DEPARTMENT OF HEALTH CARE ACCESS AND INFORMATION
FACILITIES DEVELOPMENT DIVISION

APPLICATION FOR PREAPPROVED PREFABRICATED COMPONENTS AND SYSTEMS

OFFICE USE ONLY
APPLICATION #: PCS- 0004

HCAI Preapproved Prefabricated Components and Systems (PCS)

Type: ☒ New ☐ Renewal

Manufacturer Information

Manufacturer: DuraFuse Frames connections are fabricated by the shop that is approved
Manufacturer's Technical Representative: to do the fabrication for the rest of the particular steel structure.
Mailing Address:
Telephone: Email:

Product Information

Product Name: DuraFuse Frames
Product Type: Steel Special Moment Frame Connection
General Description: A proprietary special moment frame connection that incorporates a fuse plate to prevent beam and column damage during severe earthquake shaking.

Applicant Information

Applicant Company Name: DuraFuse Frames, LLC
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I hereby agree to reimburse the Department of Health Care Access and Information review fees in accordance with the 2022 California Administrative Code.

Signature of Applicant: Date: 8/3/2023
Title: Vice President, R&D Company Name: DuraFuse Frames

Registered Design Professional Preparing Engineering Report

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Disciplines Involved

■ Structural ☐ Architectural ☐ Mechanical ☐ Electrical ☐ Plumbing ☐ Fire Life Safety

OFFICE USE ONLY – HCAI APPROVAL

Signature: Kamal Date: 9/18/2023

Print Name: Kamalpreet Kalsi

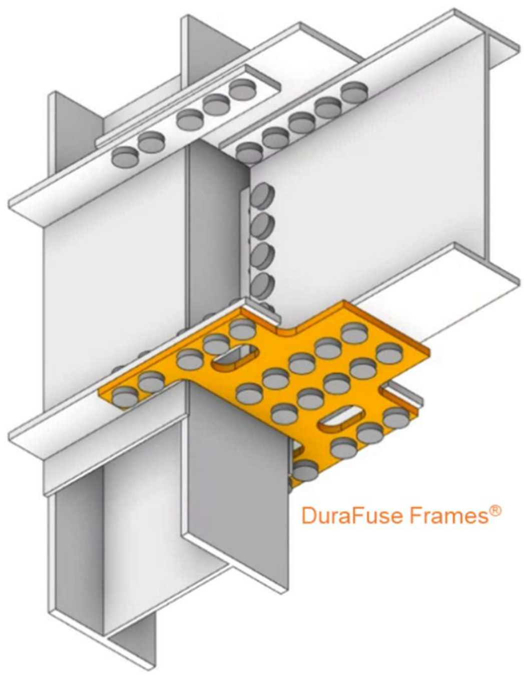
Title: Senior Structural Engineer

Approved Version Number v1.0

Version History

v1.0 September 18, 2023 - Original Approval

DuraFuse Frames® (DFF)
HCAI Plan Review Guide



Date: 09/13/2023

Version: 1.0

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1 Limits of Pre-Approval

The DuraFuse Frames (DFF) connection in accordance with 2022 California Building Code (with HCAI amendments) and AISC 358-22 Chapter 15, shall be permitted under the following additional conditions:

a. A linear analysis procedure shall be used for design of the SMF systems using the DFF when permitted in accordance with ASCE 7.
b. The biaxial dual-strong axis and column minor axis configurations of the moment connection shall be considered as an alternative system.
c. DFF connection bolts shall not slip under wind design demand loads. The connection shall be designed to prevent slip using AISC 360 Equation J3-4, where the slip resistance is taken to be 0.3. Story drift limitations per 2022 CBC § 1609A.1.2 shall also be met.
d. Beam flange width-to-thickness ratio shall not exceed λ_p of 0.38*sqrt(E/F_y) per AISC 360 Table B4.1b Case 11.
e. The beam weight shall be limited to a maximum of 232 lbs/ft.

2 Connection Description

2.1 Overview

The DuraFuse Frames (DFF) connection is prequalified in AISC 358-22 for the use in steel special moment frames (SMF). The DFF 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 [Fig. 1(a) and (b)]. For box or HSS columns, the sides of the column may function as the cover plates [Fig. 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 (Fig. 1). The beam flanges and external continuity plates have standard holes, while the top plates and fuse plate have short-slotted or 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 (Fig. 2). The fuse plate is bolted in place, so that it could be removed and replaced following a severe earthquake. The top plates are intended to experience minimal yielding, such that they would not require repair following a severe earthquake. The various plates in the connection are proportioned such that the beam and column remain essentially elastic.

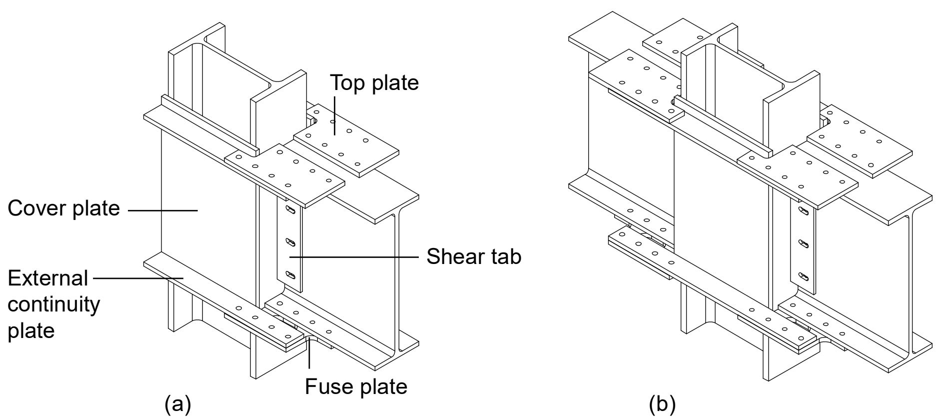


Fig. 1. DuraFuse Frames (DFF) connections (a) one-sided with wide-flange beam and column (b) two-sided with wide flange beams and column.

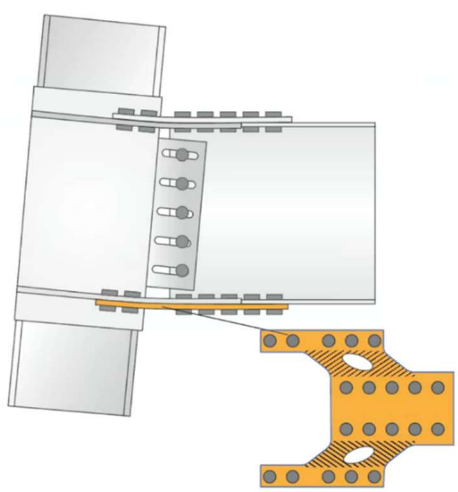


Fig. 2 Inelastic response of a DuraFuse connection with yielding confined to the fuse plate.

