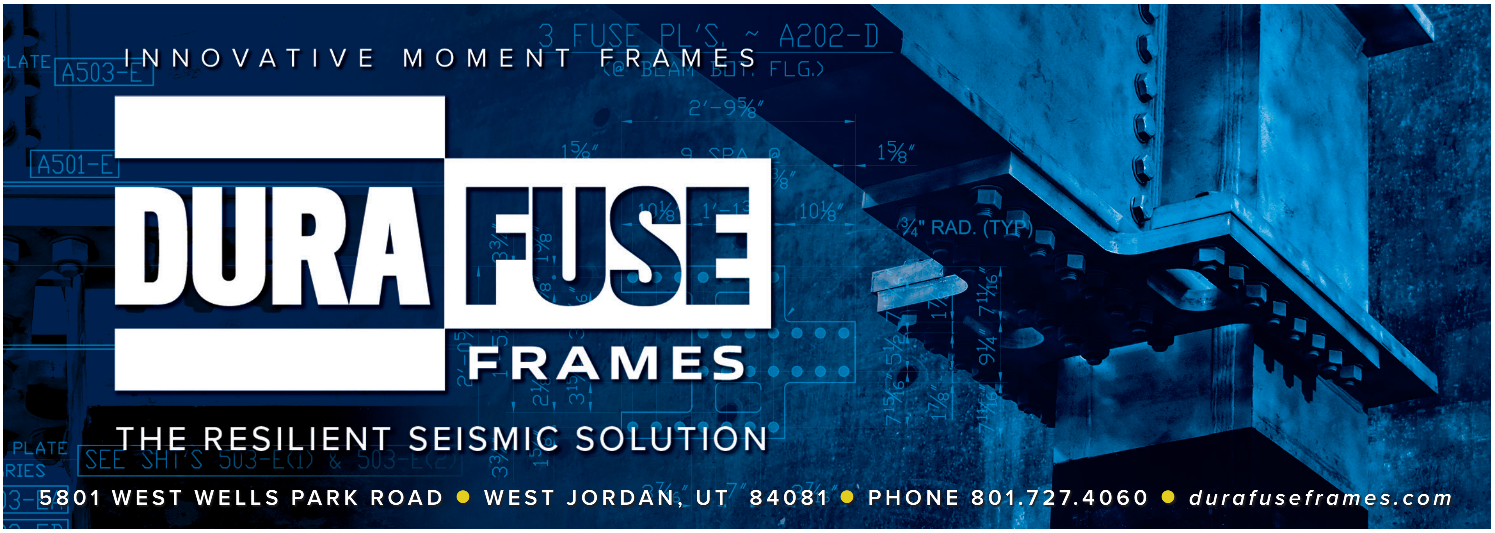


08/21

TECHNICAL BULLETIN 2

AISC 358
CHAPTER 15*
DuraFuse Frames Connection

IAPMO UES ER 610

*This text represents the draft proposal of the ANSI/AISC 358 Chapter 15 reviewed by the AISC Connection Prequalification Review Panel (CPRP). Its adoption into ANSI/AISC 358 is expected in 2022.

CHAPTER 15

DURAFUSE FRAMES MOMENT CONNECTION

The user's attention is called to the fact that compliance with this chapter of the standard requires use of an invention covered by multiple U.S. and foreign pa-tents. 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 pa-tent holder has filed a statement of willingness to grant a license under these rights on reasonable and nondiscriminatory terms and conditions to applicants desiring to obtain such a license, and the statement may be obtained from the standard's developer.

15.1 GENERAL

The DuraFuse Frames connection utilizes plates to connect beams to columns. For I-shaped columns, the column has cover plates on each side that are fillet welded to the column flanges [Figure 15.1(a) and (b)]. For box or HSS col-umns, the sides of the column may function as the cover plates [Figure 15.1(c)]. Four external continuity plates that extend past the face of the column are fillet welded to the column cover plates. The column has a shear tab, with horizontal slotted holes, that is fillet welded to the column face. The beam web, with standard holes, is attached to the shear tab with pretensioned bolts. The beam flanges are attached to the external continuity plates via top plates and a fuse plate (Figure 15.1). The beam flanges and external continuity plates have standard holes, while the top plates and fuse plate have oversized holes. The fuse plate is proportioned such that certain regions of the plate experience shear yielding when the connection is subjected to severe earthquake loading. The fuse plate is bolted in place, so that it could be removed and replaced following a severe earthquake. The top plates are intended to experience minimal yield-ing, such that they would not require repair following a severe earthquake. The various plates in the connection are proportioned such that the beam remains essentially elastic.

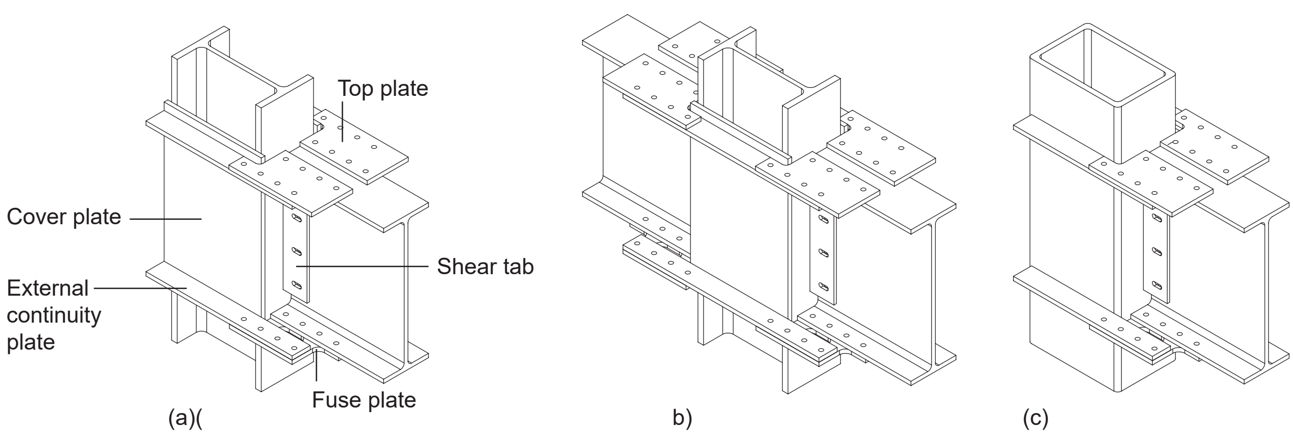


Fig. 15.1. DuraFuse Frames connections (a) one-sided with wide-flange beam and col-umn (b) two-sided with wide flange beams and column (c) wide-flange beam with HSS column (two-sided HSS column connection permitted but not shown.)

The inelastic mechanism is the same for DuraFuse Frames biaxial configurations (Figure¬ 15.2). Fuse plates are proportioned to be the yielding element. The other plates and welds in the connections are capacity-based design elements, sized in accordance with the AISC Specification and the AISC Seismic Provisions using the maximum forces that can be delivered by the fuse plates.

