiffer than the flange connections with respect to resisting vertical forces.

Step 18. Bearing and tearout of the shear bolts relative to the T-stem and beam flange is checked consistent with AISC *Specification* Section J3.11 using actual dimensions and the expected flange force, F_f .

Step 19. Block shear of the T-stem and beam flange should be checked in a manner consistent with AISC *Specification* Section J4.3. For the purpose of this design, the block shear failure is considered a ductile failure mode.

The “alternate block shear” mechanism illustrated in Figure 13.7 need not be checked. For this failure mechanism to form, a net rupture of the beam flange must occur, and this is precluded by the check in **Step 2**. In numerous documented tests of double-tee and bolted flange plate connections, this alternative block shear mechanism has not been observed.

Step 21. Because of the large setback required, the shear connection will most likely need to be designed as an “extended” single-plate shear connection. Most importantly, the length of the shear connection, L_{sc} , should be determined so as to fit between the flanges of the T-stubs allowing ample clearance.

Consistent with AISC *Specification* Section J4.3, block shear of the beam web should be checked for the failure path shown in Figure C-13.2. For the purpose of this design, the block shear failure shall be considered a ductile failure mode.

Step 22. The strength of the column flange adjacent to the T-stub is checked in this step. A yield line analysis is the basis for the strength in Equations 13.5-53 through 13.5-59. The yield-line pattern is shown in Figure 13.8, wherein it is assumed that the top T-stub is in tension. With the requirement that both T-stubs be identical, checking only one location is satisfactory.

Note that the yield-line pattern shown in Figure 13.8 is applicable to both the cases of T-stubs with eight tension bolts or four tension bolts. In the case where eight tension bolts are used, the yield pattern is defined by the inner four bolts and the associated gage, g_{ic} , shown in the figure.

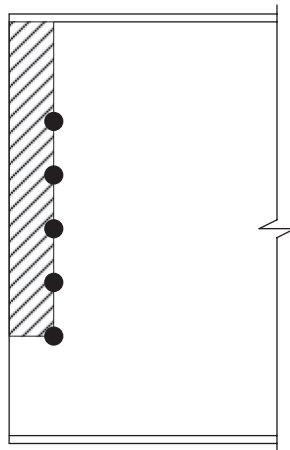


Fig. C-13.2. Block shear failure mode for the beam web.

Step 23. In checking the column for web local yielding and web local crippling consistent with AISC *Specification* Sections J10.2 and J10.3, respectively, the bearing length, l_b , can be taken as $5k_T + t_{st}$, where k_T is the k dimension of the T-stub and t_{st} is the T-stub stem thickness. Given the requirement for continuity plates, however, web local yielding and web local buckling are not expected to control.

CHAPTER 14

SLOTTED WEB (SW) MOMENT CONNECTION

14.1. GENERAL

The slotted web (SW) connection is a welded steel beam-to-steel column connection developed through private funding by Seismic Structural Design Associates Inc. (SSDA). In the SW moment connection, slots in the beam web are made parallel and adjacent to the beam flanges. These slots, which start at the end of the beam and are typically one-third to one-half the nominal beam depth in length, are terminated at a round stress relief hole. The beam web is welded to the column flange and also to the shear plate to give the web both shear and moment capacity.

Analytical studies by Yu (1959) and finite element analyses (FEA) by Abel and Popov (1968) have shown that the shear distribution at the support of cantilever beams differs drastically from that predicted by classical Bernoulli-Euler beam theory that led to the popular design concept wherein “the flanges carry the moment and the web carries the shear.” It was shown that in the case of a rigid support (beam web and flanges welded to a rigid column flange), the entire shear is resisted by the flanges. For typical “flange-welded, web-bolted” connections such as the so-called pre-Northridge connection, however, about 50% of the shear is resisted by the beam flanges. It is this 50% shear component in combination with the tension component that causes severe stress and strain gradients across and through the beam flanges of these connections.

By separating the beam flanges from the web in the region of the connection to the column, essentially all the beam shear is resisted by the beam web and, if the beam web is welded to the column, the web also resists a moment equal to the plastic moment capacity of the web, which is typically 30% of the beam plastic moment. Moreover, the elimination of the beam flange shear results in stress and strain gradients across and through the flanges to be nearly uniform.

Cyclic qualifying tests on the SW connection have been made using the single-cantilever type and bare steel specimens; see test results in Table C-14.1. This pseudo-static test with the loading protocol developed by the FEMA/SAC program (FEMA, 2000b) has been adopted in AISC *Seismic Provisions* Section K2 (AISC, 2022a). These tests, along with the FEA of the SW connection, show that the yielding region is concentrated in the separated portion of the beam flanges and in the beam web at the end of the shear plate. Peak strengths of the test specimens are usually achieved at an interstory drift angle of approximately 0.03 and 0.04 rad. Reduction in strength, if any, is gradual and due to the out-of-plane buckling of both the beam flanges and web. Buckling of the flanges and web occurs concurrently but independently, which eliminates the lateral torsional mode of buckling. Review of the SSDA test data indicates that the SW connection, when designed and constructed in accordance with the limits and procedures presented herein, have developed interstory drift

TABLE C-14.1
SSDA Cyclic Tests and Summary of Results

Test No.	<u>Beam</u> <u>Column</u>	Interstory Drift (%)
17	W33×141	4.2
18	W14×283	5.1
19	W27×94	4.3
20	W14×176	5.0
21	W36×300	4.5
22	W14×500	4.4
23	W24×94	4.1
24	W30×135	4.1
25	W36×170	4.0
26	W30×235	4.0
1a	W36×256 W27×307	4.9
2a	W36×393	5.1
3a	W14×550 (Gr. 65)	6.0

angles of a least 0.04 rad under cyclic loading on a consistent basis. Ultimate failure typically occurs at drift angles of 0.05 to 0.07 rad by low cycle fatigue fracture of the flange near the end of the slot or partial fracture of the beam web/shear plate weldment to the column flange (Richard, et al., 2001; Partridge, et al., 2002).