2.2 Details and Nomenclature

Figures 3 through 7 define the symbols for various plate thicknesses, dimensions, weld sizes, and bolt quantities that are pertinent to the connection and discussed in the design guide.

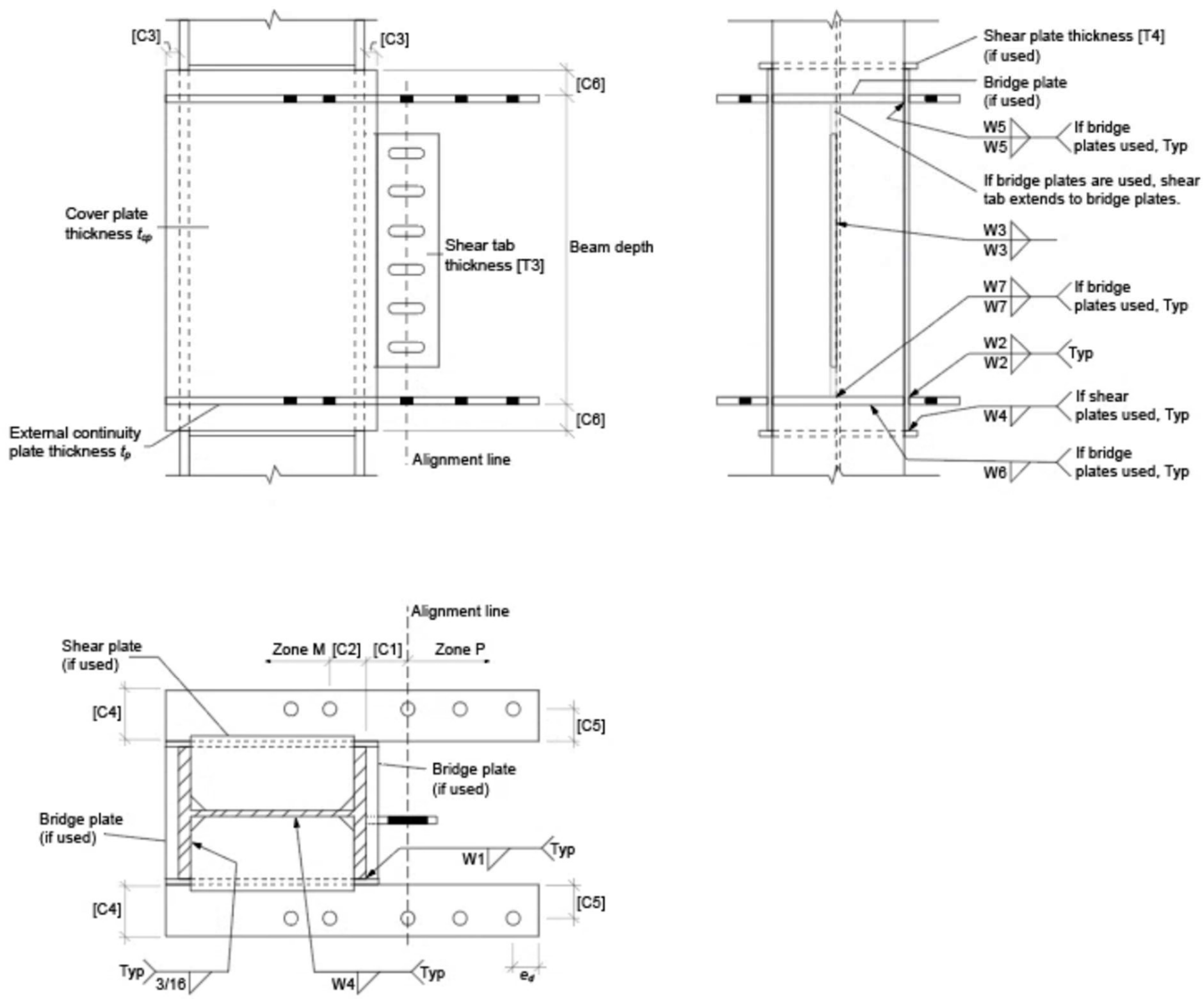


Fig. 3. Column assembly detail.

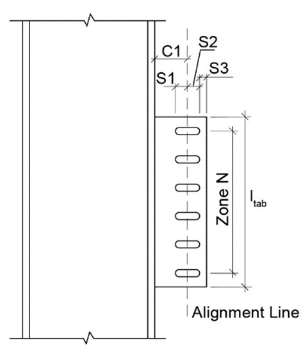


Fig. 4. Shear tab detail.

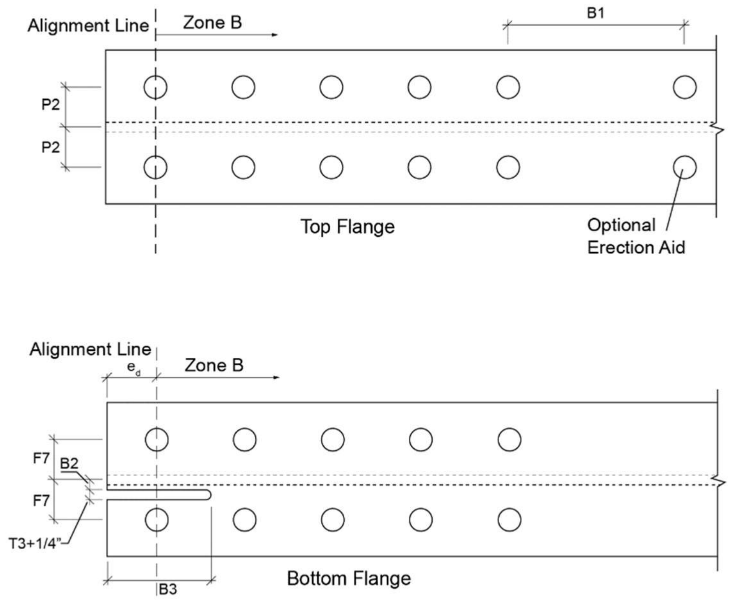


Fig. 5. Beam end detail.

