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

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Approved by the Connection Prequalification Review Panel

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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_{vb}	Gross area of a shear bolt measured through its shank, in.^{2} (mm^2)	13.5
$A_{y\text{-}link}$	Yield area of reduced Yield-Link section, in.^{2} (mm^2)	12.9
$A'_{y\text{-}link}$	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 1.0E_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 *Specification*, ksi (MPa)	6.7.1
F_{nv}	Nominal shear stress of bolt from the AISC *Specification*, 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
$F_{u\text{-}link}$	Specified minimum tensile strength of Yield-Link stem material, ksi (MPa)	12.9
F_w	Nominal weld design strength per the AISC *Specification*, 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_yF_{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
$F_{y\text{-}link}$	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 ¼ of column depth above the top edge of lower-story side plates to ¼ 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
$L_{bm\text{-}side}$	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)
$L_{col\text{-}side}$	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
$L_{slot\text{-}horz}$	Shear plate horizontal bolt slot length, in. (mm)	12.9
$L_{slot\text{-}vert}$	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
$L_{y\text{-}link}$	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 moment 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_{uv}	Additional moment due to shear amplification from center of reduced beam section to centerline of column, kip-in. (N-mm)	5.4(2)
M_{uv}	Additional moment due to shear amplification from the plastic hinge, kip-in. (N-mm)	14.4
M^*_{pb}	Expected flexural strength of the beam, kip-in. (N-mm)	11.4(3)
M^*_{pc}	Nominal flexural strength of the column, kip-in. (N-mm)	11.4(3)
M^*_{pcl}	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}	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
$M_{u\text{-}sp}$	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
$M_{ye\text{-}link}$	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
$P_{r\text{-}weld}$	Required strength of Yield-Link stem to Yield-Link flange weld, kips (N)	12.9
$P_{r\text{-}link}$	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)
$P_{u\text{-}sp}$	Required axial strength of beam web-to-column flange connection, kips (N)	12.9
$P_{ye\text{-}link}$	Expected yield strength of the Yield-Link, kips (N)	12.9
$P'_{y\text{-}link}$	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 *Manual*, 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
$V_{u\text{-}bolt}$	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 ¼ 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 *Specification*, 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
$b_{bm\text{-}side}$	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
$b_{col\text{-}side}$	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
$d_{b\text{-}brp}$	Diameter of bolt connecting buckling restraint plate to beam flange, in. (mm)	Figure 12.3
$d_{b\text{-}flange}$	Diameter of bolt connecting Yield-Link flange to column flange, in. (mm)	12.9
$d_{b\text{-}sp}$	Diameter of bolts in shear plate, in. (mm)	12.9
$d_{b\text{-}stem}$	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
$n_{bolt\text{-}sp}$	Total number of bolts in shear plate	12.9
$n_{bolt\text{-}sp\text{-}horz}$	Total number of horizontal bolts resisting axial force in the shear plate in line with the central bolt	12.9
$n_{bolt\text{-}sp\text{-}vert}$	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_{vb}	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
$t_{BRP\text{-}min}$	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

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:

<u>American Institute of Steel Construction (AISC)</u>
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*)

<u>American Society of Mechanical Engineers (ASME)</u>
ASME B46.1-2019 *Surface Texture, Surface Roughness, Waviness, and Lay*

<u>American Society for Nondestructive Testing (ASNT)</u>
ASNT SNT-TC-1A-2016 *Personnel Qualification and Certification in Nondestructive Testing*

<u>ASTM International (ASTM)</u>
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 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 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 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

$$M_{pr} = C_{pr} R_y F_y Z_e \tag{2.4-1}$$

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

$$C_{pr} = \frac{F_y + F_u}{2F_y} \le 1.2 \tag{2.4-2}$$

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} (mm^3)

4. Shear Force at Plastic Hinge

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

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

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

$$M_f = M_{pr} + V_h S_h \tag{2.4-4}$$

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 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 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 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 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 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 ⅛ in. (3 mm) of the base metal surface and the end of the weld finished, except at continuity plates where removal to within ¼ 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 μin. (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 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 1½ 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 ½ 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 ½ 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 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.

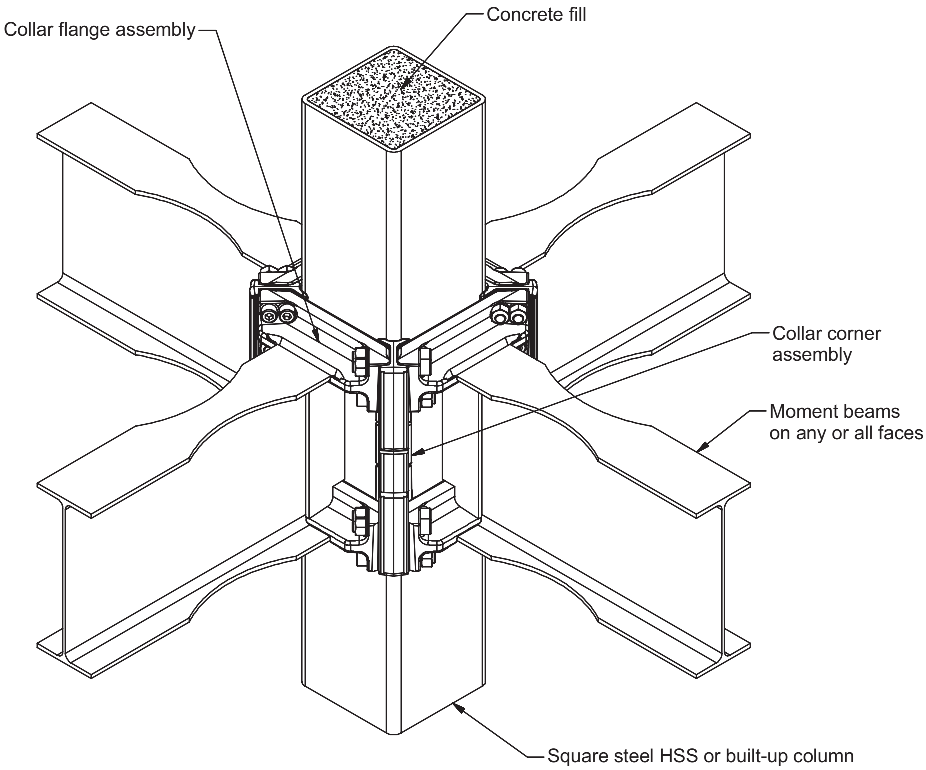


Fig. 10.1. Assembled ConXL moment connection.