14.2. SYSTEMS

Review of the design rationale and the test results shown in Table C-14.1 indicates that the SW connection meets the prequalification requirements for special moment frames in AISC *Seismic Provisions* Section K1.

14.3. PREQUALIFICATION LIMITS

1. Beam Limitations

A wide range of beam sizes have been tested by SSDA with the SW connection. The smallest beam tested was a W24×94 (W610×140). The largest was a W36×393 (W920×585). The AISC *Seismic Provisions* permit limited increases in beam weight and depth compared to the maximum sections tested, and there is no evidence that modest deviations from the maximum tested specimen would result in significantly different performance.

Both beam depth and beam span-to-depth ratios are significant in the inelastic behavior of beam-to-column connections. For the same induced curvature, deep beams will

experience greater strains than shallower beams. Similarly, beams with shorter span-to-depth ratios will have a sharper moment gradient across the beam span, resulting in a reduced length of the beam participating in the plastic hinging and increased strains under inelastic rotational demands. The beam-to-column assemblies that were tested by SSDA with the SW connection are given in Table C-14.1, which includes the test interstory drift ratios.

2. Column Limitations

All of the SW tests have been performed with the beam flange welded to the column flange (i.e., strong-axis connections). The column sizes used in the tests ranged from W14 columns to W30 columns.

The behavior of SW connections with cruciform columns and box columns is expected to be similar to that of a rolled wide-flange column because the beam flanges frame into the column flange and the column panel zone is oriented parallel to that of the beam. For cruciform columns, the web of the cut wide-flange column is welded with a CJP groove weld to the continuous web 1 ft (300 mm) above and below the depth of the frame girder. Given these similarities and the lack of evidence suggesting behavior limit states different from those associated with rolled wide-flange shapes, cruciform and box column depths are permitted equal to those for rolled wide-flange column depths.

14.4. BEAM FLANGE-TO-COLUMN FLANGE WELD LIMITATIONS

CJP groove welds joining the beam flanges to the column flanges of the SW test connections were made using E70T-6-H16 electrodes with a minimum specified CVN toughness as specified in the AISC *Seismic Provisions* for demand critical welds. Further, the beam bottom flange backing was removed. The root weld pass was backgouged out and replaced with new weld passes as required. A reinforcing fillet was then added to the bottom flange weld. At the top flange weld, the backing was fillet welded to the column flange. Weld tabs were removed at both the top and bottom flange welds.

14.5. BEAM WEB AND SHEAR-PLATE CONNECTION LIMITATIONS

In all SW test connections, the shear plate was welded directly to the column flange using either a CJP or a PJP weld over the full height of the shear plate. The beam web was welded to the face of the column flange, and the shear plate served as the backing for this weld. Further, an eccentrically loaded weld group consisting of fillet welds was used to join the shear plate to the beam web. These welds were made using E71T-8-H16 electrodes with the minimum CVN toughness specified in the AISC *Seismic Provisions*. Additionally, the shear plate was joined to the beam web with high strength pretensioned bolts.

14.6. FABRICATION OF BEAM WEB SLOTS

The beam web slots in the SW test specimens were flame cut along the “k-line” of the beam to a termination hole that was either drilled or thermally cut. The narrow slot

width over the shear plate is designed to inhibit beam flange buckling near the face of the column (to protect the beam flange-to-column flange weld) and force the major beam flange buckling to occur over the wider part of the slot.

14.7. DESIGN PROCEDURE

The design rationale for the SW connection is based upon:

- (a) The IBC (ICC, 2015), the AISC *Specification* (AISC, 2016b), and the principles of plastic design.
- (b) Results of cyclic qualification tests using beams ranging from W24×94 (W610×140) to W36×393 (W920×585) and columns ranging from W14×176 (W360×262) to W14×550 (W360×818) and W27×307 (W690×457) to W30×235 (W760×350).
- (c) Inelastic finite element analyses to evaluate the stress and strain distributions and buckling modes.

In **Step 5**, the beam slots are designed to:

- (1) Force the beam shear at the connection to be carried predominately by the beam web.
- (2) Provide a nearly uniform stress and strain distribution horizontally across and vertically through the beam flanges from the column face to the end of the beam web slot.
- (3) Allow plastic beam flange and beam web buckling to occur independently in the region of the beam web slot. This eliminates the lateral-torsional mode of buckling found in beams where the beam web is not slotted.
- (4) Ensure plastic beam flange buckling so that the full plastic moment capacity of the beam is developed:

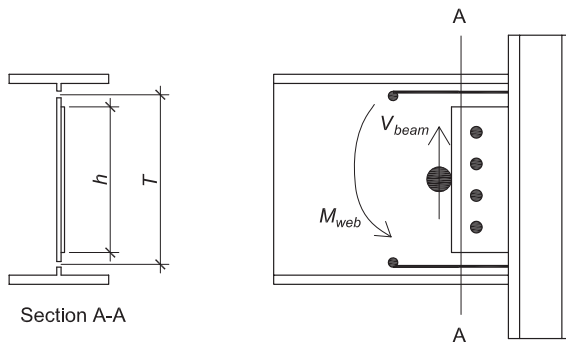
$$\frac{l_s}{l_f} \leq 0.60 \sqrt{\frac{E}{F_y}} \quad (\text{C-14.7-1})$$

In **Step 6**, a maximum nominal height of the shear plate is used that can accommodate the slot and the weld across the top and bottom of the shear plate. The minimum thickness of the shear plate is based upon the moment increase in the connection from the plastic hinge at the end of the shear plate to the face of the column. Observations from the SW tests have shown that a shear plate equal to or greater than two-thirds the beam web thickness should be used to stabilize the beam web and shear plate from out-of-plane bending to protect the web and plate welds at the column flange. To stabilize the beam web at the column flange, use a minimum shear plate thickness of two-thirds of the beam web thickness but not less than 3/8 in. (10 mm).