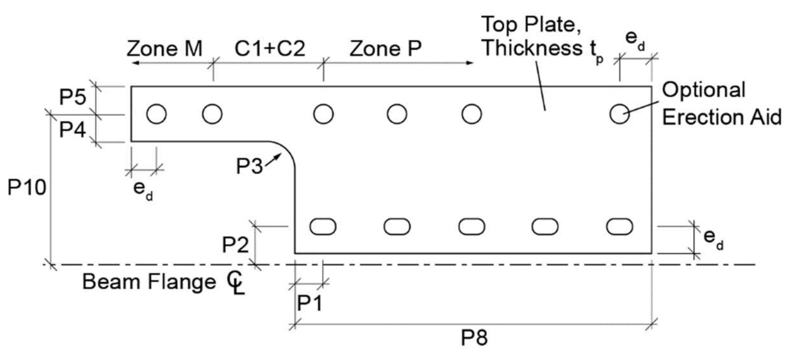


Fig. 6. Top plate detail.

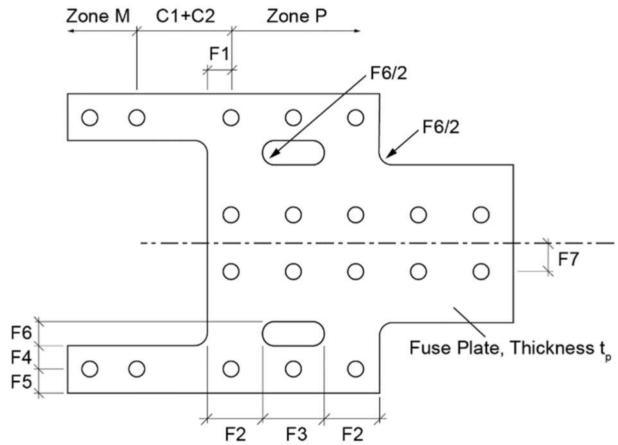


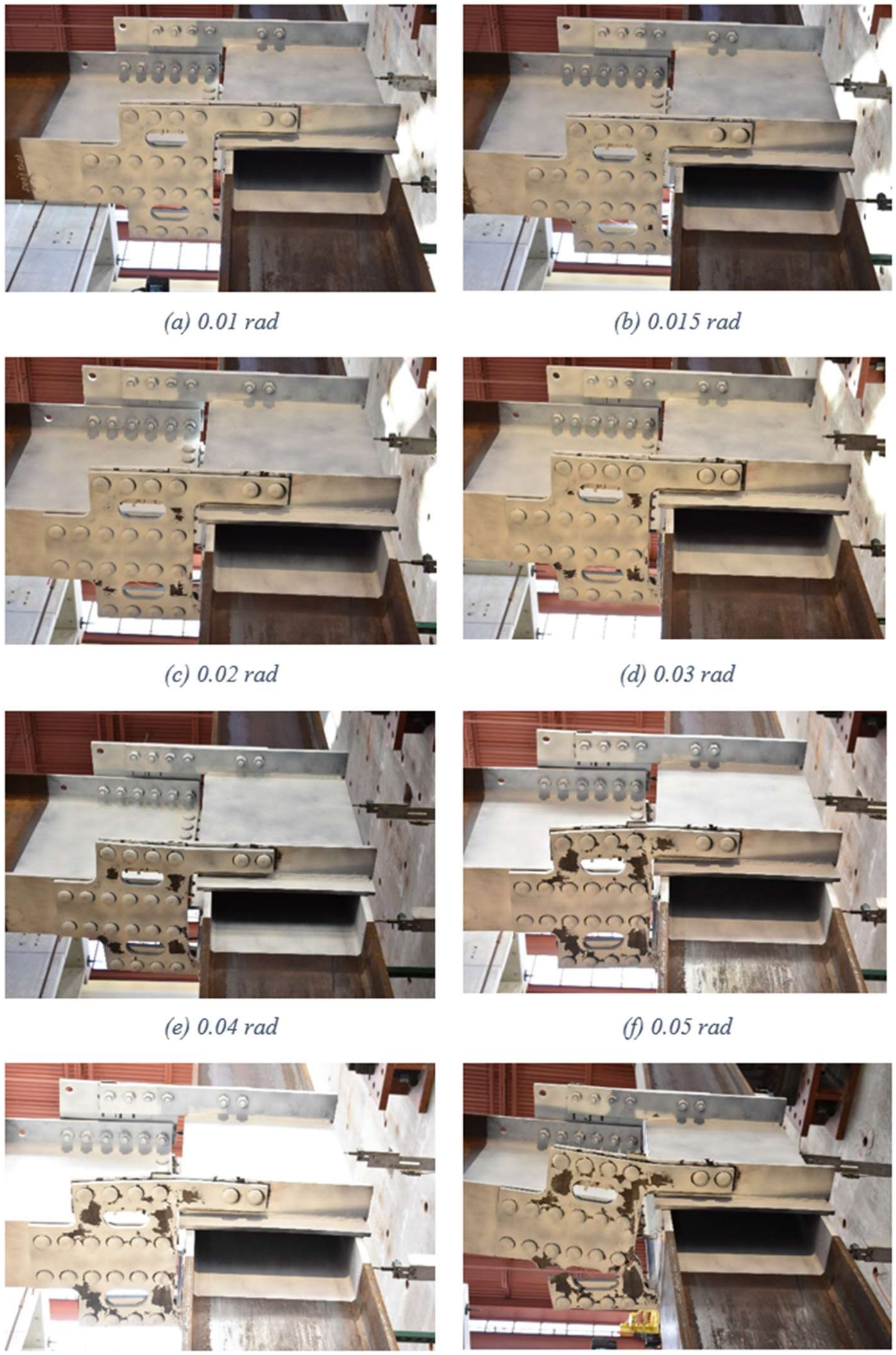
Fig. 7. Fuse plate detail.

2.3 Connection Behavior

The behavior of DuraFuse Frames (DFF) connections has been established through several series of experiments (Reynolds and Uang 2019a; Reynolds and Uang 2019b) that have been published in peer-reviewed journals (Richards 2019; Richards 2021; Richards 2022; Richards and Oh 2019).

Figure 8 shows a connection test with typical behavior [Specimen E1.2 (Reynolds and Uang 2019a)]. The connection remained elastic throughout the 0.00375, 0.005, 0.0075 rad cycles. During the 0.01 rad cycles, slight flaking of the mill scale indicated localized yielding in the yield regions [Fig. 8(a)]. Bolt slip occurred during the 0.015 rad and subsequent cycles. During the 0.02, 0.03, 0.04 rad cycles, inelastic deformations of the fuse plate became more pronounced [Fig. 8(c)-(e)]. The external continuity plates had noticeable curvature at 0.05 rad drift [Fig. 8(f)] but were still primarily elastic (the plates were essentially straight after testing). During the second cycle at 0.06 rad, ductile tearing of the fuse plate initiated [Fig. 8(g)], and during the first excursion to 0.07 rad, the west side of the fuse plate tore through [Fig. 8(h)].