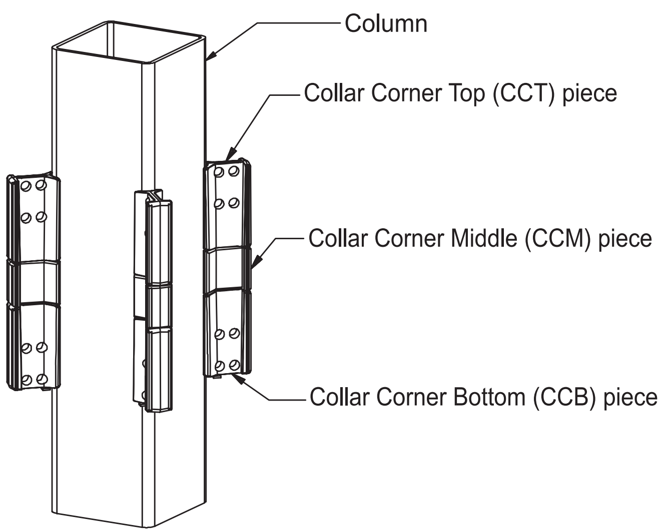


Fig. 10.2. Column with attached collar corner assemblies.

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

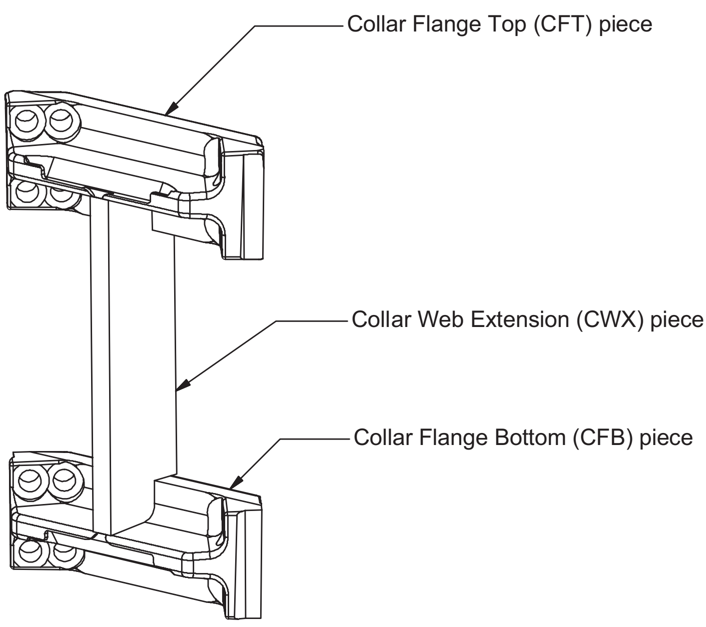


Fig. 10.3. Collar flange assembly.

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.

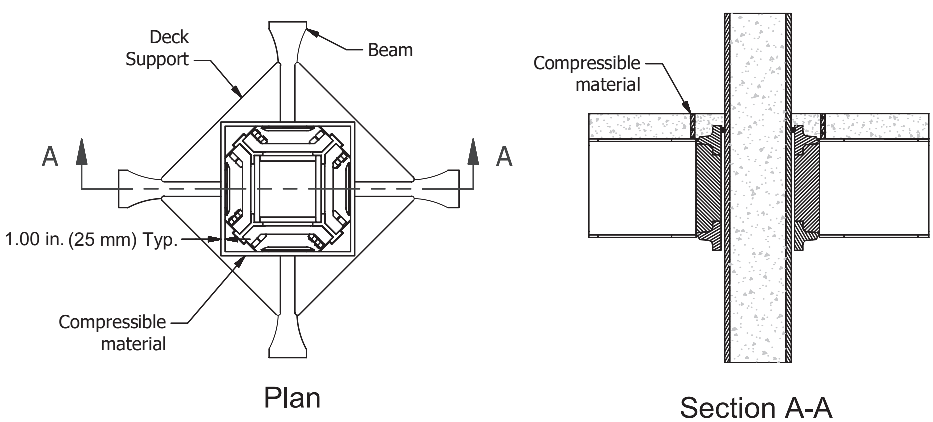


Fig.10.4. Use of compressible material to isolate structural slab from connection.

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 ⅜ in. (10 mm). Column wall thickness for HSS columns shall not be less than ⅜ 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/ft^3 (1 800 kg/m^3). 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 ¾ 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 ⅛ in. (3 mm) larger than the nominal bolt diameter. Bolt holes shall be drilled.

(5) Collar corner bolt holes shall be ⅛ 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 ±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 ⅜ in. (10 mm) fillet reinforcing per Figure 10.7.

(5) Collar flanges shall be welded to CWX pieces with 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 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.

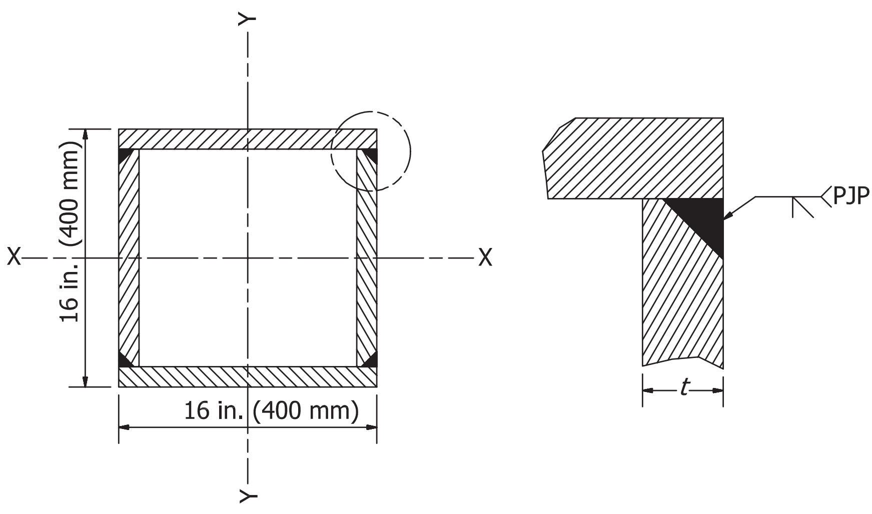


Fig. 10.5. Built-up box column flange-to-web connection detail.