In **Step 7**, AISC Manual tables may be used to determine the weld size of an eccentrically loaded weld group made from fillet welds for the shear plate based upon the shear plate moment and shear forces as shown in Figure C-14.1.

In **Step 8**, the shear plate-to-column flange weld must exceed the fillet weld strength of the shear plate eccentrically loaded fillet weld group that resists the increase in the connection moment from the plastic hinge at the end of the shear plate to the column flange.

In **Step 9**, the bolts are designed for erection purposes and also to clamp the shear plate to the beam web. The effect of this clamping action minimizes the out-of-plane buckling of the plate and beam web near the column flange weldment.



- The centroid of the SW connection plastic moment is located at the end of the shear plate.
- At Section A-A, the beam web moment and shear are resisted by the beam web and the shear plate.
- Distribute these forces between the shear plate and the beam web based on the strength distribution.
- $V_{plate} = \left(\frac{t_p}{t_p + t_w} \right) V_{beam}$, $M_{plate} = \left(\frac{t_p}{t_p + t_w} \right) \left(\frac{h}{T} \right) M_{web}$
- Use V_{plate} and M_{plate} to design the shear plate to the beam web weld.

Fig. C-14.1. Beam web—shear-plate force distribution.

CHAPTER 15

DURAFUSE FRAMES MOMENT CONNECTION

15.1 GENERAL

The DuraFuse Frames connection is a field-bolted connection with a replaceable fuse plate below the beam bottom flange. The fundamental seismic behaviors expected with the DuraFuse Frames connection include:

- (1) Shear yielding in particular regions of the replaceable fuse plate, followed by strain hardening at larger deformations.
- (2) Slip of the fuse plate bolts and top plate bolts, which occurs at a similar resistance level to the initial yielding.

The combination of bolt slip and fuse plate yielding results in large inelastic deformation capacity for the DuraFuse Frames connection while preventing yielding in the beam and column.

Prequalification of the DuraFuse Frames connection is based on full-scale tests performed in accordance with AISC *Seismic Provisions* Chapter K (AISC, 2016a). Ten qualifying tests were performed on connections with beams ranging in depth from W21 to W40 at the University of California, San Diego (Reynolds and Uang, 2018; Reynolds and Uang, 2019). The deformation capacity varied depending on the depth of the beams, and ranged from 0.04 rad to 0.08 rad, with shallower beams having greater capacity. I-shaped and box columns were included in the testing program. Eight other full-scale qualifying tests are reported in Richards and Oh (2018). For reference, reduced scale prototype tests are reported in Richards and Oh (2019).

The stiffness of the DuraFuse Frames connection is sufficient to be considered fully restrained (FR). The stiffness of experiments and finite element models have been compared with the stiffness of 2D centerline models where the flexibility of the panel zone is explicitly considered and the connection is replaced by a rotational spring with a stiffness of $18EI/L$. The experimental and FE stiffness have been equal to or greater than that of the centerline models, justifying the classification as a fully rigid (FR) connection.

15.2 SYSTEMS

The DuraFuse Frames connection meets the qualification requirements for both SMF and IMF frames. However, no test data are available for the DuraFuse Frames connection with composite slabs, so prequalification is restricted to the case where the concrete slab has a minimum separation or isolation from the column. In general, isolation is achieved if the slab is separated from the column face by an open gap or by use of compressible material.

15.3 PREQUALIFICATION LIMITS

1. Beam Limitations

The DuraFuse Frames connection has been investigated with a variety of beam sizes. The specimens with the smallest beam size [W14×38 (W360×64)] were early prototype tests and had rectangular fuse plates. The smallest beam size that has been tested with fuse plates with extensions, similar to Figure 15-1, is a W21×50 (W530×74). On the bigger side, more than 10 tests have been performed with W33 (W840), W36 (W920), and W40 (W1000) beams. The deepest beam that has been tested is a W40×167 (W1000×249). The heaviest beam that has been tested is a W36×232 (W920×345).

Although the AISC *Seismic Provisions* permit limited increase in beam depth compared to the maximum sections tested, the prequalification limit for maximum beam depth was not extrapolated beyond the test data for a W40×167 (W1000×249) to be consistent with other connections in this standard. The strain demands on the fuse plate are directly related to the beam depth, so it is conservative to base the beam depth prequalification limit on the maximum tested specimen.

The prequalification limit for beam weight, 309 lb/ft (460 kg/m), was established based on the weight increase permitted in the AISC *Seismic Provisions* (1.33 times the heaviest beam tested). Connections with beams that have thicker flanges also have thicker fuse plates and external continuity plates. Experiments have not shown negative consequences of thicker plates.

Because there is no beam yielding in the DuraFuse Frames connection, it is not necessary for the beam to satisfy the width-to-thickness ratios given in the AISC *Seismic Provisions*. Several tests have been performed with beams with higher width-to-thickness ratios than λ_{hd} . The greatest flange width-to-thickness ratio that has been tested is 7.80 [W30×99 (W760×147)]. The greatest web width-to-thickness ratio that has been tested is 51.9 [W30×99 and W36×150 (W760×147 and W920×223)]. The requirement that the beam flange width-to-thickness ratio should not exceed λ_p is deemed conservative because no plastic hinge formation is expected in the beam.