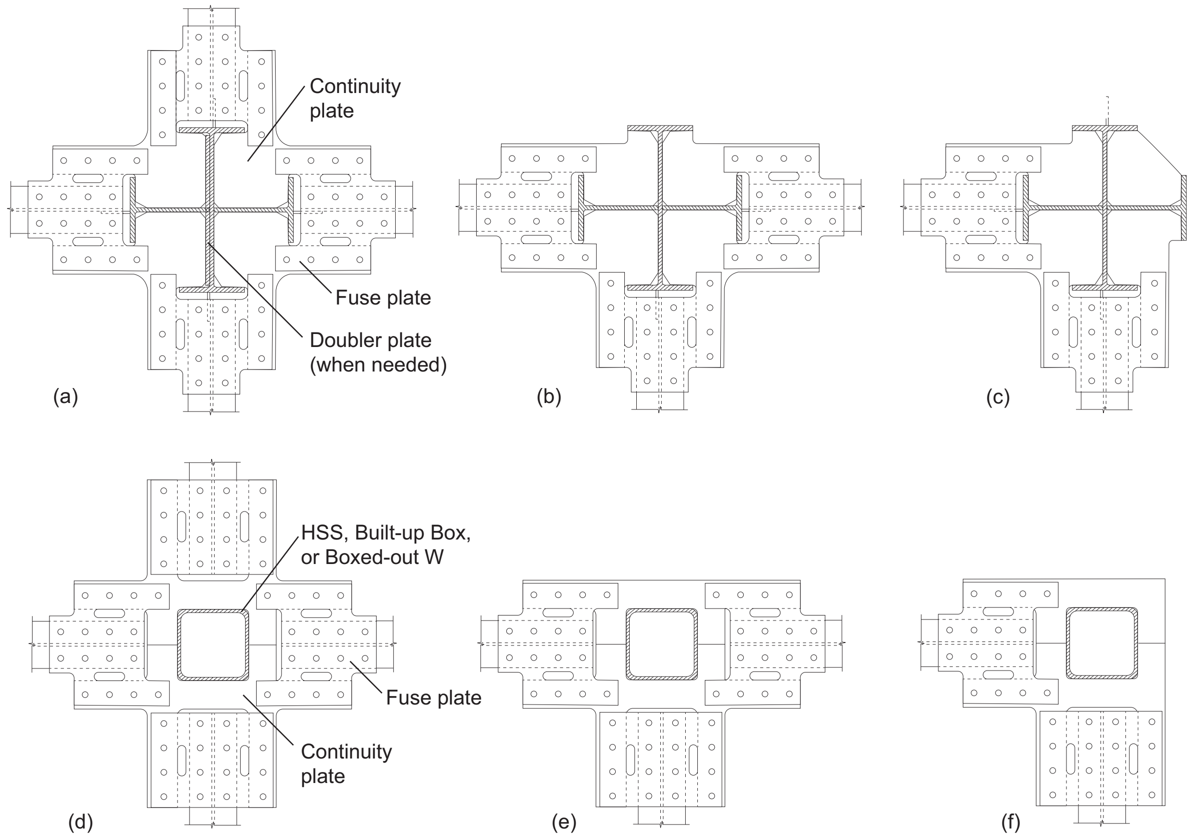


Fig. 15.2 Bottom views of DuraFuse Frames biaxial configurations: (a) four-sided flanged cruciform; (b) three-sided flanged cruciform; (c) two-sided flanged cruciform; (d) four-sided HSS or box; (e) three-sided HSS or box; (f) two-sided HSS or box.

15.2 SYSTEMS

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

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

15.3 PREQUALIFICATION LIMITS

1. Beam Limitations

Beams shall satisfy the following limitations:

(1) Beams shall be rolled wide-flange or built-up I-shaped members conforming to the requirements of Section 2.3.

(2) Beam depth shall be limited to a maximum of W40 (W1020) for rolled shapes. Depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes.

(3) Beam weight shall be limited to a maximum of 309 lb/ft (464 kg/m).

(4) There are no limits on the beam web width-to-thickness ratio beyond those listed in the AISC *Specification*. The beam flange width-to-thickness ratio shall not exceed λp per AISC *Specification* Table B4.1b.

(5) Lateral bracing of beams and joints: there are no requirements for stability bracing of beams or joints beyond those in the AISC *Specification*.

2. Column Limitations

Columns shall satisfy the following limitations:

(1) Columns shall be any of the shapes permitted in Section 2.3. (2) The beam shall be connected to the column via connection plates.

(3) Rolled shape column depth shall be limited to W36 (W920). Column depth of built-up sections shall not exceed the depth permitted for rolled wide-flange shapes. Built-up box-columns shall not have a width exceeding 17 in. (610 mm).

(4) There is no limit on weight per foot of columns.

(5) There are no additional requirements for flange thickness.

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

(7) Lateral bracing of columns shall conform to the requirements of the AISC *Seismic Provisions*.

3. Plate Limitations

(1) All connection plates shall be fabricated from structural steel that complies with ASTM A572/A572M Grade 50 (Grade 345). Fuse plates shall be fabricated from plates that have mill certified tensile strengths less than or equal to 85 ksi, unless independent material testing determines that the tensile strength is less than or equal to 85 ksi.

(2) The thickness of the external continuity plates, top plates, and fuse plates shall not exceed 2 in. (50 mm) and shall not be less than 0.5 in. (13 mm).

(3) The width of the yielding regions in the fuse plates shall not exceed 4.0 in. (102 mm) and shall not be less than 1.5 in. (38 mm)

(4) The width-thickness ratio of the yielding regions in the fuse plates shall not exceed 4.25 and shall not be less than 1.5.

(5) The width-depth ratio of the yielding regions in the fuse plates shall not exceed 1.25 and shall not be less than 0.5.

(6) The protected zone of the connection consists of the fuse plate.

15.4 COLUMN-BEAM RELATIONSHIP LIMITATIONS

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

Column-beam moment ratios shall be limited as follows.

(a.) For SMF systems, the column-beam moment ratio shall conform to the requirements of the AISC *Seismic Provisions*. The value of ΣM^*_{pb} shall be taken equal to $\Sigma(M_{pr} + M_{uv})$ where M_{pr} is the maximum probable moment at the fuse, and M_{uv} can be computed as $V_b(d_c/2)$, where d_c is the depth of the column and V_b can be computed as $2M_{pr}/L_h + V_g$, where L_h is the clear distance between column faces and V_g is the beam shear caused by gravity loads based on the load combination $1.2D + f_1L + 0.2S$, where f_1 is the load factor determined by the applicable building code for live loads, but not less than 0.5. The value of ΣM^*_{pc} shall be the sum of the projections of the nominal flexural strengths (M_{pc}) of the column above and below the joint, at the potential hinge location located one quarter of the column depth above and below the cover plates edges.

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

15.5 CONNECTION DETAILING

Figure 15.3 through Figure 15.7 define the symbols for various plate thickness-es, dimensions, weld sizes, and bolt quantities that are pertinent to the connection and discussed in the design steps.

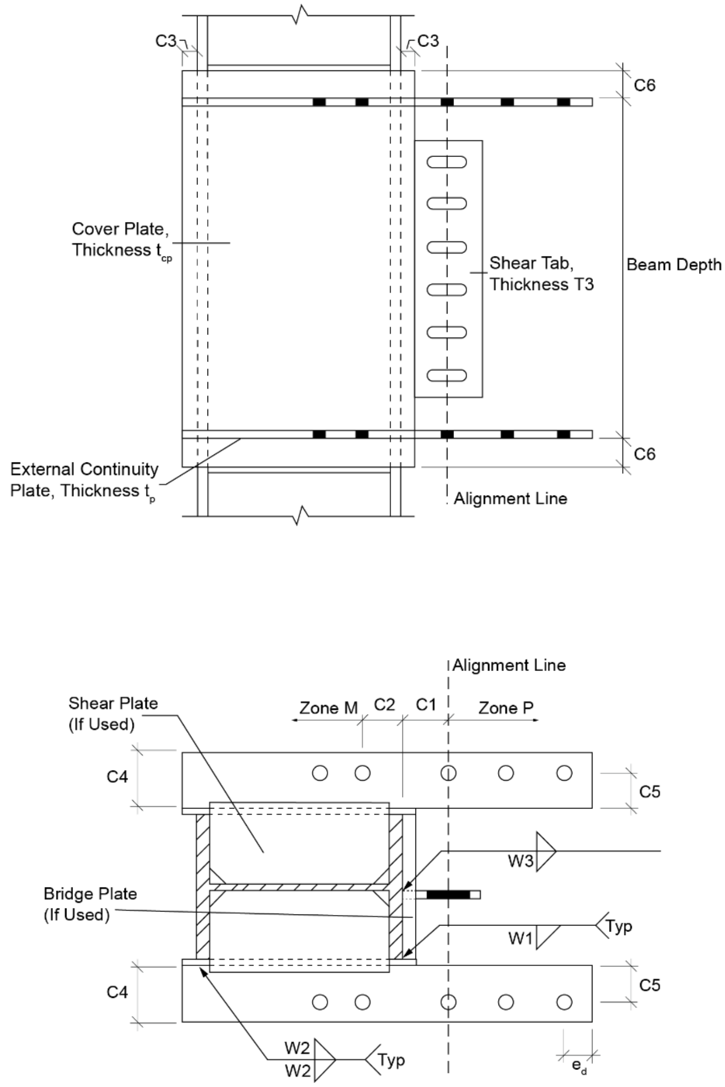


Fig. 15.3. Column assembly detail.