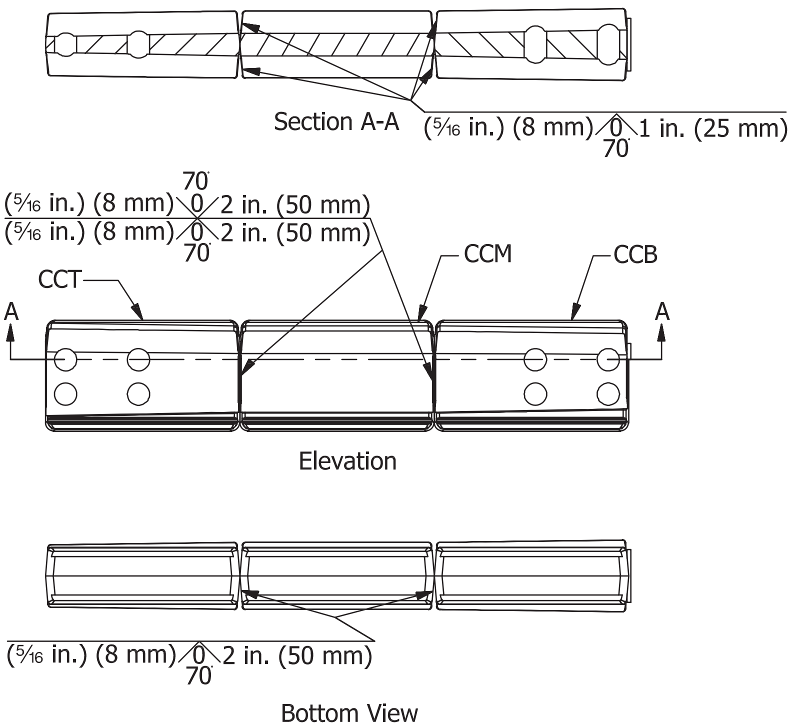


Fig. 10.6. Collar corner assembly welding.

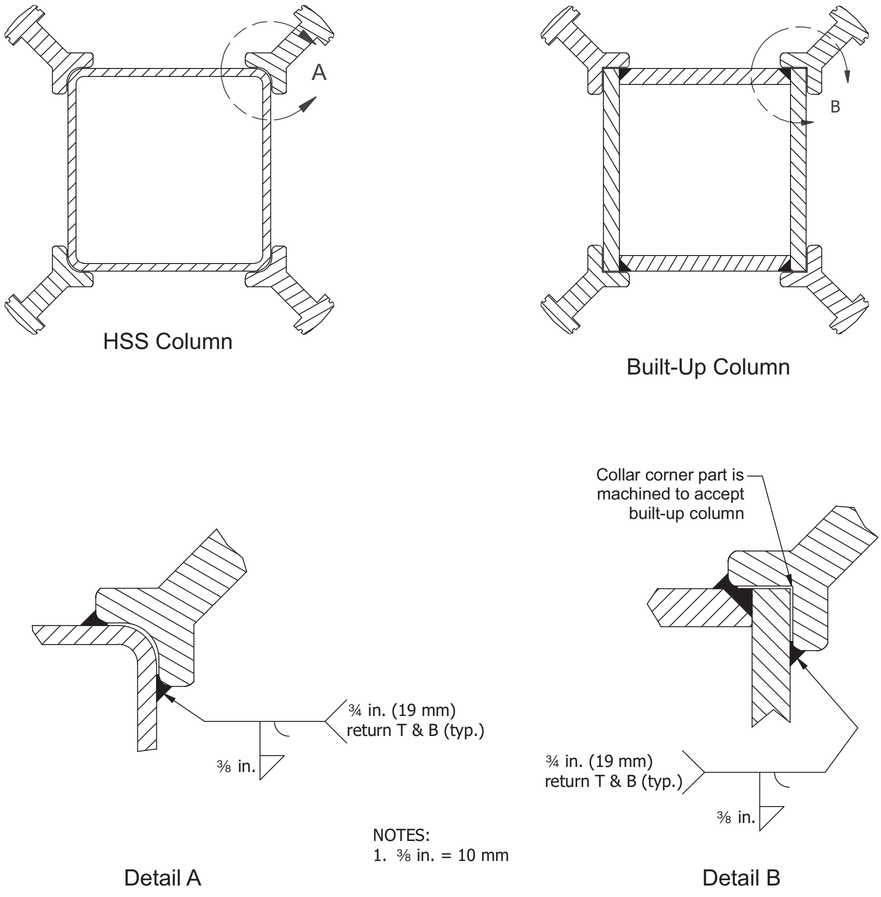


Fig. 10.7. Collar corner assembly-to-column weld, plan view.

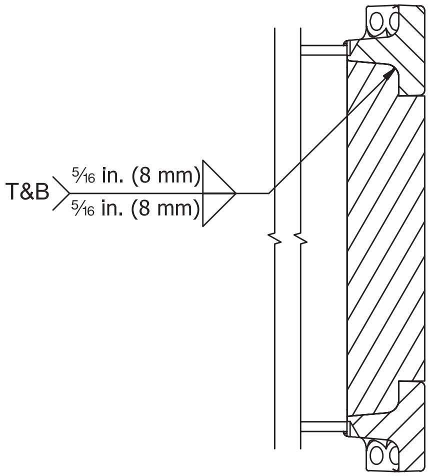


Fig. 10.8. Collar web extension-to-collar flange welds, elevation.

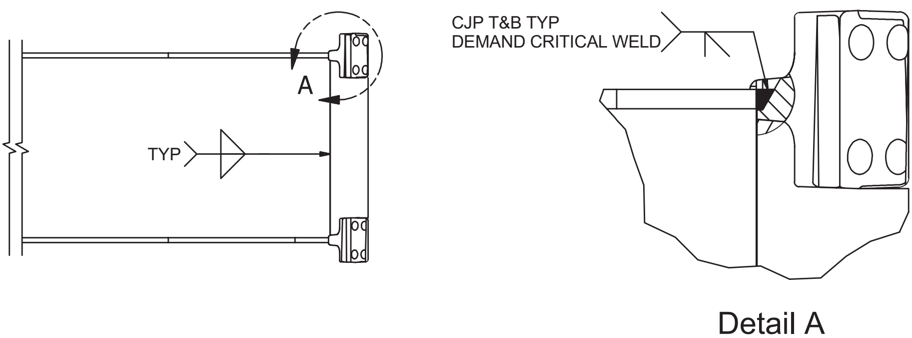


Fig. 10.9. Collar flange assembly-to-beam welds, elevation.

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:

$$M_{bolts} = M_{pr} + V_h s_{bolts} \tag{10.7-1}$$

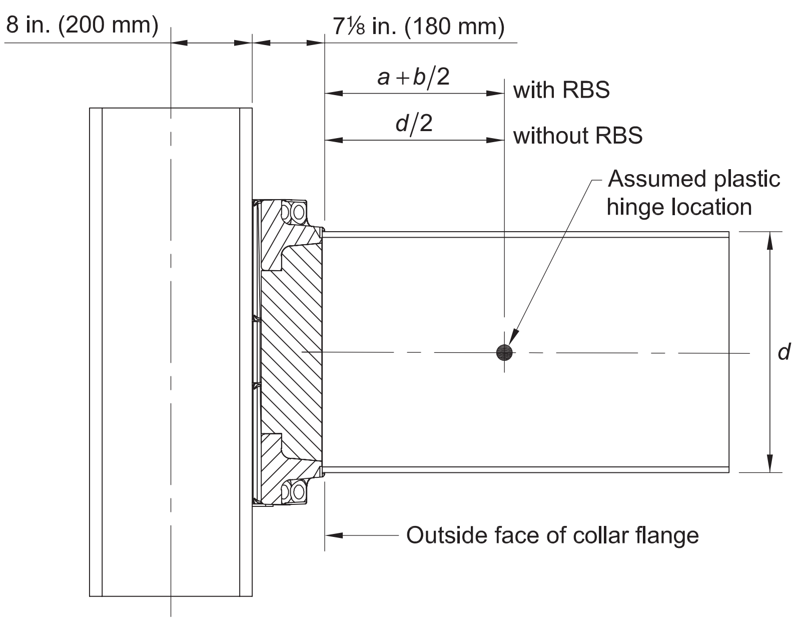


Fig. 10.10. Assumed plastic hinge location.