All experiments for the DuraFuse Frames connections have demonstrated good performance without the presence of lateral bracing near the beam-column connection. The only beam restraint in the tests was at the beam tip. The DuraFuse Frames connection is much less susceptible to beam lateral-torsional buckling or column twisting because the beam does not yield and there is no local buckling in the fuse plate to disrupt symmetry of the load path at the connection. Thus, the beam stability bracing can be determined using the AISC *Specification*, neglecting the prescriptive requirements for SMF that assume beam yielding.

2. Column Limitations

The DuraFuse Frames connection has been investigated with a variety of column shapes and sizes. The W14 (W360) columns that have been tested are W14×38 (W360×57.8) and W14×68 (W360×101). The W21 (W530) column that has been

tested is W21×132 (W530×196). The W24 (W610) columns that have been tested are W24×103 (W610×153) and W24×250 (W610×372). The W36 (W920) columns that have been tested are W36×150 (W920×223) and W36×231 (W920×344). The box columns that were tested were 24 in. (600 mm) deep, 17 in. (430 mm) wide, with a plate thickness of 1¾ in. (45 mm). The connection details for box columns and HSS columns are the same, so testing of box columns also represents HSS columns. The prequalification limit for column depth for DuraFuse Frames connections is based on the maximum tested specimen, even though the AISC *Seismic Provisions* permit an 11% increase in depth beyond what has been tested.

As with most other prequalified connections, the DuraFuse Frames moment connection has not been tested with built-up I-shaped or cruciform column sections. However, the CPRP has deemed such columns to be similar enough to rolled shapes to justify prequalification.

3. Plate Limitations

All tests that have been performed for the DuraFuse Frames connection have used plate material that satisfies ASTM A572/A572M Grade 50 (Grade 345) for the cover plates, fuse plates, top plates, and external continuity plates. In the absence of experimental data for other plates, prequalification is limited to plates that satisfy ASTM A572/A572M Grade 50 (Grade 345).

The fuse plates have the additional requirement that the material not have a tensile strength greater than 85 ksi (590 MPa). Because there is no upper limit on the tensile strength of ASTM A572/A572M Grade 50 (Grade 345) steel in the ASTM specifications, it is prudent to have some protection against steel that is too strong in the fuse. The strongest steel that was used for fuses in testing had a mill certified tensile strength of 86.5 ksi (600 MPa).

A variety of geometries for the yielding regions (YR) of the fuse plates have been investigated and found to provide sufficient deformation capacity. The limits on the thickness and width of the YR are based on the dimensions of the YR that have been tested. Figure C-15.1 shows the range of yielding region (YR) width and depths from the qualification tests at UCSD and BYU. The limits for the width [not greater than 4 in. (100 mm) and not less than 1.5 in. (38 mm)] are based on the limits of what has been tested, with some extension because excess rotation capacity was observed for all specimens where fuse yielding or fracture was the governing limit state.

Figure C-15.2 shows the YR width-to-thickness ratios from the qualification tests at UCSD and BYU. The limits for the width-to-thickness ratio (not greater than 4.25 and not less than 1.5) are based on the limits of what has been tested, with some extension because excess rotation capacity was observed for all specimens where fuse yielding or fracture was the governing limit state.

Figure C-15.3 shows the YR width-depth ratios from the qualification tests at UCSD and BYU. The limits for the width-depth ratio (not greater than 1.25 and not less than 0.5) are based on the limits of what has been tested, with some extension because

excess rotation capacity was observed for all specimens where fuse yielding or fracture was the governing limit state.

15.5 CONNECTION DETAILING

For most of the specimens that were tested, bolt holes in the beam and external continuity plates were drilled, and bolt holes in the fuse plates were cut with a plasma cutter. The plasma cut holes were left rough, and some did not meet AISC *Specification* surface requirements. Good performance was observed even when rough holes were present.

The specific bolts that were used in qualification testing were 7/8-in.- (22-mm-) diameter ASTM F3125/F3125M Grade F1852; 1-in.- (25-mm-) diameter ASTM F3125/F3125M Grade F2280; 1 1/8-in.- (29-mm-) diameter ASTM F3125/F3125M Grade F2280; and 1 1/4-in.- (32-mm-) diameter ASTM F3125/F3125M Grade A490 or A490M.

Many of the qualification tests involved a column assembly that had been used for previous tests and that had external continuity plates that were bent. In most cases, the bent plates were not straightened prior to the installation of the fuse plate, and gaps of 1/2 in (13 mm) or more between the fuse plate and external continuity plates were closed without providing shims, with the fuse plate and external continuity plates bending during bolt tightening. These tests exhibited acceptable rotation capacity, with multiple cycles beyond the minimum required for prequalification. The 1/4 in. (6 mm) limit on gap is given to be consistent with the Bolted Flange Plate connection.

15.6 DESIGN PROCEDURE

The design procedure for the DuraFuse Frames connection results in fully restrained connections where the beams and columns remain essentially elastic during seismic events.

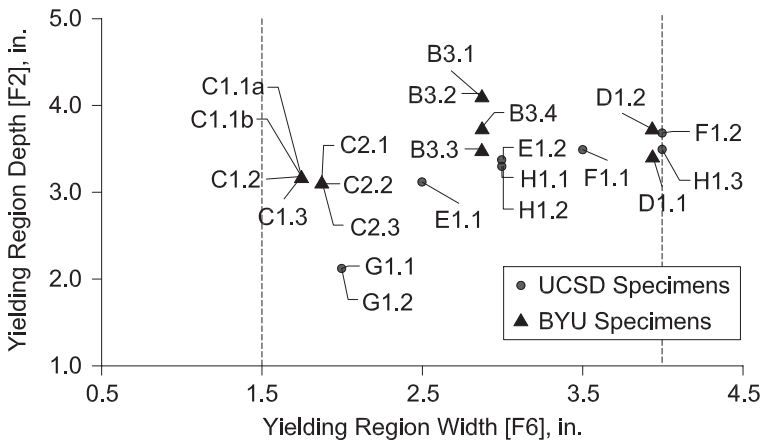


Fig. C-15.1. Yielding region widths and depths that have been tested.