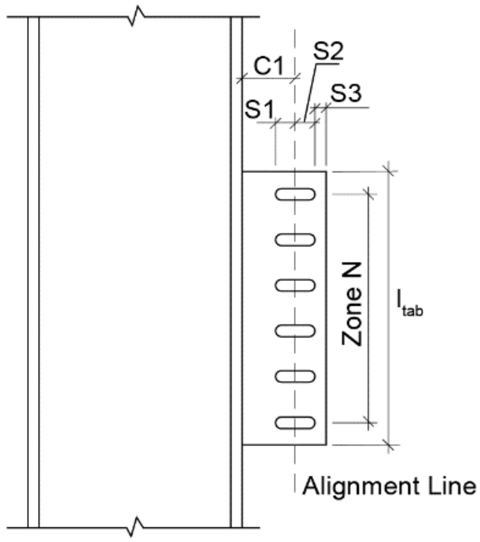


Fig. 15.4. Shear tab detail.

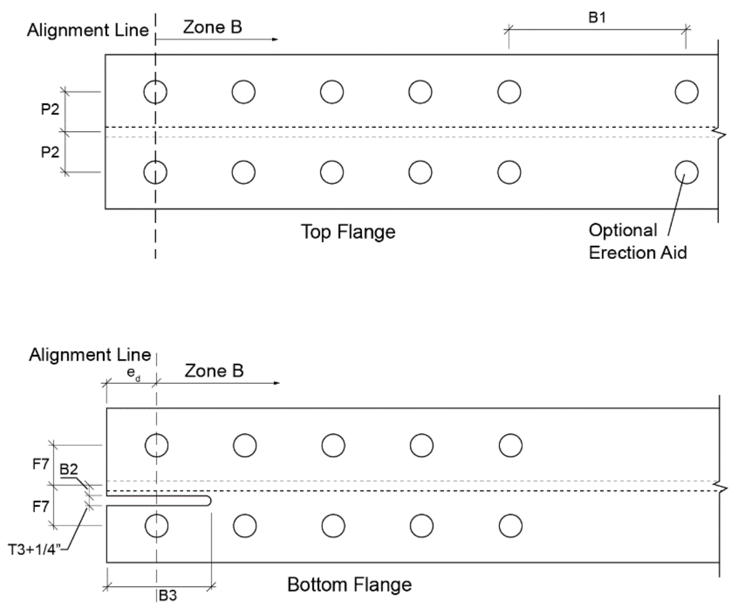


Fig. 15.5. Beam end detail.

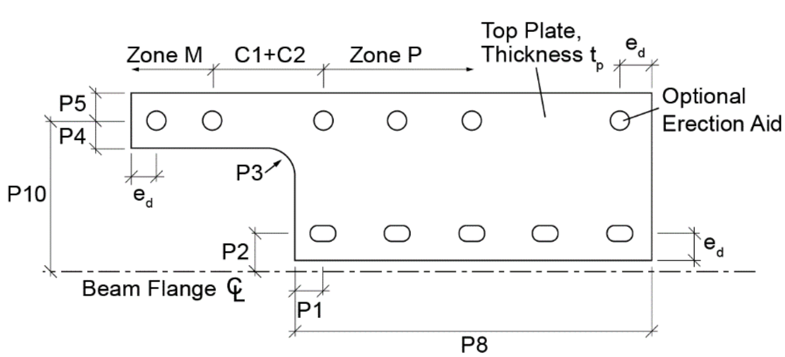


Fig. 15.6. Top plate detail.

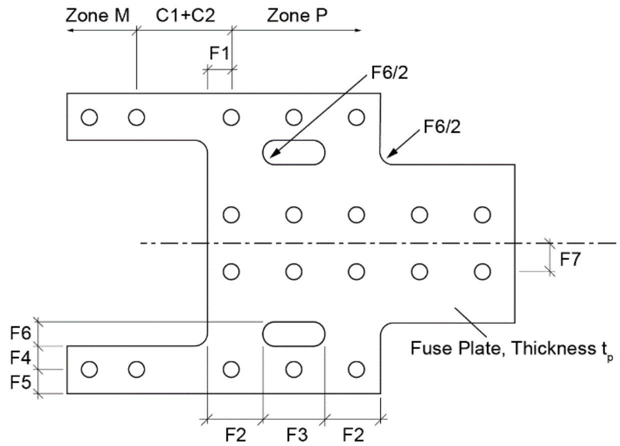


Fig. 15.7. Fuse plate detail.

1. Welds

The welds designated [W1] and [W2] in Figure 15.3 shall be held back one weld size. When [W2] is made to an HSS column, the weld holdback shall be measured from the end of the workable flat. [W1] is omitted when the walls of an HSS or box column function as sole cover plates.

2. Bolts

(1) Bolts shall be arranged symmetrically about the axis of the beam and shall be limited to two lines of bolts in each beam flange.

(2) Standard holes shall be used in the beam flanges and in the external continuity plates. Holes in the top plates and fuse plate shall be oversized except for the short-slotted holes indicated in Fig. 15.6

(3) Bolt holes in beam flanges and plates shall be made by drilling, sub-punching and reaming, laser cutting, plasma cutting, or water-jet cutting.

(4) All bolts shall be installed as pretensioned high-strength bolts.

(5) Bolts shall be ASTM F3125 Grade A325, Grade A325M, Grade A490, Grade A490M, Grade F1852, or Grade F2280 assemblies. Threads shall be excluded from the shear plane. Bolt diameter is limited to 1-1/4 in (32 mm) maxi-mum.

(6) Faying surfaces shall have a Class A slip coefficient but shall not have a surface roughness that achieves a friction coefficient in excess of 0.5 (Class B)

(7) A gap of up to and including ¼ in. (6 mm) between the top plate and the external continuity plate or beam flange and between the fuse plate and the external continuity plate or beam flange may be closed through deformation of the plates. Shim plates with a maximum overall thickness of ¼ in. (6 mm) may also be used.

3. Beam Slot

Beam bottom flanges may be slotted in accordance with Figure 15.5. The distance between the edge of the bolt holes and the cut shall be at least 3/8 in. (10 mm).

User Note: The purpose of the slot in the beam bottom flange is to permit the beam to drop down past the shear tab during erection.

4. Plate Fabrication and Repair

The roughness of all thermally cut surfaces shall be no greater than an ANSI surface roughness of 1000 micro-inches. Gouges and notches in the protected zone shall be repaired per AWS D1.8.

15.6 DESIGN PROCEDURE

GENERAL

Step 1. Choose trial sizes for the beam section(s) and column section. Confirm that the column-beam moment ratio is satisfied per Section 15.4 (2).

Step 2. Determine the maximum probable force that will develop at the bottom flange level on each side, V_{fe}.

$$V_{fe} = \frac{M_{pr}}{2(d_b + t_p)} \tag{15.6-1}$$

where:

M_{pr} = maximum probable moment at the fuse location, kip-in. (N-mm). This moment shall not exceed M_p and shall not be less than the beam flexural demand defined by the applicable building code. If M_{pr} is less than M_p, analysis shall be performed to demonstrate the connection is fully restrained.

User Note: The fuse plate is proportioned in **Step 19** to have an expected ultimate capacity corresponding to M_{pr}.

d_b = depth of the beam, in. (mm)

t_p = thickness of the top plates, fuse plate, and external continuity plates, in. (mm). This thickness is typically equal to the beam flange thickness, t_{bf}, rounded up to the next standard plate size.