The hysteretic response of DFF connections is similar to other bolted SMF connections, except there is higher post-yield stiffness and no strength degradation at large drifts. Fig. 9 shows the hysteretic response of the connection from Fig. 8. As with other bolted SMF connections, the DFF hysteretic plots have a flatter region in the middle of each large cycle corresponding to bolt slip (Fig. 9). Once bolts returned to bearing, the strength continues to climb. One difference, as compared to other bolted SMF connections is the lack of strength degradation at large drifts. SMF connections that form plastic hinges in the beam have strength degradation after 0.03 or 0.04 rad due to flange and web local buckling of the beam in the plastic hinge region (Uang and Fan 2001). The DFF connections do not experience flange local buckling or local buckling in the fuse plate and maintain flexural strength through large drift cycles until the fuse plate fatigues. Even when the fuse plate tears and the flexural strength of the connection is reduced, the shear strength of the connection remains intact to carry gravity loads. The DFF fuse plates are proportioned so that the ultimate moment at the column face is roughly equal to the nominal moment capacity of the beam (Fig. 9).



(a) 0.01 rad *(b) 0.015 rad*

(c) 0.02 rad *(d) 0.03 rad*

(e) 0.04 rad *(f) 0.05 rad*

(g) 0.06 rad *(h) 0.07 rad*

Fig. 8. DFF connection behavior [Specimen E1.2 (Reynolds and Uang 2019a)].

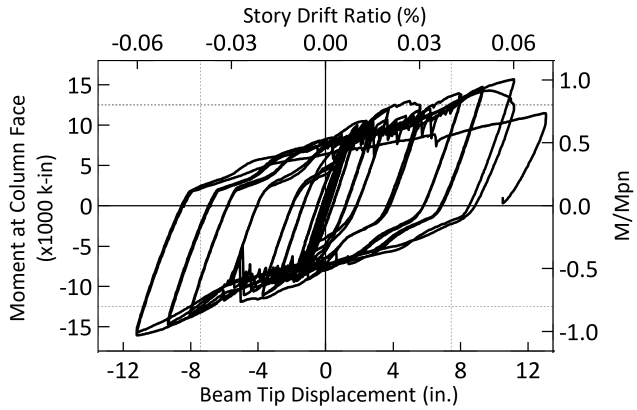


Fig. 9. DFF hysteretic behavior [Specimen E1.2 (Reynolds and Uang 2019a)].

2.4 Design Responsibility

The DuraFuse Frames connection is a proprietary connection used under the supervision of the EOR. Licenses to use the connection are granted on reasonable and nondiscriminatory terms, with the license fee being included in the fabrication bid and paid by the fabricator. When DFF connections are used on a project, the SMF connections are designed by DuraFuse Frames, LLC, based on the member sizes and design loads from the EOR (structural model or other calculations). The connection calculations and connection details/schedules are prepared by DuraFuse Frames, LLC and included in the overall structural design drawings and calculation package assembled by the EOR.

The DFF general specification states: “DuraFuse Frames engineering design responsibilities are limited to design of the beam-to-column connections in the moment frames.”

3 Design Methodology

3.1 Overview

The design of DFF SMF consists of selecting beam and column sizes to meet drift and strength requirements (per ASCE 7) while satisfying seismic provision checks (per AISC 341-16 and 358-22), and then proportioning the connection plates, welds, and bolts, to meet strength requirements and ensure that fuse yielding is the governing limit state (per AISC 360-16and 358-22). The design procedure (AISC 358-22 Section 15.6) addresses these various requirements throughout twenty design steps.

3.2 Drift Checks

Since SMF are inherently flexible, drift requirements often govern beam and column sizes in DFF SMF. When using ASCE 7 to determine the allowable story drift, SMF that are four stories or less, are usually categorized as "structures, other than masonry shear walls structures, four stories or less…with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts". When SMF are greater than four stories, they are categorized as "all other structures" when determining the allowable story drift.

The equivalent lateral force (ELF) procedure or modal response spectrum analysis (MRSA) Is used to determine elastic drifts, as permitted/required in ASCE 7. Elastic drifts are amplified (ASCE 7 12.8-15) to obtain design drifts which can be compared with the allowable. When MRSA is used, special load cases are employed for the drift checks where the natural period used for calculations is not limited to C_uT_a as it is in the ELF procedure and for strength checks (with either ELF or MRSA). Since the drift demands from MRSA are period dependent, iteration is required to ensure that the scaling factors used for the analysis are consistent with the period of vibration of the as-designed frame.

3.3 Strength

3.3.1 Checks Done with ASCE 7 Load Combinations

The equivalent lateral force (ELF) procedure or modal response spectrum analysis (MRSA) is used to determine strength demands on *beams* and *columns*. Unlike the drift checks, the period used in determining strength demands cannot exceed C_uT_a. Load effects from lateral forces are combined with gravity load effects using ASCE 7 load combinations. Structural design software (RAM, ETABS, RISA, or similar) is used to compare capacities with demands. LRFD design is used for strength checks.

While the member (beam and column) strength checks are performed using ASCE 7 load combinations, only some of the *connection* strength checks use load combinations. The others are performed using capacity-design principles as described in the next section. Connection strength checks that only use ASCE 7 load combination demands are bolt slip under wind loading and fuse plate rupture under seismic loading. In some other connection strength checks, gravity loads from load combinations are considered in combination with capacity-design seismic forces.

3.3.2 Capacity Design Checks

Most of the connection strength checks involve capacity-design. The connection plates, welds, and bolts are designed to resist the maximum force that can develop in the connection, with limit state capacities calculated per AISC 360. The sub-sections that follow indicate the way that the seismic demands are calculated using capacity design (Sections 3.3.2.1 through Section 3.3.2.3 of this document) and then specify the pertinent checks from AISC 360 that are used to design the connection (Sections 3.3.2.4 of this document).

3.3.2.1 Capacity-design demands at beam bottom flange level

The maximum probable force that will develop at the *bottom flange* level on each side of the connection is computed by assuming the moment at the column face, M_{pr}, is transmitted via a couple at the top and bottom flange levels (Fig. 10). The force V_{fe} is the force transmitted to each side of the connection, so 2* V_{fe} is the magnitude of the force acting at the bottom flange level.