With the DuraFuse Frames connection, the maximum probable moment at the column face is governed by the geometry of the fuse plate. The dimensions of the yielding regions on the fuse plate, which are determined in the final steps of the design procedure, are selected so that when the fuse plate has yielded and fully strain-hardened, the moment at the face of the column, M_{pr} , will not exceed M_p of the beam.

This approach differs from most other special moment frame connections where the plastic hinge capacity of the beam is determined first and then extrapolated to the column face. The procedure is different for the DuraFuse Frames connection because a plastic hinge never forms in the beam. The beam does not yield at the face of the column, even though the moment at the face of the column may reach M_p , because the beam is reinforced by the top plates and fuse plates in the connection region.

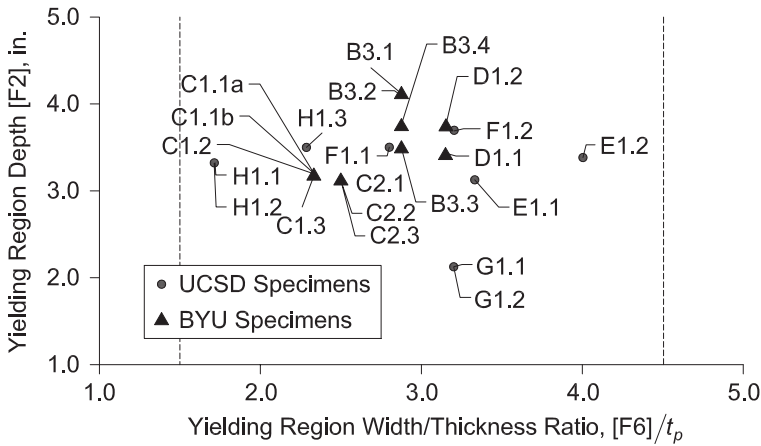


Fig. C-15.2. Yielding region width/thickness ratios that have been tested.

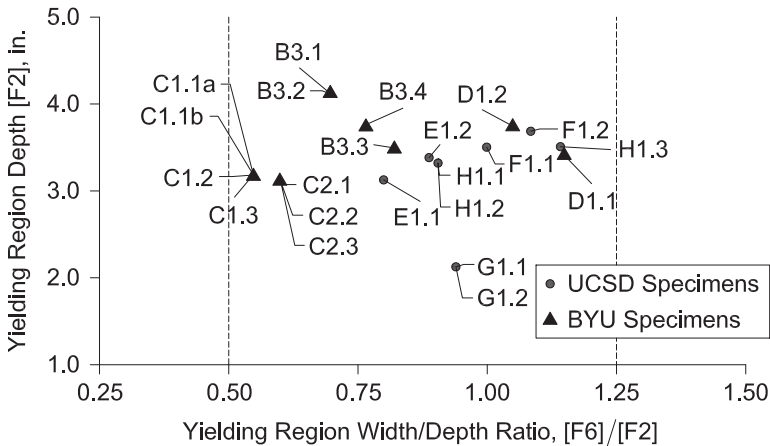


Fig. C-15.3. Yielding region width/depth ratios that have been tested.

The force determined in **Step 2** is the maximum force that is transmitted to each of the external continuity plates at the bottom flange level.

It is the intent of the design criteria to provide a fully restrained (FR) connection. In **Step 2**, if M_{pr} is less than M_p , analysis is required to justify that the connection is FR. The stiffness of the connection, K_s , may be computed using the following equations, which are derived based on the stiffness of the individual connection plates. Alternatively, other methods or FE models may be used to quantify connection stiffness.

$$K_s = \frac{(d_b + [T2])2}{\left(\frac{1}{K_{top}} + \frac{1}{K_{bot}}\right)} \quad (C15.6-1)$$

where

$$K_{bot} = \frac{2}{\left(\frac{1}{K_1} + \frac{1}{K_2} + \frac{1}{K_3} + \frac{1}{K_4}\right)} \quad (C15.6-2)$$

$$K_{top} = \frac{(2P + M - 1)s[T2]G}{[P10] - [P2]} \quad (C15.6-3)$$

and where

G = shear modulus of elasticity of the plate, ksi (MPa)

$$K_1 = \frac{(2[F2] + [F3])[T2]G}{[F4]} \quad (C15.6-4)$$

$$K_2 = \frac{2[F2][T2]G}{[F6]} \quad (C15.6-5)$$

$$K_3 = \frac{2(0.37)E[T2][F2]^3}{[F6]^3} \quad (C15.6-6)$$

$$K_4 = \frac{(2[F2] + [F3])[T2]G}{[F8]} \quad (C15.6-7)$$

The cover plate design in **Step 3** is similar to the design of doubler plates in conventional special moment frame connections. The cover plates are designed to remain elastic under the maximum forces, and they are designed with the same local buckling criteria as traditional doubler plates.

The maximum bolt size requirement in **Step 4** is the same as the requirement for the Bolted Flange Plate and Double-Tee connections, even though a plastic hinge is not expected to form in the beam of the DuraFuse Frames connection.

The required number of bolts in **Step 5** is governed by bolt bearing strength. Similar to other prequalified bolted connections, the bolts in the DuraFuse Frames connection are pretensioned, but the connection is not slip critical.