COVER PLATE THICKNESS

Step 3. Design the cover plates. The required cover plate shear strength is:

$$R_{u,cp} = \sum\left(V_{fe} + P_d\right) \tag{15.6-2}$$

where:

P_d = drag force to be transmitted through the cover plate, computed as the beam axial force defined by the applicable building code divided by two, kips (N)

Check the limit state of shear yielding of the cover plates according to AISC *Specification* Chapter J.

User Note: Because the cover plates will have shear strength greater than ΣV_{fe}, the panel zone check from the AISC Seismic Provisions will be satisfied.

User Note: For Built-up Box and HSS columns the sides of the column may function as cover plates.

The thickness of the cover plate, t_{cp}, shall conform to:

$$t_{cp} \geq \left(d_z + w_z\right)/90 \tag{15.6-3}$$

where:

$d_z = d_b - 2t_{fb}$ of the deeper beam at the connection, in. (mm)

w_z = width of the panel zone between column flanges (d_c -2t_{fc}), in. (mm)

For flanged cruciform columns without cover plates, the required shear strength of the panel zone shall be $2R_{u,cp}$.

User Note: $R_{u,cp}$ is the demand on one cover plate. The total panel zone shear demand is $2R_{u,cp}$.

BOLTS

Step 4. Determine the maximum bolt diameter. Beam plastic hinging or net section fracture is not expected to occur, but to ensure that beam yielding would occur prior to fracture, the net section of the beam shall satisfy the following:

$$Z_{x,net}R_tF_{ub} \geq Z_xR_yF_{yb} \tag{15.6-4}$$

where:

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

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

$Z_{x,net}$ = plastic section modulus of the net section of the beam, in.^{3} (mm^3)

User Note: F3125 Grade F2280 bolts, or similar, with a diameter of 1-1/8 diameter are reasonable for most beams weighing between 100 and 200 lb/ft. A smaller bolt diameter may be more appropriate for lighter beams, or a bigger bolt diameter for heavier beams.

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

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates, n_b . The number of bolts required for the bolt shear limit state is:

$$n_b \geq \frac{V_{fe} + P_d}{\phi r_n} \tag{15.6-5}$$

where:

ϕ_{rn} = design shear strength of one bolt, assuming threads excluded from shear plane

Some of the bolts are on- or to-the-right-of the alignment line (Fig. 15.3). The number of bolts on- and ahead-of the alignment line on the outside lines is n_p (see Figure 15.6) and shall be sufficient to accommodate the depth of two fuse regions on the fuse plate and 3 in. of space between.

$$n_p \geq \frac{\{2[F2_a] + 3\text{in.} - 2e_d\}}{s} + 1 \tag{15.6-6}$$

where:

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

R_{tp} = tensile overstrength factor of the plate, ksi (MPa)

e_d = minimum edge distance for bolt in oversized hole, in. (mm)

s = bolt spacing, in. (mm)

[$F2_a$] = the approximate longitudinal dimension of each fuse region on the plate, in. (mm).

$$= V_{fe}/[2(0.6R_{tp}F_{up}t_p)], \tag{15.6-7}$$

The number of bolts behind the alignment line on the outside lines, n_m, is:

$$n_m = n_b - n_p \tag{15.6-8}$$

where:

n_b = the total number of bolts on the lines of the fuse plate

n_p = number of bolts on or ahead of the alignment line on each outside line of the fuse plate

The nominal slip resistance of each line of bolts, calculated assuming Class B surface and a resistance factor of 1.0, shall be less than V_{fe}.

User Note: DuraFuse Frames moment connections accommodate inelastic deformations through a combination of fuse plate yielding and bolt slip. Since the number of bolts is based on bearing capacity, this check is performed to ensure that bolt slip will occur as part of the inelastic response.

Check bolts for the limit states of bearing and tear-out according to AISC *Specification* Chapter J.

Step 6. Locate the alignment line. The beam-to-column gap and the shear tab slots in the connection shall accommodate a beam rotation, γ_{max}, of at least 0.06 rad.

$$\gamma_{max} \geq 0.06 \tag{15.6-9}$$

The alignment line is shown in Fig. 15.3, Fig. 15.4, and Fig. 15.5. The distance from the column face to the alignment line [C1] is:

$$[C1] \geq \gamma_{max} d_b + e_d + \alpha[C3] \tag{15.6-10}$$

where:

α = factor that accounts for overlap of beam flange and cover plates; has a value of 1.0 if the beam flange is wider than the column flange, and a value of 0 if the beam flange width is equal to or less than the column flange width.

[$C3$] = distance from the column face to the edge of the cover plate, in. (mm)

Other terms have been previously defined.

EXTERNAL CONTINUITY PLATES AND COVERPLATES

Step 7. When cover plates are used, size the cover plate-to-column flange weld [W1]. The moment in the plane of the cover plate causes shear demands in the cover plate-to-column flange weld, r_{uv1}. A moment in the plane of the external continu-ity plates causes normal forces in the cover plate-to-column flange weld, r_{un1}.

$$r_{uv1} = \frac{\sum\left(V_{fe} + P_d\right)\left(d_b + t_p\right)}{d_c\left(d_b + 2[C6]\right)} \tag{15.6-11}$$

$$r_{un1} = \frac{\sum\left(V_{fe} + P_d\right)[C5]}{l_{we1}\left(d_c\right)} \tag{15.6-12}$$

where:

l_{we1} = length at each end of the weld between the cover plates and the col-umn flanges that is considered effective for resisting normal forces, in. (mm).

$$= t_p + t_{cp} + [C6] \tag{15.6-13}$$

[$C5$] = distance from the cover plate to the line of bolts on the external conti-nuity plates, in. (mm). See Figure 15.3.

[$C6$] = distance of the cover plate extension above and below the external continuity plates, in. (mm). See Figure 15.3.

The resultant demand is r_{u1}:

$$r_{u1} = \sqrt{r_{uv1}^2 + r_{un1}^2} \tag{15.6-14}$$

Alternatively, weld demands may be determined using the instantaneous center of rotation method.

Check weld strength according to AISC Specification Chapter J.

User Note: For built-up box and HSS columns where the sides of the column function as cover plates, [W1] is not required. For flanged cruciform columns without cover plates, [W1] is not required.

Step 8. Size the external continuity plate-to-cover plate weld [W2]. The applied forces on each external continuity plate result in shear, r_{uv2}, and normal, r_{uv1}, demands on the weld.

$$r_{uv2} = \frac{\sum\left(V_{fe} + P_d\right)}{\left(d_c + 2[C3]\right)} \tag{15.6-15}$$

$$r_{un2} = \frac{\sum\left(V_{fe} + P_d\right)[C5]}{l_{we2}\left(d_c + 2[C3] - 2l_{holdback} - l_{we2}\right)} \tag{15.6-16}$$

where:

l_{we2} = the length at each end of the weld between the external continuity plates and the cover plates that is considered effective for resisting normal forces, in. (mm)

$$= t_{fc} + t_{cp} + [C3] - l_{holdback} \tag{15.6-17}$$

$l_{holdback}$ = the length of the weld holdback, taken as the weld size [W2], in. (mm)

The resultant demand is r_{u2}:

$$r_{u2} = \sqrt{r_{uv2}^2 + r_{un2}^2} \tag{15.6-18}$$

Alternatively, weld demands may be determined using the Instantaneous Center of Rotation Method. Check weld strength according to AISC *Specification* Chapter J.

For flanged cruciform columns, [W2] demands are determined considering $V_{fe} + P_d$ loading on the external continuity plate in both orthogonal directions acting simultaneously.

Step 9. Check external continuity plates for the limit state of tensile rupture in the net section according to AISC Specification Chapter D. Three modes shall be checked, corresponding to rupture through the bolt hole on the alignment line and the next two bolt holes in the direction of the column. For each check, the tension demand is $V_{fe} + P_d$ multiplied by the percentage of bolts that are ahead of the bolt in question, n_p/n_b or $(n_p+1)/n_b$ or $(n_p+2)/n_b$, that have transferred load into the external continuity plate. Check external continuity plates for the limit state of block shear rupture according to AISC Specification Chapter J.