where

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

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

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

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

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 7⅛ in. (180 mm) as illustrated in Figure 10.10

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

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

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

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)

$$= \frac{M_{bolts}}{n_{cf} d \sin 45°} = 0.177 \frac{M_{bolts}}{d} \tag{10.7-5}$$

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:

$$t_f^{CWX} \geq \frac{\sqrt{2}V_{cf}}{\phi_n F_w l_w^{CWX}} \tag{10.7-6}$$

where

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

l_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:

$$t_f^{CC} \geq \frac{\sqrt{2}V_f}{\phi_n F_w l_w^{CC}} \tag{10.7-7}$$

where

l_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

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

where

$A_{pz} = 2d_c t_{col} + 4(d_{leg}^{CC} t_{leg}^{CC})$ (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

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

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:

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

where

A_c = area of concrete in column, in.^{2} (mm^2)

A_s = area of steel in column, in.^{2} (mm^2)

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} (mm^3)

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.

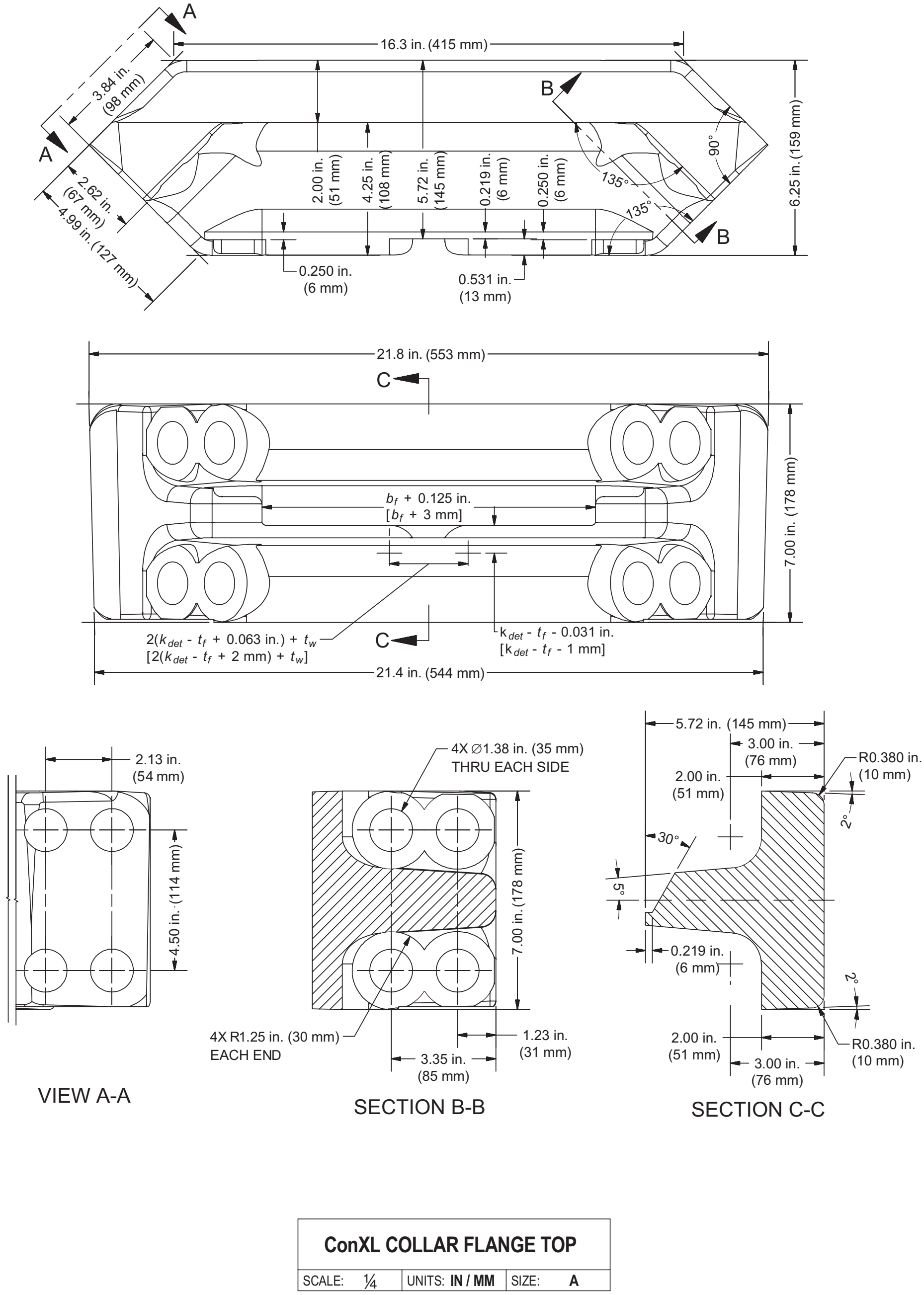
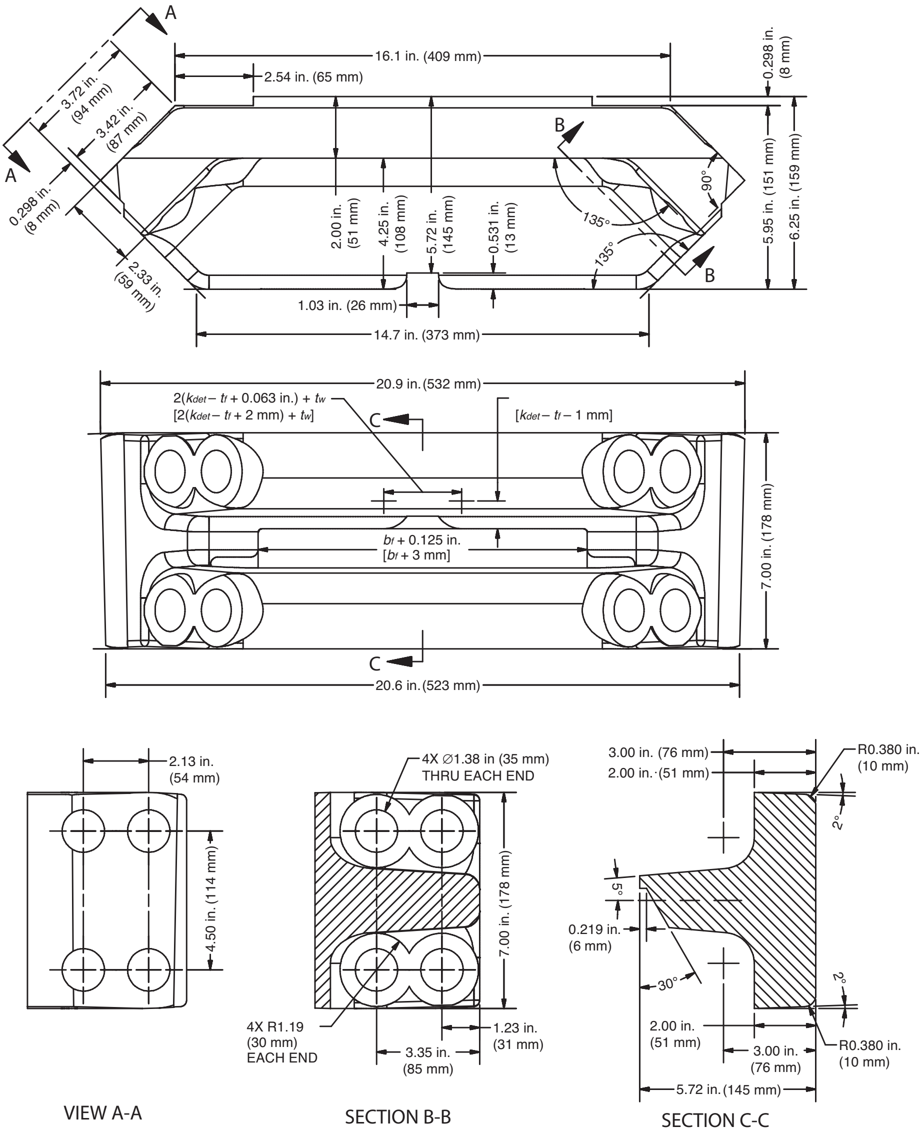
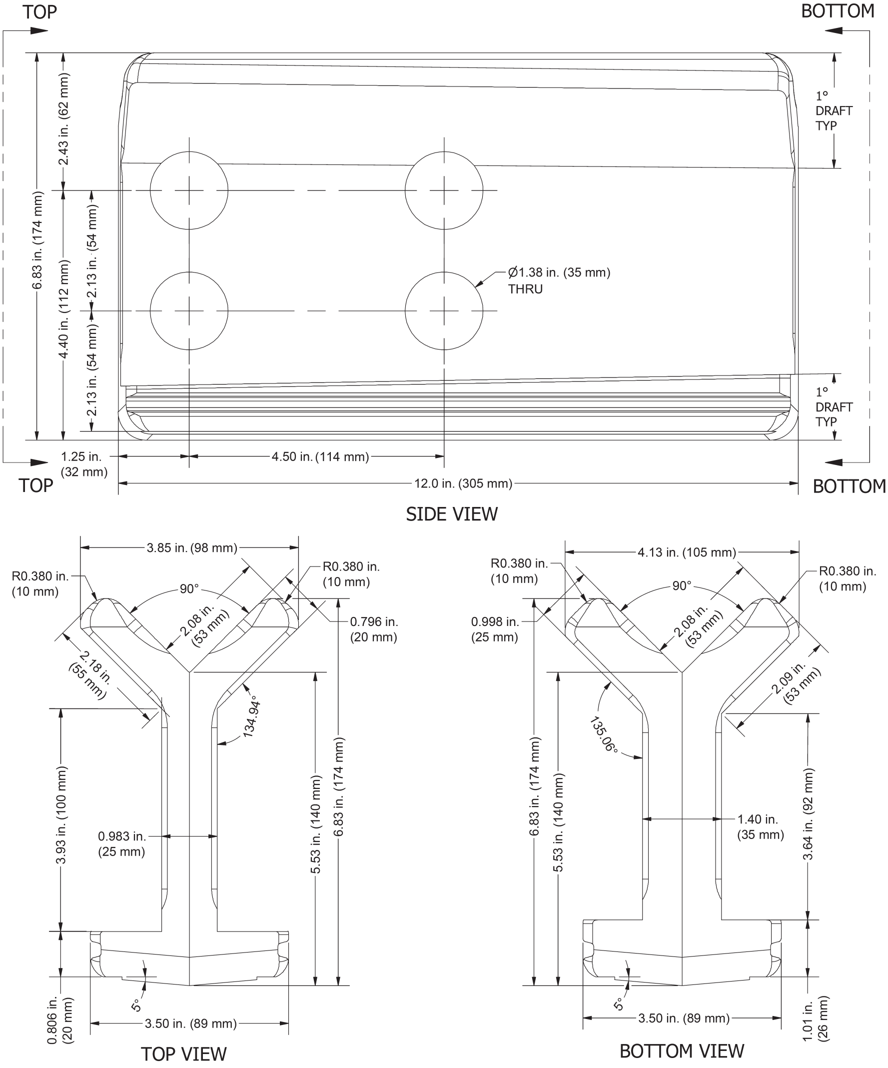


Fig. 10.11. Forged collar flange top (CFT).