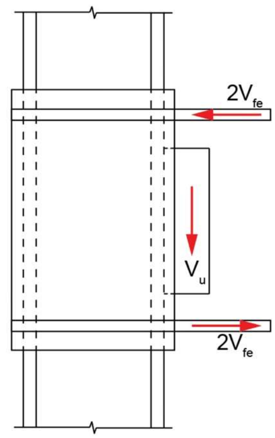


Fig. 10. The bottom flange force associated with M_{pr}.

The bottom flange demand on each side of the connection, V_{fe}, from this capacity-design approach is:

$$V_{fe} = \frac{M_{pr}}{2(d_b + t_p)} \quad \text{(Eq.1)}$$

where

M_{pr} = maximum probable moment at the fuse location. 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. Note, the fuse plate is proportioned during the design to have an expected ultimate capacity corresponding to M_{pr}.

d_b = depth of the beam

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

M_{pr} is usually equal to M_p, but the definition is flexible to ensure good behavior in situations where the strong-column weak-beam is not required to be satisfied (single-story frames and top-story situations). In situations where the beams are stronger than the column, it is desirable to limit the moment at the joint to less than the beam M_p to preclude column yielding. However, it is still the intent that the connection be fully restrained (FR), so additional calculation is done to justify the FR assumption when M_{pr} is less than M_p.

For joints where two beams frame in, the demands on the external continuity plates at the lower flange level are equal to the sum of the demands from each of the beam ends.

3.3.2.2 Capacity-design demands at beam top flange level

The maximum force that can develop at the *top flange* level will be greater than the bottom flange if drag forces are being passed through the connection (Fig. 11). The drag force on each side of the connection is designated P_d, so $2*P_d$ corresponds to the total drag force.

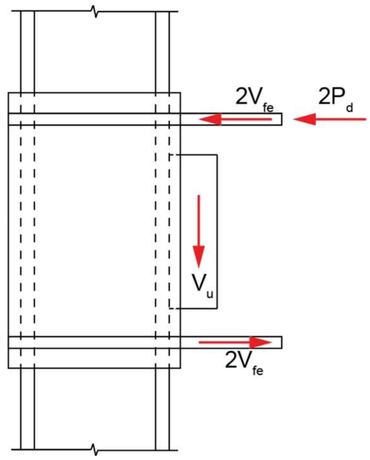


Fig. 11. Demands at top flange level when drag forces are present.

3.3.2.3 Capacity-design demand at shear tab

The required shear for the tab, V_u, is computed in accordance with 2.5 of AISC 358-22 which states "the required shear strength of the beam 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."

3.3.2.4 Capacity-design strength limit state checks

The capacity-design demands determined in the previous sections are used to check connection limit states per AISC 360-22. These various limit state checks are described in the design steps in AISC 358-22 Chapter 15 Section 15.6 (See Appendix for example calculations).

3.3.2.5 Expected fuse yield strength

In the calculations, the yield force of the fuse based on the final fuse plate geometry is documented in Step 19 and demonstrated to be less than V_{fe} used in the design of the other elements. The ratio of V_{fe}/V_y is usually around 1.9. This reflects material overstrength, material strain hardening, and the indirect effect of tensioned web bolts that increase moment capacity at the connection. The procedure for proportioning fuses (see AISC 358-22 Chapter 15 Section 15.6 Step 19 and associated Commentary) has calibrated parameters based on the experimental testing.

3.4 Seismic Provisions

In addition to drift and strength checks per ASCE 7 and AISC 360, the seismic provisions give additional checks to ensure ductile behavior. The checks that pertain to the DFF elements are the strong-column weak-beam check, lateral bracing checks, and member width-thickness checks.

3.4.1 Strong-Column Weak-Beam

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.

3.4.2 Lateral Bracing

3.4.2.1 Beam Stability Bracing

There are no requirements for stability bracing of beams or joints beyond those in AISC 360-22.

The prequalification experiments for the DuraFuse Frames connection have been conducted without lateral bracing near the beam-column connection or along the length of the beam. The only beam restraint in the tests was at the beam tip (actuator location). 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.

Table 1 summarizes the unbraced lengths of test specimens, relative to the unbraced length parameters in AISC 341-16 and AISC 360-16. The unbraced lengths in the test specimens, L_b, far exceeded the bracing in AISC 341-16 for highly ductile beams [$0.95r_yE/(R_yF_y)$], and the plastic unbraced length, L_p [$1.76r_y\sqrt{E/F_y}$].

Table 1. Unbraced lengths of test specimens.

Series	Beam Size	Unbraced Length, L_b [a] (ft)	$0.95r_yE/(R_yF_y)$ [b] (ft)	L_p (ft)
E	30×99	15.5	8.8	7.42
F	40×167	15.5	10	8.48
G	21×50	15.5	5.4	4.59
H	36×232	20.5	10.9	9.25

a. Distance from the column centerline to the beam tip in the experiments

b. AISC 341 bracing requirement for highly ductile beams that are expected to yield

In DFF design checks, beam stability bracing is provided based on the maximum force that can develop in the beam (capacity-design) and the stability requirements in AISC 360-16. DFF calculations include a maximum unbraced length of the beam based on capacity design.

3.4.2.2 Column Stability Bracing

Lateral bracing of the columns shall comply with AISC 341-16. The fuse plates at the connection are proportioned to preclude column yielding, so only restraint at the top beam flange level is required. The cover plates in the column create a torsionally rigid segment of the column that provides indirect lateral restraint to the column in the regions immediately outside the torsionally rigid segment.

3.4.3 Member Width-Thickness Requirements

For beams, there are no limits on the web width-to-thickness ratio beyond those listed in the AISC *Specification*, since the connection design precludes beam yielding. The beam flange width-to-thickness ratio shall not exceed λ_p per AISC *Specification* Table B4.1b.

For columns, width-to-thickness ratios for the flanges and web shall conform to the requirements for highly ductile sections in the AISC *Seismic Provisions*.

3.5 Prequalification Limits

3.5.1 Beams

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 in AISC 341-16.

(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) For HCAI, the beam weight shall be limited to a maximum of 232 lb/ft, since CBC 2205.A.4.9 does not allow extrapolation above the heaviest beams tested.

The limitations on beam sizes are based on the extent of experimental testing prior to 2019 when the DFF connection was reviewed by the AISC CPRP. The DuraFuse Frames connection has been investigated with a variety of beam sizes. The smallest beam size that has been tested with fuse plates with extensions, similar to Fig. 15-1, is W21×50. On the bigger side, more than ten tests have been performed with W33, W36, and W40 beams. The deepest beam that had been tested at the time of AISC 358-22 review was W40×167 (Reynolds and Uang 2019a). The heaviest beam that had been tested at the time of AISC 358-22 review was W36×232 (Reynolds and Uang 2019b).