The bolts on each line are divided into m -bolts and p -bolts in **Step 5** because there may be different spacing between the bolt on the alignment line and the next bolt in the column direction (see Figure 15.2). It is also convenient to have them differentiated into two groups for some demand calculations (see **Step 9** for example). The total number of bolts on each line is the sum of the m -bolts and the p -bolts.

The maximum beam rotation considered in design is set by the engineer. In **Step 6**, a minimum value of 0.06 rad ensures good behavior beyond the 0.04 drift limit. The design calculation for the gap between the beam and the column is based on the expected movement of the beam bottom flange toward the column, assuming a center of rotation at the beam top flange. Assuming the center of rotation at the top flange is conservative because in testing, bolt slip of the top bolts has caused the center of rotation to be somewhere between the top flange and the mid-depth of the beam. When the center of rotation is between the top flange and the mid-depth of the beam, rather than at the top flange level, there is less movement of the beam bottom flange for a given rotation angle.

In **Step 7**, the welds between the cover plates and the column flanges are designed based on the demands shown in Figure C-15.4. Only part of the weld is considered effective for resisting normal forces [Figure C-15.4(c)] because the normal forces are being delivered at the external continuity plate levels.

In **Step 8**, the welds between the external continuity plates and cover plates are designed based on the demands shown in Figure C-15.5. Only part of the weld is considered effective for resisting normal forces because the load paths near the column flanges are much stiffer for normal forces than the paths through the center of the cover plate.

If normal demands are resulting in large welds, bridge plates (Figure 15.3) can be added to the connection to assist in transmitting the normal force.

The limit states that are checked in **Step 9** are illustrated in Figures C-15.6 and C-15.7. The Mode 2 and Mode 3 limit states shown in Figure C-15.6 are outside the definition of “rupture in the net section” because they overlap the welded portion of the external continuity plate. However, Mode 2 and Mode 3 are conservatively checked in addition to the block shear limit state (Figure C-15.7) because they sometimes give lower capacity.

In **Step 13**, the single-plate shear connection has a required normal strength even though the bolts are slotted in the normal direction. The required normal strength corresponds to the maximum probable slip force that can develop in the bolts.

Step 15 is checking the limit state illustrated in Figure C-15.8 and the similar mode for the fuse plate. Because the top plates may be from a different heat than the fuse plate, the R_t factor is not used for top plate checks, and greater shear area is required for the top plates than the fuse plate.

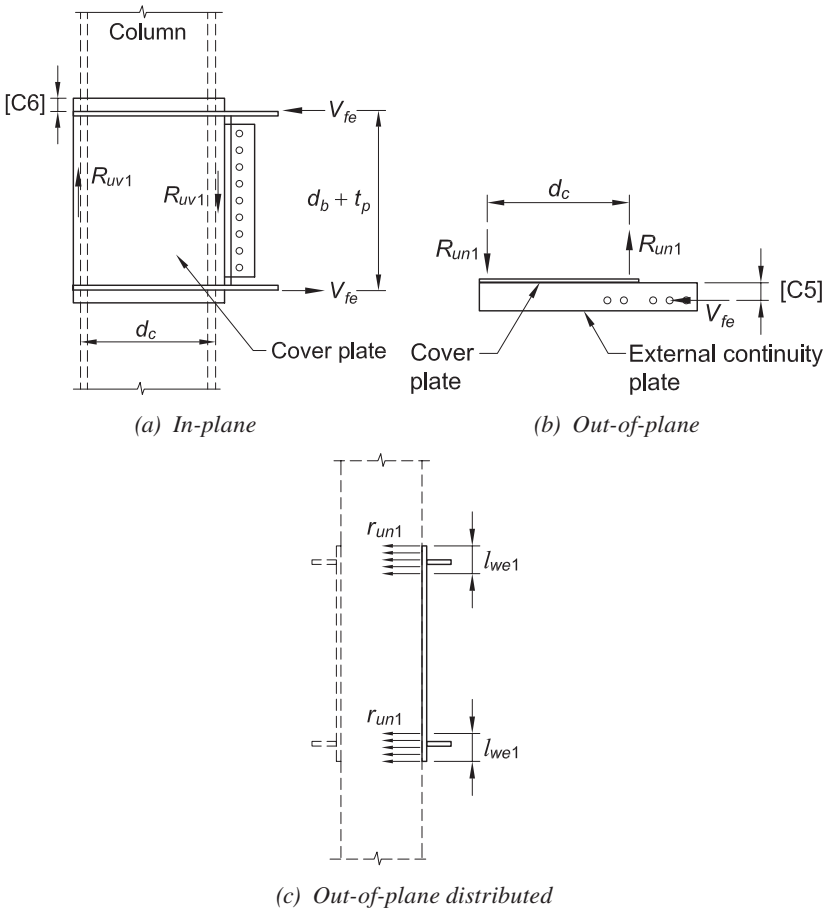


Fig. C-15.4. Weld demands for cover plate welds.

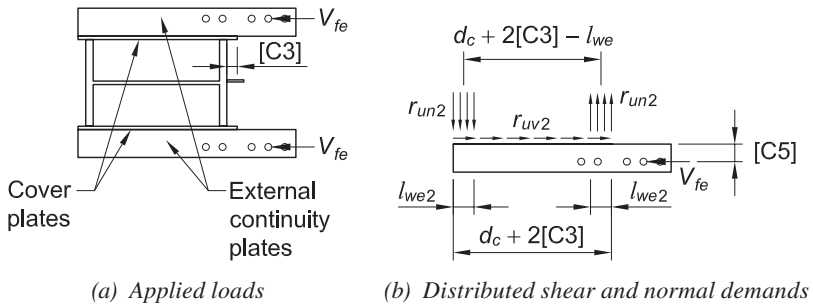


Fig. C-15.5. Weld demands for external continuity plate welds.