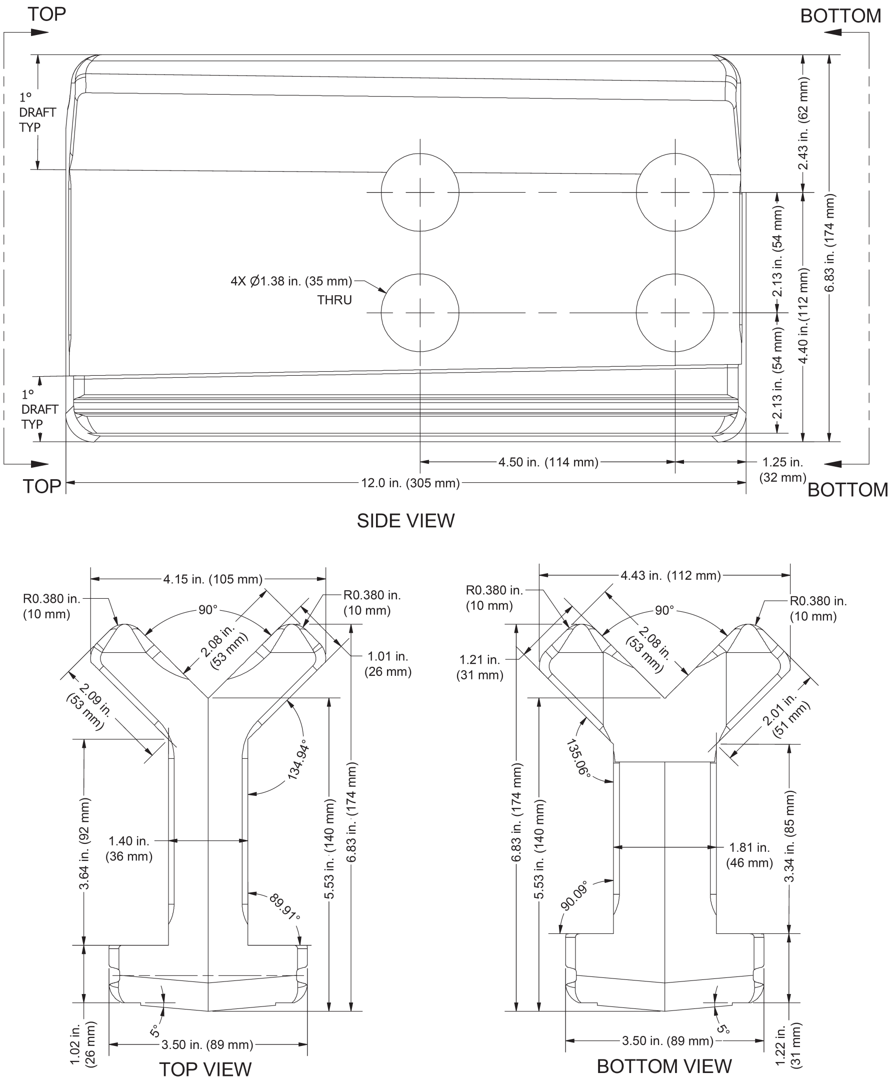
ConXL COLLAR FLANGE BOTTOM		
SCALE: ¼	UNITS: IN / MM	SIZE: A

Fig. 10.12. Forged collar flange bottom (CFB).



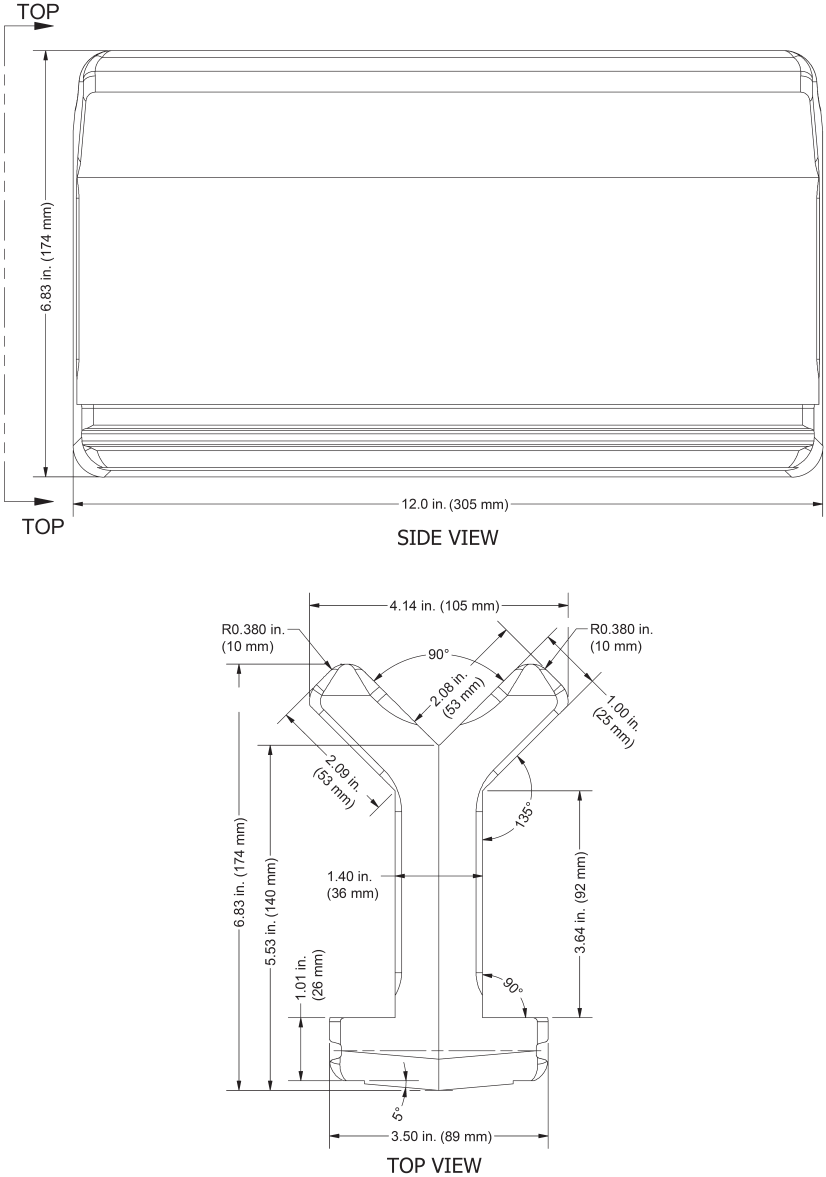
ConXL COLLAR CORNER TOP		NOTES	
		Description	Tolerance Value
		FORGING TOLERANCE	+/- 0.3%
		DIE WEAR TOLERANCE	+ 0.5% / - 0.0
UNITS: **IN / MM**	SIZE: **A**	MILLING TOLERANCE	+/- 0.020"