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 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 heaviest beams tested at the time of AISC review were two H-series beams that were W36×232. Both beams met the HCAI acceptance criteria by completing at least two cycles at 0.04 rad drift and 0.03 rad inelastic rotation (Reynolds and Uang 2019b). The tests indicated ultimate rotational capacity beyond 0.06 rad. The H-series tests were supported by two others in the F-series with W40x167 beams. The F-series tests indicated ultimate rotation capacity beyond 0.06 rad (Reynolds and Uang 2019a).

3.5.2 Columns

Columns shall satisfy the following limitations:

(1) Columns shall be any of the shapes permitted in AISC 358 Section 2.3.

(2) 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).

The DuraFuse Frames connection has been investigated with a variety of column shapes and sizes. The W14 columns that have been tested are: W14×38 and W14×68. The W21 column that has been tested is W21×132. The W24 columns that have been tested are: W24×103 and W24×250. The W36 (W920) columns that have been tested are: W36×150 and W36×231 (Reynolds and Uang 2019a). The box columns that were tested were 24 in. deep, 17 in. wide, with a plate thickness of 1.75 in (Reynolds and Uang 2019b). 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 at the time of AISC 358-22 review.

3.5.3 Plates

Plates shall satisfy the following 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 roughness of all thermal cut surfaces on the plates (including fuse plates) shall be no greater than an ANSI surface roughness of 1000 micro-inches. Roughness exceeding this value or gouges not more than 3/16 in. shall be removed by machining or grinding. The fuse plates used for prequalification tests were fabricated with thermal cutting.

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 (Reynolds and Uang 2019a; Reynolds and Uang 2019b). 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 (Reynolds and Uang 2019a).

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.

3.5.4 Bolts

Bolts shall satisfy the following limitations:

(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. 6

(3) Bolt holes in beam flanges, fuse plates, and other connection 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) maximum.

(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)

3.6 Drags and Collectors

3.6.1 In-line

For in-line drags and collectors, the drag force is designed to be transmitted at the joint at the top flange level. The drag/collector force is combined with the maximum force at the top flange that can be developed by the fuse plate (Fig. 11). This combined force is used when checking bolts, welds, and connection plates (see Section 2.3.2 of this report). The drag force does not reduce the moment capacity of the connection, since the moment capacity is governed by the fuse strength at the bottom flange level and drag forces are not transferred at the bottom flange level.

Beams that are used as drags and collectors should be checked by the EOR for combined axial, moment, and shear. This is independent of the connection design performed for the DFF connections.

3.6.2 Orthogonal

For orthogonal drags and collectors, the drag force is designed to be transmitted through the joint at the top flange level. The geometry of the external continuity plate is adjusted to simplify the load path of the orthogonal forces through the connection. Fig. 12 illustrates the detail (shown for cantilever beam, but the same detail applies for orthogonal collectors). The thickness of the external continuity plate and the weld sizes are checked based on the demands from the moment frame (V_{fe} + P_d) in combination with the orthogonal demands, $P_{d,o}$.

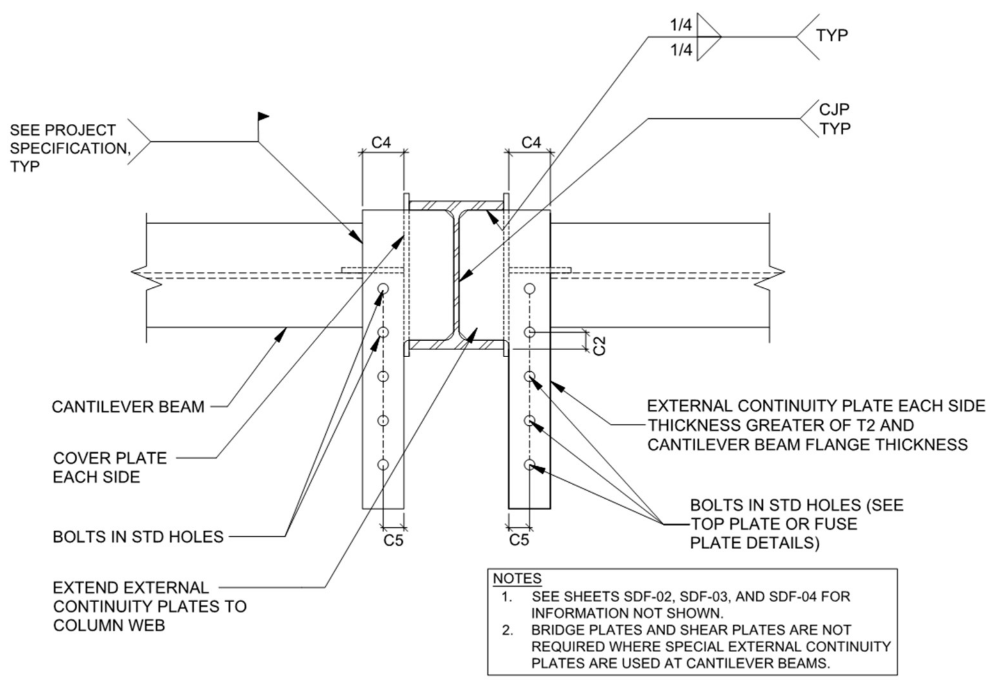


Fig. 12. Special external continuity plate for orthogonal drags or cantilevers.

The equations used for the design checks are derived below. Figure 13 summarizes terms that are used in the derivations.

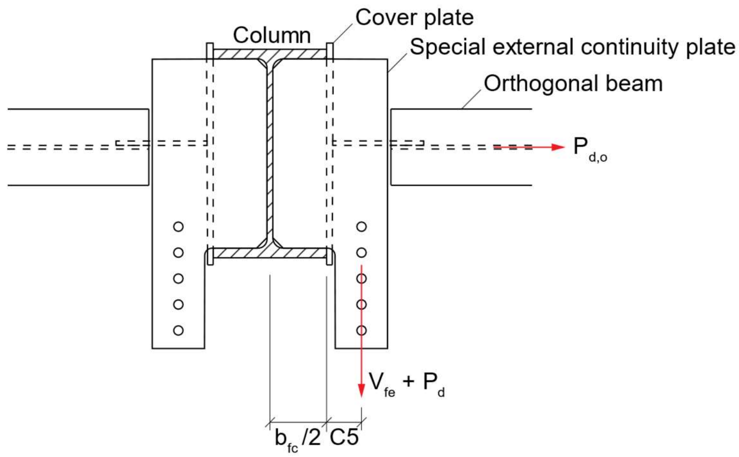


Fig. 13. Terms used in orthogonal load equations.