For flanged cruciform columns, continuity plate demands are determined considering $V_{fe} + P_d$ loading on the external continuity plate in both orthogonal directions acting simultaneously.

BEAM

Step 10. The required shear strength, V_u, for the beam and shear tab shall be determined in accordance with Section 2.5.

Check the design shear strength of the beam according to AISC Specification Chapter G.

Step 11. The required block shear strength of the beam flange is:

$$R_{u,bf} = 2(V_{fe} + P_d) \tag{15.6-20}$$

Check design shear strength of the beam flange for the limit state of block shear according to AISC Specification Chapter J.

SHEAR TAB

Step 12. The required number of shear tab bolts, n_n, for the limit state of bolt shear is:

$$n_n \geq \frac{V_u}{\phi r_n} \tag{15.6-21}$$

The required length of the shear tab is:

$$l_{tab} \leq T - 1 \text{ in.} \tag{15.6-22}$$

where:

T = flat distance on the beam web, in. (mm)

1 in. = buffer to ensure shear tab remains clear of the k-area during seismic response, in. (mm)

User Note: The shear tab is as long as possible, without encroaching on the k-area of the beam, to maximize connection stiffness.

Step 13. The required shear strength of the shear tab is V_u (**Step 10**). The required normal strength of the shear tab is:

$$P_u = n_n F_b \tag{15.6-23}$$

where:

F_b = the maximum slip force that can develop in one bolt, kips (N), cal-culated as 1.5 times the nominal slip capacity assuming Class A faying surface.

The required flexural strength of the shear tab is:

$$M_u = V_u [C_1] \tag{15.6-24}$$

where:

$[C_1]$ = distance from the column face to the shear tab bolts, in. (mm)

Design a shear connection for the required strength meeting the requirements of the AISC *Specification*.

Step 14. Determine the shear tab slot dimensions. The shear tab slot dimension [S1] shall accommodate the maximum beam rotation from Step 6, for the critical case where the center of rotation is assumed at the top flange level.

$$[S1] \geq \gamma_{\max} \left[\frac{d_b}{2} + \frac{l_{tab}}{2} - e_d \right] + \frac{d_{bolt}}{2} \tag{15.6-25}$$

where:

d_{bolt} = diameter of the shear tab bolt, in. (mm)

e_d = edge distance at bottom of shear tab, in. (mm)

l_{tab} = length of the shear tab, in. (mm)

γ_{max} = maximum beam rotation considered for design, rad

$[S1]$ = distance from alignment line to inside edge of slot, in. (mm); see Figure 15.4

The slot dimension [S2], from the alignment line to the outside edge of the slot, is typically the same as [S1].

$$[S2] = [S1] \tag{15.6-26}$$

TOP PLATE

Step 15. The required shear strength of the top plates, $V_{u,tp}$, is:

$$V_{u,tp} = V_{fe} + P_d \tag{15.6-27}$$

Check the top plate for shear yielding and shear rupture according to AISC *Specification* Chapter J.

User Note: If the top plate has insufficient shear capacity, the plate can be thickened, the bolt size can be reduced, or the bolt spacing can be increased.

Step 16. The required tensile strength of the top plates in the narrow portion is:

$$R_{u,tpn} = \left(\frac{n_m}{n_b}\right)\left(V_{fe} + P_d\right) \tag{15.6-28}$$

Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion according to AISC *Specification* Chapter D.

Step 17. Check that [P2] is sufficient for entering and tightening requirements:

$$[P2] \geq K_1 + d_h \tag{15.6-29}$$

where:

$K1$ = flange fillet dimension, in. (mm)

d_h = hole diameter used for rupture limit state calculations, in. (mm)

Check that [P2] is sufficient to preclude yielding of the top plate due to combined flexure and shear:

$$[P2] \geq [P10] - \frac{\left(0.9F_{yp}\right)e\left(m+e\right)t_p}{V_{fe} + P_d} \tag{15.6-30}$$

where:

$$m = \frac{V_{fe} + P_d}{0.9\left(0.6F_{yp}\right)t_p} \tag{15.6-31}$$

$$e = \frac{[P8] - m}{2} \tag{15.6-32}$$

$[P8]$ = inside length of the top plate, in. (mm); see Figure 15.6

FUSE PLATE

Step 18. The width of the yielding regions in the fuse plates [F6] shall not exceed 4.0 and shall not be less than 1.5 in. The width-thickness ratio, [F6]/[T2], of the yielding region in the fuse plates shall not exceed 4.25 and shall not be less than 1.5.

The required strength of the fuse plate ahead of the first yielding region (YR) is:

$$R_{u,fp} = 2\left(\frac{n_m}{n_b}\right)\left(V_{fe} + P_d\right) \tag{15.6-33}$$

Check the fuse plate for the limit states of tensile yield and tensile rupture in the net section ahead of the first yielding region (YR) according to AISC *Specification* Chapter D.

User Note: The expected tensile strength ($R_{tp}F_{up}$) of the plate may be used when calculating the capacity for rupture of the net section.

Check the fuse plate for edge distances according to AISC *Specification* Chapter J.

Step 19. Determine the yielding region (YR) depth, [F2]. The YR depth is determined such that the maximum force that will develop in the strain-hardened regions on one side of the fuse plate will not exceed Vfe . The expected strain-hardening in the fuse is a function of the width/depth ratio and the width/thickness ratio. The formula for [F2] that accounts for strain hardening is:

$$\left[F2\right]=\frac{.95V_{fe}+2t_p\left(0.6R_{tp}F_{up}\right)B\left[F6\right]}{2t_p\left(0.6F_{up}R_{tp}\right)\left[A-\frac{C\left[F6\right]}{t_p}\right]} \tag{15.6-34}$$

where:

A = 1.52, a coefficient in the strain hardening expression, calibrated from experiments,

B = 0.16, a coefficient in the strain hardening expression, calibrated from experiments,

C = 0.09, a coefficient in the strain hardening expression, calibrated from experiments,

The width-depth ratio of the yielding regions in the fuse plates, [F6]/[F2], shall not exceed 1.25 and shall not be less than 0.5.

Step 20. The required tensile strength of the fuse plates in the narrow extensions is:

$$R_{u,fpn}=\left(\frac{n_m}{n_b}\right)\left(V_{fe}+P_d\right) \tag{15.6-35}$$

Check the fuse plate for the limit states of tensile yield and tensile rupture in the net section of the narrow extension according to AISC *Specification* Chapter D.

User Note: The expected tensile strength, $R_{tp}F_{up}$, of the plate may be used when calculating the capacity for rupture of the net section for the fuse plate extension.

CHAPTER 15 - COMMENTARY

DURAFUSE FRAMES CONNECTION

15.1 GENERAL

The DuraFuse Frames connection is a field-bolted connection with a replaceable fuse plate below the beam bottom flange. The fundamental seismic behaviors expected with the DuraFuse Frames connection include:

(1) Shear yielding in particular regions of the replaceable fuse plate, followed by strain hardening at larger deformations.
(2) Slip of the fuse plate bolts and top plate bolts, which occurs at a similar resistance level to the initial yielding.

The combination of bolt slip and fuse plate yielding results in large inelastic deformation capacity for the DuraFuse Frames connection while preventing yielding in the beam and column.

Prequalification of the DuraFuse Frames connection is based on full-scale tests performed per Chapter K of the AISC *Seismic Provisions* (AISC, 2016a). Ten qualifying tests were performed on connections with beams ranging in depth from W21 to W40 at the University of California, San Diego (Reynolds and Uang, 2018; Reynolds and Uang, 2019). The deformation capacity varied depending on the depth of the beams, and ranged from 0.04 rad to 0.08 rad, with shallower beams having greater capacity. I-shaped and box columns were included in the testing program. Eight other full-scale qualifying tests are reported in Oh and Richards (2018). For reference, reduced scale prototype tests are reported in Oh and Richards (2019).