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



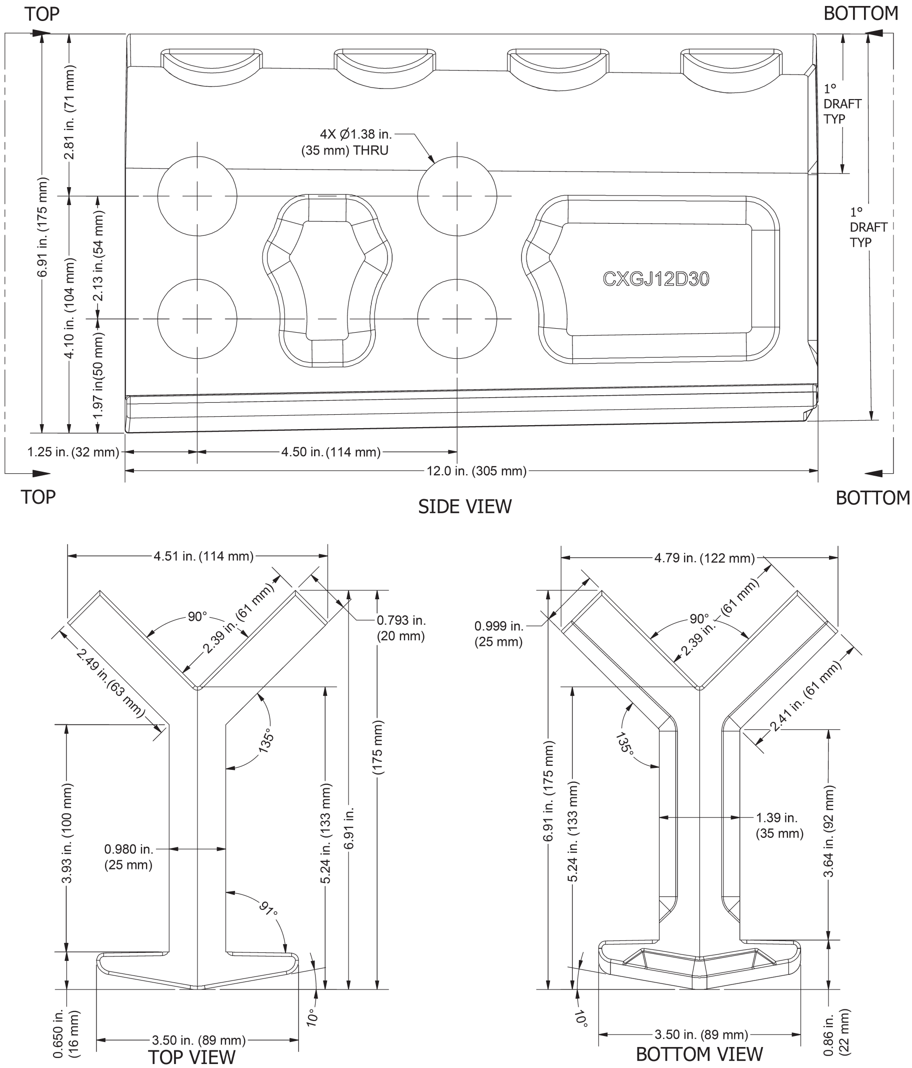
ConXL COLLAR CORNER BOTTOM		NOTES	
		Description	Tolerance Value
		FORGING TOLERANCE	+/- 0.3%
		DIE WEAR TOLERANCE	+ 0.5% / - 0.0
UNITS: IN / MM	SIZE: A	MILLING TOLERANCE	+/- 0.020"

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



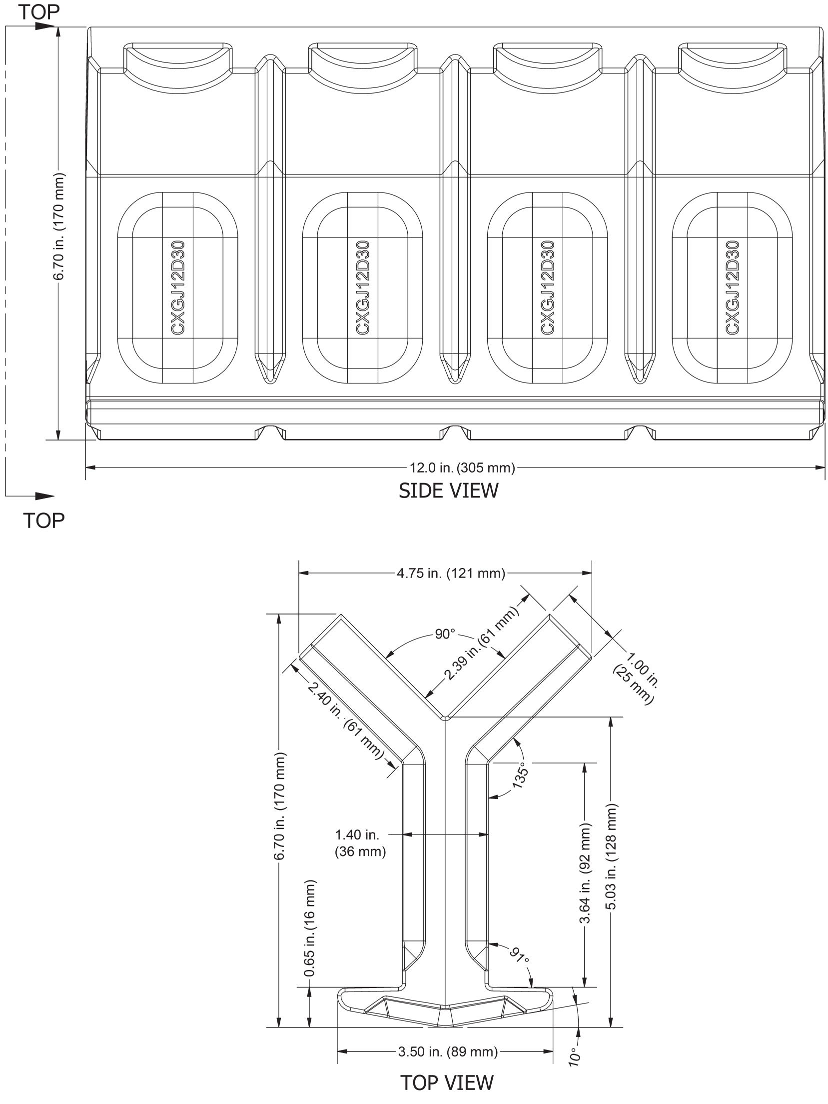
ConXL COLLAR CORNER MIDDLE				NOTES	
				Description	Tolerance Value
				FORGING TOLERANCE	+/- 0.3%
				DIE WEAR TOLERANCE	+ 0.5% / - 0.0
	UNITS:	**IN**	SIZE: **A**	MILLING TOLERANCE	+/- 0.020"