The demands on the special external continuity plate are summarized in Fig. 14. The derivations conservatively neglect any force transferred by the fillet welds between the special external continuity plate and the column flanges.

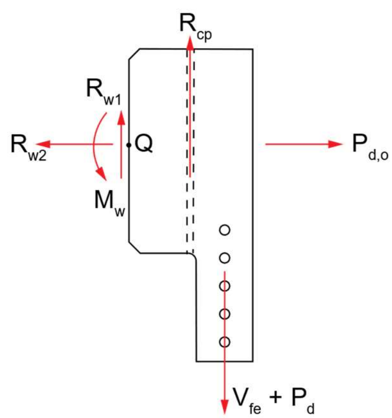


Fig. 14. Demands on special external continuity plate.

The demands in Fig. 14 are related through three equilibrium equations and one compatibility equation.

$$\Sigma F_{x,dir} = 0\,,\ \ R_{w2} = P_{d,o} \tag{Eq. 2}$$

$$\Sigma F_{y,dir} = 0\,,\ R_{cp} + R_{w1} = V_{fe} + P_d \tag{Eq. 3}$$

$$\Sigma M_{@Q} = 0\,,\ \ (V_{fe} + P_d)\left(\frac{b_{fc}}{2} + C5\right) - R_{cp}\left(\frac{b_{fc}}{2}\right) = M_w \tag{Eq. 4}$$

$$\frac{R_{cp}}{t_p} = \frac{R_{w1}}{0.5t_{wc}} \tag{Eq. 5}$$

The compatibility equation (Eq. 5) results in similar shear stress/deformation in the column web and the cover plates, and reflects the observation of essentially uniform panel zone deformation.

Equations 2-5 are used to solve for the four unknowns: R_{cp}, R_{w1}, R_{w2}, and M_w.

Combining Eq. 3 and Eq. 5:

$$R_{cp} = \frac{\left(V_{fe} + P_d\right)}{\left(1 + \frac{0.5t_{wc}}{t_{cp}}\right)} \qquad \text{(Eq. 6)}$$

Then from Eq. 3:

$$R_{w1} = \left(V_{fe} + P_d\right) - R_{cp} \qquad \text{(Eq. 7)}$$

Equations 2 and 4 provide direct calculations for R_{w2} and M_w, respectively, as is.

The capacity of the special external continuity plate is checked at two sections, at the cover plate and at the beam web. At both locations, the connections are CJP welded, so member limit states govern.

A free-body diagram with a cut at the cover plate section is shown in Fig. 15.

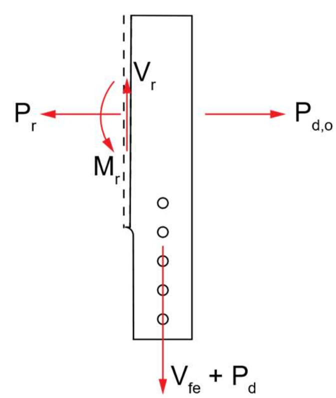


Fig. 15. Special external continuity plate with a cut at the cover plate section.

The flexural, normal, and shear demands on the section (Fig. 15) are expressed in terms of previously defined forces:

$$M_r = \left(\Sigma V_{fe} + P_d\right)[C5] \qquad \text{(Eq. 8)}$$

$$P_r = P_{d,o} \qquad \text{(Eq. 9)}$$

$$V_r = \left(\Sigma V_{fe} + P_d\right) \qquad \text{(Eq. 10)}$$

The capacities of the plate, M_c, P_c, and V_c are determined per AISC 360.

The plate is checked per the general connection interaction equation in the AISC Manual (Eq 9-1 in the fifteenth edition).

$$\frac{M_r}{M_c}+\left(\frac{P_r}{P_c}\right)^2+\left(\frac{V_r}{V_c}\right)^4\leq 1.0 \quad \text{(Eq. 11)}$$

A similar check is performed at the section at the column web section (Fig. 16).

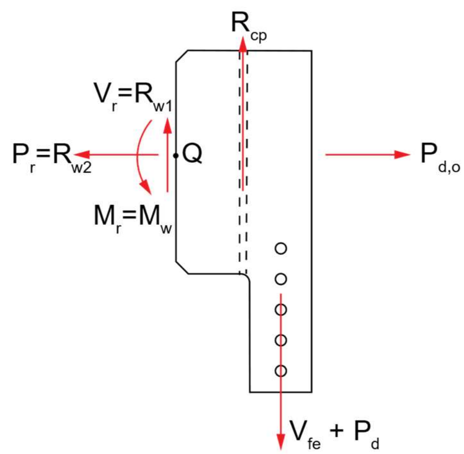


Fig. 16. Special external continuity plate with a cut at the cover plate section.

$$M_r = M_w \quad \text{(Eq. 12)}$$

$$P_r = R_{w2} \quad \text{(Eq. 13)}$$

$$V_r = R_{w1} \quad \text{(Eq. 14)}$$

$$\frac{M_r}{M_c}+\left(\frac{P_r}{P_c}\right)^2+\left(\frac{V_r}{V_c}\right)^4\leq 1.0 \quad \text{(Eq. 15)}$$

3.7 Orthogonal Gravity Cantilevers

For orthogonal gravity cantilevers, the geometry of the external continuity plates is adjusted to accommodate cantilevers. Figures 17-19 illustrate the details for cases where the orthogonal cantilever beam is the same depth as the DFF beam, is shallower than the DFF beam, or is deeper than the DFF beam.