The stiffness of the DuraFuse Frames connection is sufficient to be considered fully restrained (FR). The stiffness of experiments and finite element models have been compared with the stiffness of 2D centerline models where the flexibility of the panel zone is explicitly considered and the connection is replaced by a rotational spring with a stiffness of $18EI/L$. The experimental and FE stiffness have been equal-to or greater-than that of the centerline models, justifying the classification as a fully rigid (FR) connection.

15.2 SYSTEMS

The DuraFuse Frames connection meets the qualification requirements for both SMF and IMF frames. However, no test data are available for the DuraFuse Frames connection with composite slabs, so prequalification is restricted to the case where the concrete slab has a minimum separation or isolation from the column. In general, isolation is achieved if the slab is separated from the column face by an open gap or by use of compressible material.

15.3 PREQUALIFICATION LIMITS

1. Beam Limitations

The DuraFuse Frames connection has been investigated with a variety of beam sizes. The specimens with the smallest beam size [W14×38 (W360×64)] were early prototype tests and had rectangular fuse plates. The smallest beam size that has been tested with fuse plates with extensions, similar to Fig. 15-1, is W21×50 (W530×74). On the bigger side, more than ten tests have been performed with W33 (W840), W36 (W920), and W40 (W1000) beams. The deepest beam that has been tested is W40×167 (W1000×249). The heaviest beam that has been tested is W36×232 (W920×345).

Although the AISC *Seismic Provisions* permit limited increase in beam depth compared to the maximum sections tested, the prequalification limit for maximum beam depth was not extrapolated beyond the test data for W40×167 (W1000×249) to be consistent with other connections in AISC 358. The strain demands on the fuse plate are directly related to the beam depth, so it is conservative to base the beam depth prequalification limit on the maximum tested specimen.

The prequalification limit for beam weight, 309 lbs/ft (464 kg/m), was established based on the weight increase permitted in the AISC *Seismic Provisions* (1.33 times the heaviest beam tested). Connections with beams that have thicker flanges also have thicker fuse plates and external continuity plates. Experiments have not shown negative consequences of thicker plates.

Since there is no beam yielding in the DuraFuse Frames connection, it is not necessary for the beam to satisfy the width-thickness ratios given in the AISC *Seismic Provisions*. Several tests have been performed with beams with higher width-thickness ratios than λ_{hd}. The greatest flange width-thickness ratio that has been tested is 7.80 [W30×99 (W760×147)]. The greatest web width-thickness ratio that has been tested is 51.9 [W30×99 and W36×150 (W760×147 and W920×223)]. The requirement that the beam flange width-thickness ratio should not exceed λ_p is deemed conservative because no plastic hinge formation is expected in the beam.

All of the experiments for the DuraFuse Frames connections have demonstrated good performance without the presence of lateral bracing near the beam-column connection. The only beam restraint in the tests was at the beam tip. The DuraFuse Frames connection is much less susceptible to beam lateral-torsional buckling or column twisting because the beam does not yield and there is no local buckling in the fuse plate to disrupt symmetry of the load path at the connection. Thus, the beam stability bracing can be determined using the AISC *Specification*, neglecting the prescriptive requirements for SMF that assume beam yielding.

2. Column Limitations

The DuraFuse Frames connection has been investigated with a variety of column shapes and sizes. The W14 (W360) columns that have been tested are: W14×38 (W360×57.8) and W14×68 (W360×101). The W21 (W530) column that has been tested is W21×132 (W530×196). The W24 (W610) columns that have been tested are: W24×103 (W610×153) and W24×250 (W610×372). The W36 (W920) columns that have been tested are: W36×150 (W920×223) and W36×231 (W920×344). The box columns that were tested were 24 in. (610 mm) deep, 17 in. (432 mm) wide, with a plate thickness of 1.75 in. (45 mm). The connection details for box columns and HSS columns are the same, so testing of box columns also represents HSS columns. The prequalification limit for column depth for DuraFuse Frames connections is based on the maximum tested specimen, even though the AISC *Seismic Provisions* permit an 11% increase in depth beyond what has been tested.

As with most other prequalified connections, the DuraFuse Frames moment connection has not been tested with built-up I-shaped or cruciform column sections. However, the CPRP has deemed such columns to be similar enough to rolled shapes to justify prequalification.

3. Plate Limitations

All of the tests that have been performed for the DuraFuse Frames connection have used plate material that satisfies ASTM A572/A572M Gr. 50 for the cover plates, fuse plates, top plates, and external continuity plates. In the absence of experimental data for other plates, prequalification is limited to plates that satisfy ASTM A572/A572M Gr. 50.

The fuse plates have the additional requirement that the material not have a tensile strength greater than 85 ksi. Since there is no upper limit on the tensile strength of ASTM A572/A572M Gr. 50 steel in the ASTM specifications, it is prudent to have some protection against steel that is too strong in the fuse. The strongest steel that was used for fuses in testing had a mill certified tensile strength of 86.5 ksi.

A variety of geometries for the yielding regions (YR) of the fuse plates have been investigated and found to provide sufficient deformation capacity. The limits on the thickness and width of the YR are based on the dimensions of the YR that have been tested. Fig. C-15.1 shows the range of yielding region (YR) width and depths from the qualification tests at UCSD and BYU. The limits for the width [not greater than 4 in. (102 mm) and not less than 1.5 (38 mm)] are based on the limits of what has been tested, with some extension because excess rotation capacity was observed for all specimens where fuse yielding/fracture was the governing limit state.

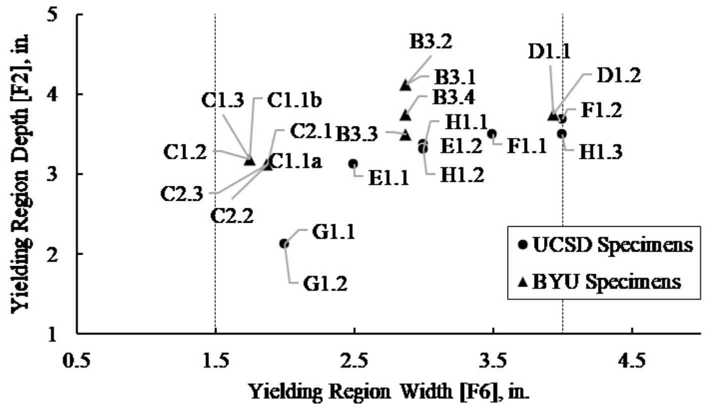


Fig. C-15.1 – Yielding region widths and depths that have been tested.

Fig. C-15.2 shows the YR width-thickness ratios from the qualification tests at UCSD and BYU. The limits for the width-thickness ratio (not greater than 4.25 and not less than 1.5) are based on the limits of what has been tested, with some extension because excess rotation capacity was observed for all specimens where fuse yielding/fracture was the governing limit state.

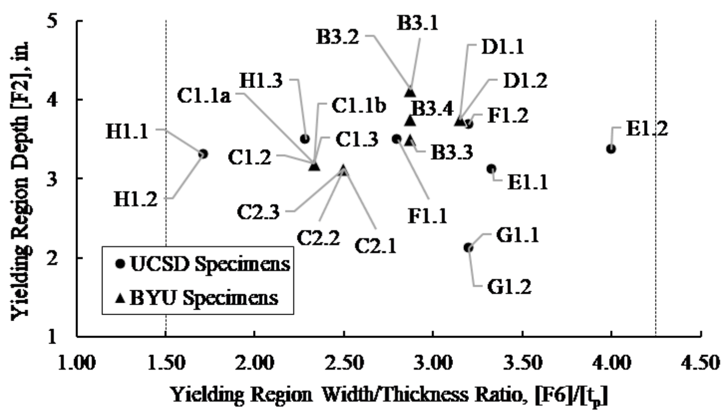


Fig. C-15.2 – Yielding region width/thickness ratios that have been tested.

Fig. C-15.3 shows the YR width-depth ratios from the qualification tests at UCSD and BYU. The limits for the width-depth ratio (not greater than 1.25 and not less than 0.5) are based on the limits of what has been tested, with some extension because excess rotation capacity was observed for all specimens where fuse yielding/fracture was the governing limit state.