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



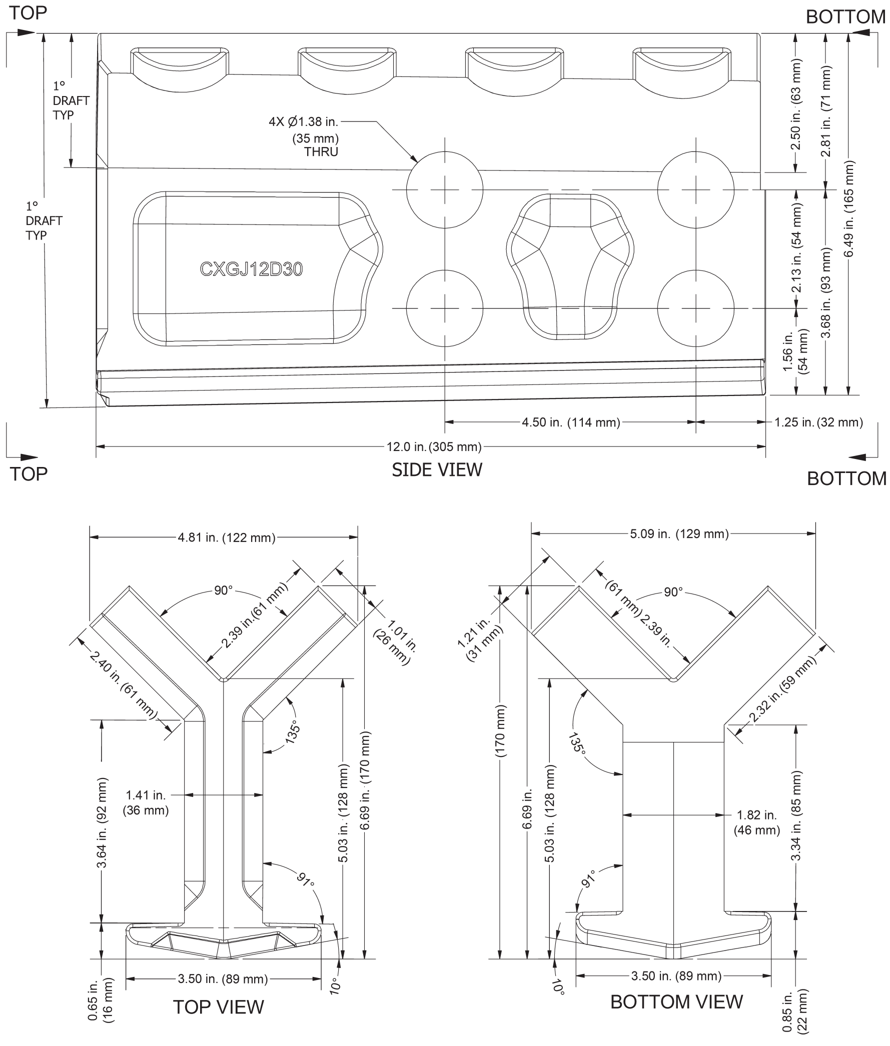
ConXL COLLAR CORNER TOP		NOTES	
		Description	Tolerance Value
		FORGING TOLERANCE	+/- 0.3%
		DIE WEAR TOLERANCE	+ 0.5% / - 0.0
UNITS: **IN / MM**	SIZE: **A**	MILLING TOLERANCE	+/- 0.020"

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



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

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

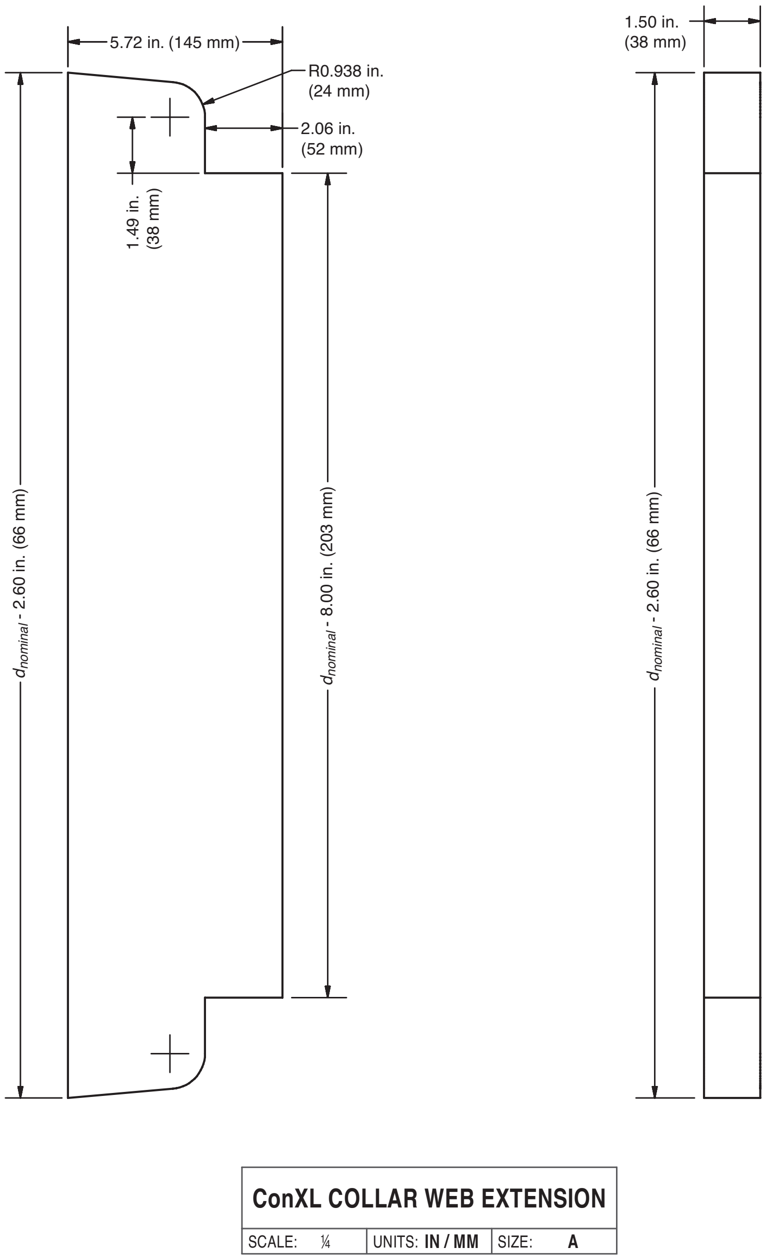


Fig. 10.19. Collar web extension (CWX).

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 16×16×⅝*	W18×76 RBS	N/A	0.05
1102	Planar	HSS 16×16×⅝*	W18×119	N/A	0.05
1103	Planar	HSS 16×16×⅝*	W24×84 RBS	N/A	0.06
1104	Planar	HSS 16×16×⅝*	W24×104	N/A	0.05
1105	Planar	HSS 16×16×⅝*	W24×117×6†	N/A	0.04
1106	Planar	HSS 6×16×⅝*	W24×117×9†	N/A	0.04
1107	Planar	HSS 16×16×⅝	W21×62 RBS	N/A	0.04
1108	Planar	HSS 16×16×⅝	W21×62 RBS	N/A	0.06
1201	Planar	HSS 16×16×⅝*	W30×108 RBS	N/A	0.05
1202	Planar	HSS 16×16×⅝*	W30×108 RBS	N/A	0.05
1203	Planar	HSS 16×16×⅝*	W30×90	N/A	0.04
1204	Planar	HSS 16×16×⅝*	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 16×16×½	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 ¾ 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 ConXtech 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:

$$R_n = \mu D_u h_f T_b n_s \qquad (\textit{Spec.}\ \text{Eq. J3-4})$$

where

D_u = 1.13
T_b = 102 kips (450 kN)
h_f = 1.0
n_s = 1
μ = 0.50
ϕ = 0.85 for oversized bolt holes

$$\phi R_n = 0.85(0.50)(1.13)(1.0)(102\text{ kips})(1.0)$$
$$= 49.0\text{ kips/bolt (220 kN/bolt)}$$

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.