Step 16 is checking the limit states illustrated in Figure C-15.9.

Figure C-15.10 shows the lower-bound model that is used in **Step 17** to evaluate the capacity of the top plate to resist the combined shear and bending caused by $V_{fe} + P_d$ and the eccentricity [P10]-[P2]. The lower-bound model assumes the shear forces are resisted by the “*m*” region, and that the bending moment is resisted by the “*e*” regions. This capacity check is translated into a minimum required value for [P2] that appears in **Step 17**.

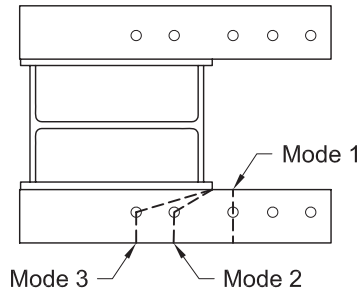


Fig. C-15.6. Tension rupture limit state for external continuity plate.

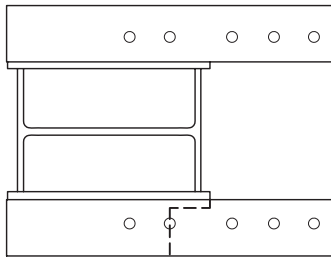


Fig. C-15.7. Block shear limit state for external continuity plate.

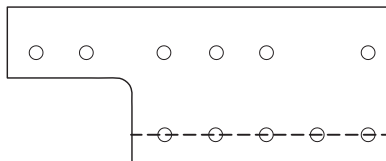


Fig. C-15.8. Shear rupture of net section on top plate.

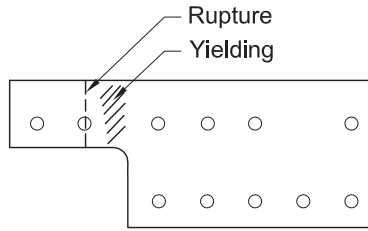
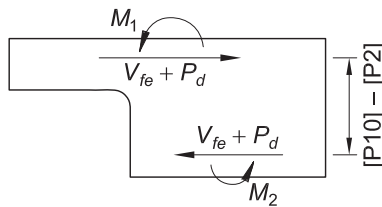


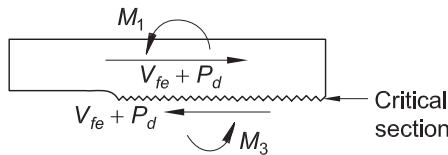
Fig. C-15.9. Tension rupture of net section on top plate.



$$M_1 + M_2 = (V_{fe} + P_d)([P10] - [P2])$$

For plate check, conservatively assume $M_2 = 0$:

$$M_1 = (V_{fe} + P_d)([P10] - [P2])$$



Conservatively assume $M_3 = M_1$

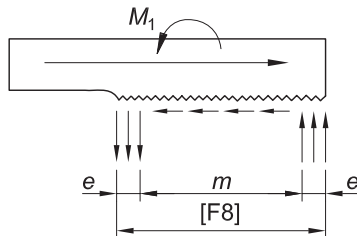


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

In **Step 18**, checks are made for limit states that are related to the dimension [F6]. Figure C-15.11 illustrates the net section that is being checked.

In **Steps 18** and **19**, the dimensions of the yielding regions (YR) on the fuse plates are determined and compared with the dimensions that have been investigated experimentally.

In **Step 19**, the equation for determining [F2] appears complicated because it incorporates an expression for strain hardening that depends on the geometry of the yielding region. The cyclic shear hardening factor, C_h , for the yielding regions of the fuse plates has been calibrated from the various qualification experiments. This special hardening factor quantifies an increase in ultimate shear stress beyond what is calculated by $R_t(0.6F_u)$.

The C_h factor is comparable, in some respects, to the C_{pr} factor used for other moment connections, but differs in that it is used in conjunction with the expected ultimate stress, R_tF_u , rather than in conjunction with the expected yield stress, R_yF_y . The cyclic shear hardening factor depends on the width-to-depth ratio of the yielding region but is also influenced by the width-to-thickness ratio. The cyclic shear hardening factor, $C_{h,des}$, that is incorporated into the design equations is:

$$C_{h,des} = A - B \left(\frac{[F6]}{[F2]} \right) - C \left(\frac{[F6]}{t_p} \right) \quad (C-15.6-8)$$

where A , B , and C are calibrated factors: $A = 1.52$, $B = 0.16$, and $C = 0.09$.

Table C-15.1 compares the cyclic shear strain hardening observed in experimental testing, $C_{h,exp}$, with values of $C_{h,des}$ from the calibrated equation for the specimens where the testing was continued all the way through fuse plate tearing. The values for $C_{h,des}/C_{h,exp}$ in Table C-15.1 indicate that the calibrated equation for $C_{h,des}$ is reasonable, although it is quite conservative for Specimen D1.1.

The limit states being checked in **Step 20** are illustrated in Figure C-15.12.

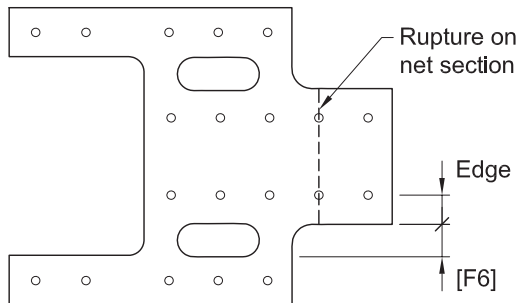


Fig. C-15.11. Limit states related to dimension [F6].