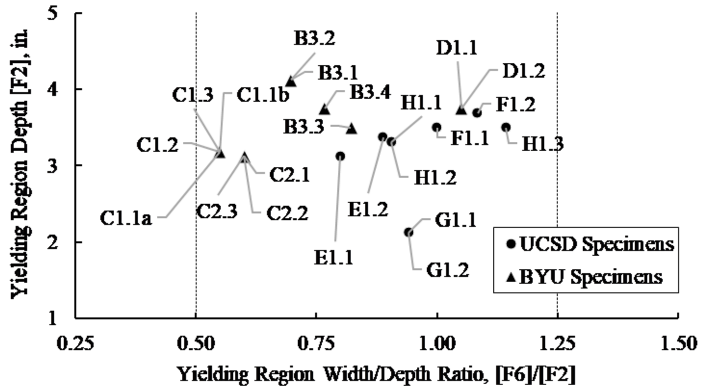


Fig. C-15.3 – Yielding region width/depth ratios that have been tested.

15.5 CONNECTION DETAILING

For most of the specimens that were tested, bolt holes in the beam and external continuity plates were drilled, and bolt holes in the fuse plates were cut with a plasma cutter. The plasma cut holes were left rough and some did not meet AISC *Specification* surface requirements. Good performance was observed even when rough holes were present.

The specific bolts that were used in qualification testing were: 7/8 in. (22 mm) diameter ASTM F3125 Grade F1852; 1 in. (25 mm) diameter ASTM F3125 Grade F2280; 1-1/8 in. (29 mm) diameter ASTM F3125 Grade F2280; and 1-1/4 in. (38 mm) diameter ASTM F3125 Grade A490/A490M.

Many of the qualification tests involved a column assembly that had been used for previous tests, and that had external continuity plates that were bent. In most cases, the bent plates were not straightened prior to the installation of the fuse plate, and gaps of 0.5 in (13 mm) or more between the fuse plate and external continuity plates were closed without providing shims, with the fuse plate and external continuity plates bending during bolt tightening. These tests exhibited acceptable rotation capacity, with multiple cycles beyond the minimum required for prequalification. The ¼ in. (6 mm) limit on gap is given to be consistent with the Bolted Flange Plate connection.

15.6 DESIGN PROCEDURE

The design procedure for DuraFuse Frames connections results in fully restrained connections where the beams and columns remain essentially elastic during seismic events.

With the DuraFuse Frames connection, the maximum probable moment at the column face is governed by the geometry of the fuse plate. The dimensions of the yielding regions on the fuse plate, which are determined in the final steps of the design procedure, are selected so that when the fuse plate has yielded and fully strain-hardened, the moment at the face of the column, M_{pr}, will not exceed M_p of the beam.

This approach differs from most other special moment connections where the plastic hinge capacity of the beam is determined first and then extrapolated to the column face. The procedure is different for the DuraFuse Frames connection because a plastic hinge never forms in the beam. The beam does not yield at the face of the column, even though the moment at the face of the column may reach M_p, because the beam is reinforced by the top plates and fuse plates in the connection region.

The force determined in **Step 2** is the maximum force that is transmitted to each of the external continuity plates at the bottom flange level.

It is the intent of the design criteria to provide a fully restrained (FR) connection. In **Step 2**, if M_{pr} is less than M_p, analysis is required to justify that the connection is FR. The stiffness of the connection, K_s, may be computed using the following equations, which are derived based on the stiffness of the individual connection plates. Alternatively, other methods or FE models may be used to quantify connection stiffness.

$$K_s = \frac{(d_b + [T2])2}{\left(\frac{1}{K_{top}} + \frac{1}{K_{bot}}\right)} \quad \text{(C 15.6-1)}$$

Where

$$K_{top} = \frac{(2P + M - 1)s[T2]G}{[P10] - [P2]} \quad \text{(C 15.6-2)}$$

$$K_{bot} = \frac{2}{\left(\frac{1}{K_1} + \frac{1}{K_2} + \frac{1}{K_3} + \frac{1}{K_4}\right)} \quad \text{(C 15.6-3)}$$

where

G = shear modulus of elasticity of the plate, ksi (MPa)

$$K_1 = \frac{(2[F2] + [F3])[T2]G}{F4} \quad \text{(C 15.6-4)}$$

$$K_2 = \frac{2[F2][T2]G}{[F6]} \quad \text{(C 15.6-5)}$$

$$K_3 = \frac{2(0.37)E[T2][F2]^3}{[F6]^3} \quad \text{(C 15.6-6)}$$

$$K_4 = \frac{(2[F2] + [F3])[T2]G}{[F8]} \quad \text{(C 15.6-7)}$$

The cover plate design in **Step 3** is similar to the design of doubler plates in conventional special moment frame connections. The cover plates are designed so that they will remain elastic under the maximum forces and they are designed with the same local buckling criteria as traditional doubler plates.

The maximum bolt size requirement in **Step 4** is the same as the requirement for the Bolted Flange Plate and Double-Tee connections, even though a plastic hinge is not expected to form in the beam of the DuraFuse Frames connection.

The required number of bolts in **Step 5** is governed by bolt bearing strength. Similar to other prequalified bolted connections, the bolts in the DuraFuse Frames connection are pretensioned, but the connection is not slip-critical.

The bolts on each line are divided into m-bolts and p-bolts in **Step 5** because there may be different spacing between the bolt on the alignment line and the next bolt in the column direction (see Figure 15.2). It is also convenient to have them differentiated into two groups for some demand calculations (see Step 9 for example). The total number of bolts on each line is the sum of the m-bolts and the p-bolts.

The maximum beam rotation considered in design is set by the engineer. In **Step 6**, a minimum value of 0.06 rad ensures good behavior beyond the 0.04 drift limit. The design calculation for the gap between the beam and the column is based on the expected movement of the beam bottom flange towards the column, assuming a center of rotation at the beam top flange. Assuming the center of rotation at the top flange is conservative because in testing, bolt slip of the top bolts has caused the center or rotation to be somewhere between the top flange and the mid-depth of the beam.

When the center of rotation is between the top flange and the mid-depth of the beam, rather than at the top flange level, there is less movement of the beam bottom flange for a given rotation angle.

In **Step 7**, the welds between the cover plates and the column flanges are designed based on the demands shown in Figure C-15.4. Only part of the weld is considered effective for resisting normal forces [Fig. C-15.4(c)] because the normal forces are being delivered at the external continuity plate levels.

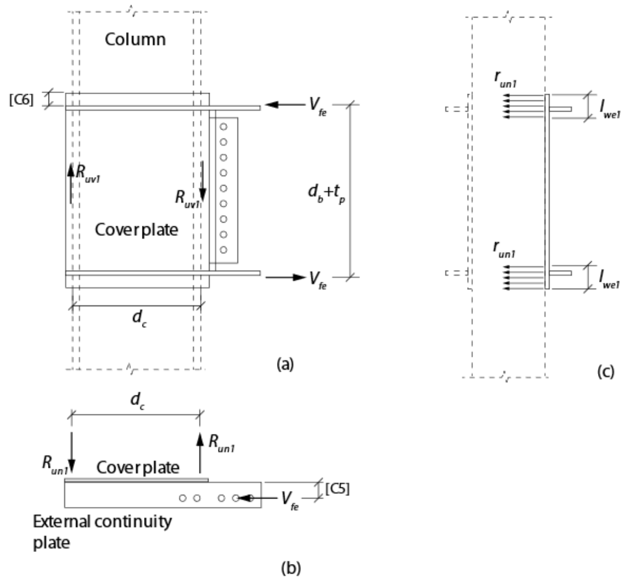


Fig. C-15.4 – Weld demands for cover plate welds.

In **Step 8**, the welds between the external continuity plates and cover plates are designed based on the demands shown in Fig. C-15.5. Only part of the weld is considered effective for resisting normal forces because the load paths near the column flanges are much stiffer for normal forces than the paths through the center of the cover plate.

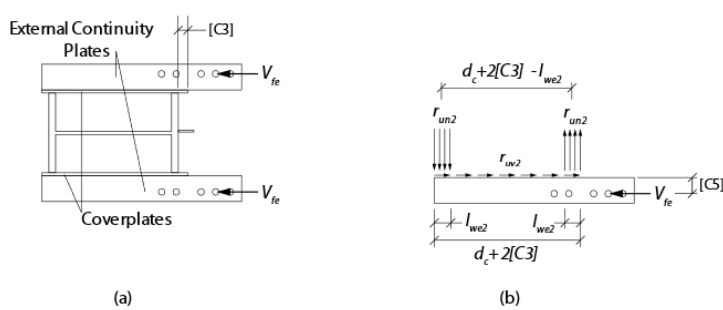


Fig. C-15.5 – Weld demands for external continuity plate welds.

If normal demands are resulting in large welds, bridge plates (Figure 15.3) can be added to the connection to assist in transmitting the normal force.

The limit states that are checked in **Step 9** are illustrated in Figure C-15.6 and Figure C-15.7 The Mode 2 and Mode 3 limit states shown in Figure C-15.6 are outside the definition of "rupture in the net section" because they overlap the welded portion of the external continuity plate. However, Mode 2 and Mode 3 are conservatively checked in addition to the block shear limit state (Figure C-15.7) since they sometimes give lower capacity.

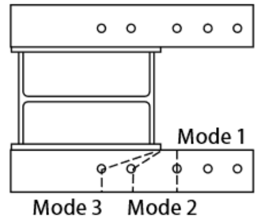


Fig. C-15.6 – Tension rupture limit state for external continuity plate.

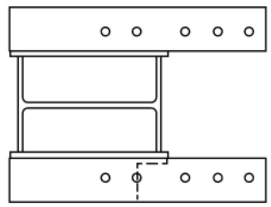


Fig. C-15.7 – Block shear limit state for external continuity plate.

In **Step 13**, the shear tab has a required normal strength even though the bolts are slotted in the normal direction. The required normal strength corresponds to the maximum probable slip force that can develop in the bolts.

Step 15 is checking the limit state illustrated in Fig. C-15.8, and the similar mode for the fuse plate. Since the top plates may be from a different heat than the fuse plate, the R_t factor is not used for top plate checks and greater shear area is required for the top plates than the fuse plate.

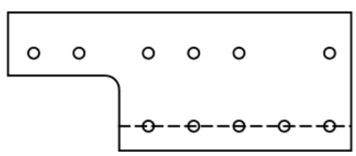


Fig. C-15.8 – Shear rupture of net section on top plate.

Step 16 is checking the limit states illustrated in Figure C-15.9.

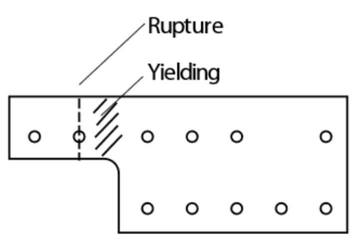
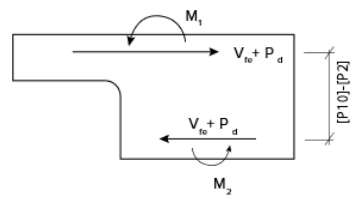


Fig. C-15.9 – Tension rupture of net section on top plate.

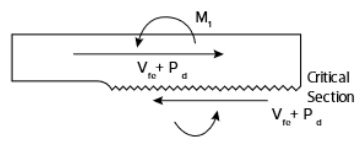
Fig. C-15.10 shows the lower-bound model that is used in **Step 17** to evaluate the capacity of the top plate to resist the combined shear and bending caused by $V_{fe} + P_d$ and the eccentricity [P10]-[P2]. The lower-bound model assumes the shear forces are resisted by the "m" region, and that the bending moment is resisted by the "e" regions. This capacity check is translated into a minimum required value for [P2] that appears in **Step 17**.



$$M_1+M_2=(V_{fe}+P_d)([P10]-[P2])$$

For plate check, conservatively assume $M_2 = 0$

$$M_1=(V_{fe}+P_d)([P10]-[P2])$$



Conservatively assume $M_3=M_1$

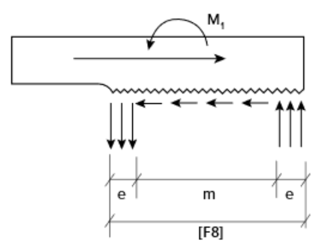


Fig. C-15.10 – Lower bound model to check top plate yield strength for combined shear and flexure.

In **Step 18**, checks are made for limit states that are related to the dimension [F6]. Fig. C-15.11 illustrates the net section that is being checked.

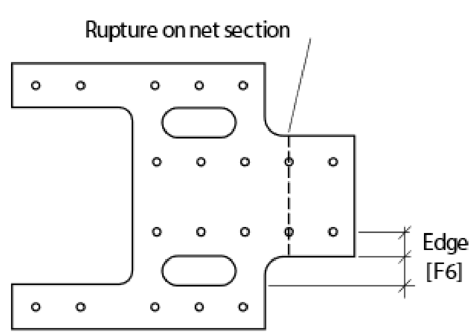


Fig. C-15.11 – Limit states related to dimension [F6].

In **Steps 18** and **19**, the dimensions of the yielding regions (YR) on the fuse plates are determined and compared with the dimensions that have been investigated experimentally.

In **Step 19**, the equation for determining [F2] appears complicated because it incorporates an expression for strain hardening that depends on the geometry of the yielding region. The cyclic shear hardening factor, C_h, for the yielding regions of the fuse plates has been calibrated from the various qualification experiments. This special hardening factor quantifies an increase in ultimate shear stress beyond what is calculated by $R_t(0.6F_u)$.

The C_h factor is comparable, in some respects, to the C_{pr} factor used for other moment connections, but differs in that it is used in conjunction with the expected ultimate stress ($R_t F_u$) rather than in conjunction with the expected yield stress ($R_y F_y$). The cyclic shear hardening factor depends on the width/depth ratio of the yielding region but is also influenced by the width/thickness ratio. The cyclic shear hardening factor that is incorporated into the design equations is:

$$C_{h,des} = A - B\left(\frac{[F6]}{[F2]}\right) - C\left(\frac{[F6]}{t_p}\right) \quad \text{(C-15.6-1)}$$

where A, B, and C are calibrated factors: A=1.52, B = 0.16, and C = 0.09.

Table C-15.1 below compares the cyclic shear strain hardening observed in experimental testing, $C_{h,exp}$, with values of $C_{h,des}$ from the calibrated equation for the specimens where the testing was continued all the way through fuse plate tearing. The values for $C_{h,des}/C_{h,exp}$ in Table C-15.1 indicate that the calibrated equation for $C_{h,des}$ is reasonable, although it is quite conservative for Specimen D1.1.

Table C-15.1. Cyclic Shear Hardening Factor from Experiments and Design Equation

Specimen	w/d	w/t	$C_{h,exp}$	$C_{h,des}$	$C_{h,des}/C_{h,exp}$
B3.1	0.697	2.875	1.16	1.15	1.00
B3.2	0.697	2.875	1.17	1.15	0.99
B3.3	0.821	2.875	1.08	1.13	1.05
B3.4	0.767	2.875	1.13	1.14	1.01
C2.1	0.600	2.5	1.13	1.20	1.06
D1.1	1.050	3.15	0.93	1.07	1.15
E1.2	0.890	4	1.03	1.02	0.99
F1.2	1.080	3.2	0.99	1.06	1.07
G1.2	0.940	3.2	1.07	1.08	1.01
H1.3	1.140	2.29	1.15	1.13	0.98

The limit states being checked in **Step 20** are illustrated in Fig. C-15.12.

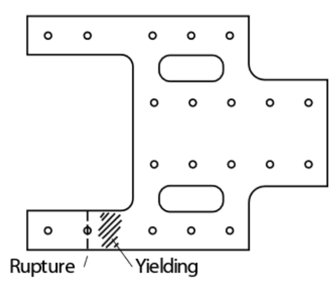


Fig. C-15.12 – Fuse fracture limit states that are checked in Step 20 of the design procedure.

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