TABLE C-15.1.
Cyclic Shear Hardening Factor from Experiments and Design Equation

Specimen	w/d	w/t	$C_{h,exp}$	$C_{h,des}$	$C_{h,des}/C_{h,exp}$
B3.1	0.697	2.88	1.16	1.15	1.00
B3.2	0.697	2.88	1.17	1.15	0.990
B3.3	0.821	2.88	1.08	1.13	1.05
B3.4	0.767	2.88	1.13	1.14	1.01
C2.1	0.600	2.50	1.13	1.20	1.06
D1.1	1.05	3.15	0.930	1.07	1.15
E1.2	0.890	4.00	1.03	1.02	0.990
F1.2	1.08	3.20	0.990	1.06	1.07
G1.2	0.940	3.20	1.07	1.08	1.01
H1.3	1.14	2.29	1.15	1.13	0.980

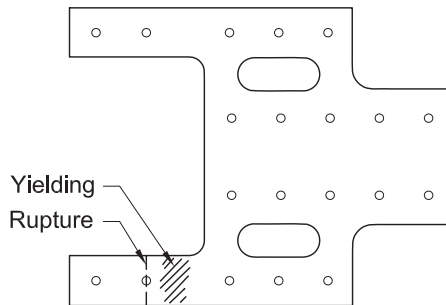


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

APPENDIX A

CASTING REQUIREMENTS

A1. CAST STEEL GRADE

The cast steel grade is selected for its ability to provide ductility similar to that of rolled steel. The material has a specified minimum yield stress and tensile strength of 50 ksi (345 MPa) and 80 ksi (550 MPa), respectively. The ASTM specification requires the castings be produced in conjunction with a heat treatment process that includes normalizing and stress relieving. It also requires each heat of steel meet strict mechanical properties. These properties include the specified tensile and yield strengths, as well as elongation and area reduction limitations.

A2. QUALITY CONTROL (QC)

See Commentary Section 3.7.

2. First Article Inspection (FAI) of Castings

The intent of this section is that at least one casting of each pattern undergo first article inspection (FAI). When a casting pattern is replaced or when the rigging is modified, FAI is to be repeated.

3. Visual Inspection of Castings

All casting surfaces shall be free of adhering sand, scales, cracks, hot tears, porosity, cold laps, and chaplets. All cored holes in castings shall be free of flash and raised surfaces. The ASTM specification includes acceptance criteria for the four levels of surface inspection. Level I is the most stringent criteria. The Manufacturers Standardization Society (MSS) specification includes a set of reference comparators for the visual determination of surface texture, surface roughness, and surface discontinuities.

4. Nondestructive Testing (NDT) of Castings

These provisions require the use of nondestructive testing (NDT) to verify the castings do not contain indications that exceed the specified requirements.

Radiographic testing (RT) is capable of detecting internal discontinuities and is specified only for the FAI. The ASTM specifications contain referenced radiographs and five levels of RT acceptance. The lower acceptance levels are more stringent and are typically required on high-performance aerospace parts such as jet engine turbine blades or on parts that may leak, such as valves or pumps. Level III is considered the industry standard for structurally critical components.

Ultrasonic testing (UT) is also capable of detecting internal discontinuities and is specified for production castings. The ASTM specification includes seven levels of UT acceptance. The lower acceptance levels are more stringent and are typically reserved for machined surfaces subject to contact stresses, such as gear teeth. Level 3 is considered the industry standard for structurally critical components.

The areas to be covered by RT or UT are those adjacent to the junctions of:

- (1) The vertical flange and the horizontal flange.
- (2) The vertical flange and the vertical stiffener.
- (3) The horizontal flange and the vertical stiffener.

Magnetic particle testing (MT) is required to detect other forms of discontinuities on or near the surface of the casting. The ASTM specification includes five levels of MT acceptance. The lower levels are more stringent and are typically reserved for pressure vessels. Level V is considered the industry standard for structurally critical components.

Shrinkage is one of the more commonly occurring internal discontinuities and is a result of metal contraction in the mold during solidification. Shrinkage is avoided using reservoirs of molten metal known as risers that compensate for the volumetric contraction during solidification. Numerical modeling of solidification and prediction of shrinkage have been the focus of a number of investigations performed in conjunction with the Steel Founders' Society of America (SFSA). Niyama et al. (1982) developed a criterion that relates the casting temperature gradient and cooling rate. Based on the Niyama criterion, Hardin et al. (1999) developed a correlation between casting simulation and radiographic testing. Subsequently, Carlson et al. (2003) determined that variation in internal porosity (shrinkage) was related to the pattern and rigging of the casting mold.

Based on these conclusions, the provisions require RT and MT on the FAI of castings to verify that the pattern and rigging are capable of producing a satisfactory casting. Subsequent castings manufactured with the same pattern and rigging require UT and MT to verify production consistency.

Research performed by Briggs (1967) on the effect of discontinuities found that castings perform satisfactorily under loads in excess of service requirements even with discontinuities of considerable magnitude. Testing demonstrated fatigue and static failures occurred at the location of maximum stress regardless of the presence of discontinuities in other sections.

6. Tensile Requirements

Coupons or keel blocks for tensile testing shall be cast and treated from the same batch of representative castings. Each test specimen shall have complete documentation and traceability. If the specimens fail to meet required specifications, then all the castings they represent shall be rejected.

A3. MANUFACTURER SUBMITTALS

Submittal documents allow a thorough review on the part of the engineer of record, the authority having jurisdiction, and outside consultants, if required.

APPENDIX B

FORGING REQUIREMENTS

There is no Commentary for this Appendix.

REFERENCES

The following references have been reviewed as a basis for the prequalification of the connections described in this standard. Although some references are not specifically cited in this standard, they have been reviewed by the AISC Connection Prequalification Review Panel and are listed here to provide an archival record of the basis for this standard in accordance with the requirements of AISC *Seismic Provisions* Chapter K.

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APPENDIX A

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