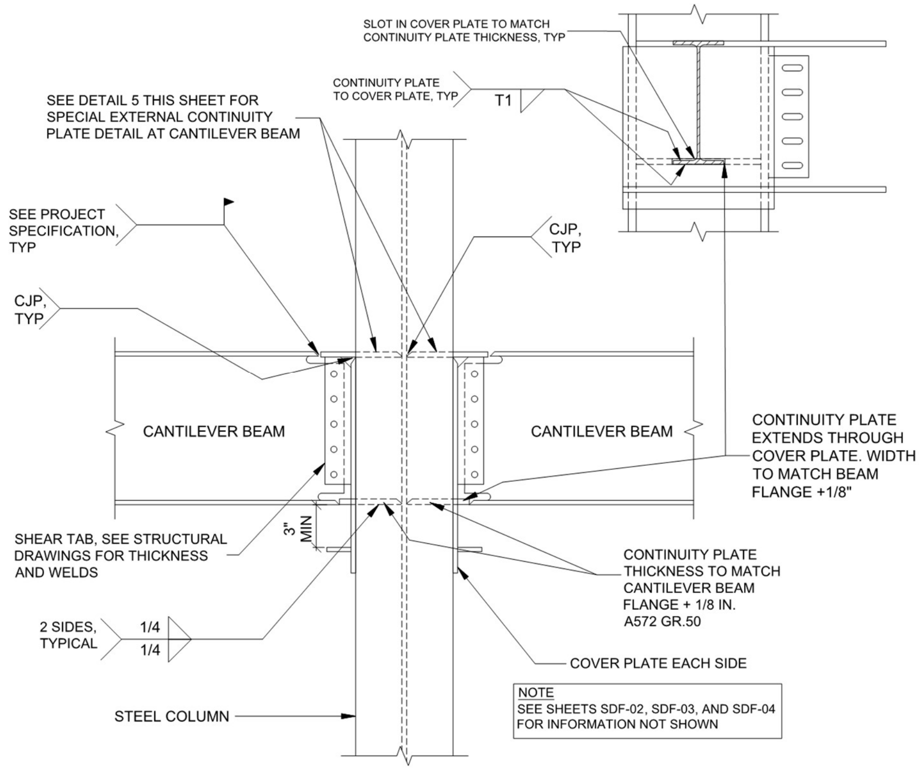


Fig. 17. Orthogonal gravity cantilever when cantilever depth is less than DFF beam depth.

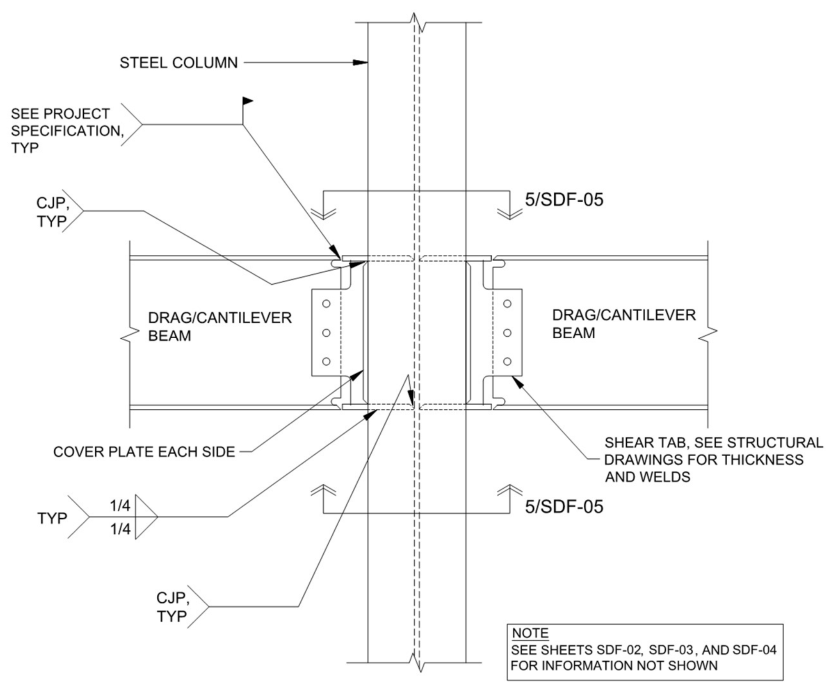


Fig. 18. Orthogonal gravity cantilever when cantilever depth equal to DFF beam depth.

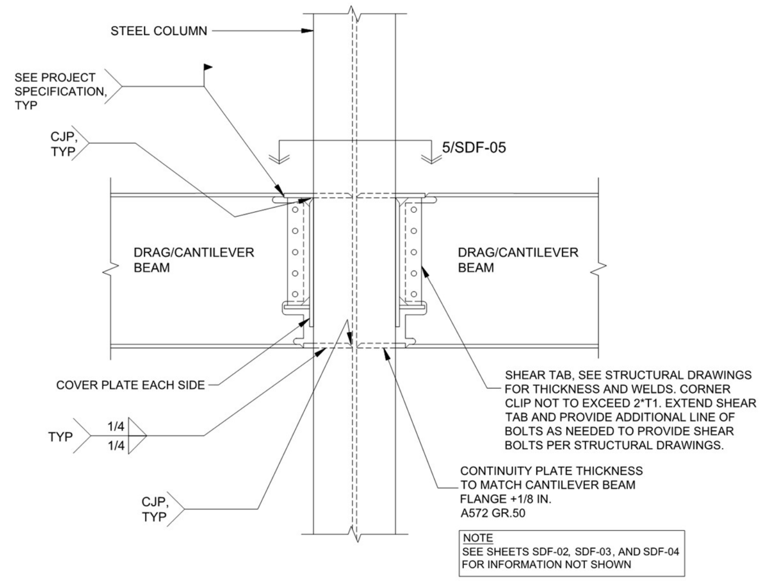


Fig. 19. Orthogonal gravity cantilever when cantilever depth is greater than DFF beam depth.

In all cases (Figs. 17-19), the top flange of the cantilever beam is directly attached to the external continuity plates of the DFF connection. The calculations to check the external continuity plate are the same as for orthogonal drags, but the force $P_{d,o}$ is taken as M_u/d_b where M_u is the design force for the cantilever beam.

In all cases (Figs. 17-19), a continuity plate is provided in the column at the bottom flange level to help transmit compression loads and mitigate cover plate deformations.

3.8 In-Plane Gravity Cantilevers

When a non-seismic gravity cantilever beam is present on the opposite face of a column with a one-sided special moment frame connection, the EOR may choose to use a two-sided DFF connection. In these situations, the bottom plate for the cantilever connection does not have the cut-outs defined by F3 (Fig. 7). The connection calculations combine the gravity load effects from the cantilever side with the capacity-design effects for the special moment frame connection on the other side.

3.9 Sloped and Stepped Connections

For roof connections that are sloped to accommodate drainage (up to a 1:12 slope), the column detail of Fig. 3 is modified to that shown in Fig. 20. The slope does not impact the calculation of the maximum forces being delivered at the top and bottom flange level (Equations in 2.3.2 are still valid). The slope does increase the distance from the centroid of the shear tab bolt group to the face of the column. The adjusted distance is used when computing the moment on the shear tab weld. The slope increases the length of W2 at the top external continuity plate and the length of W3 on the shear tab, but those increased weld lengths are conservatively neglected in calculations.

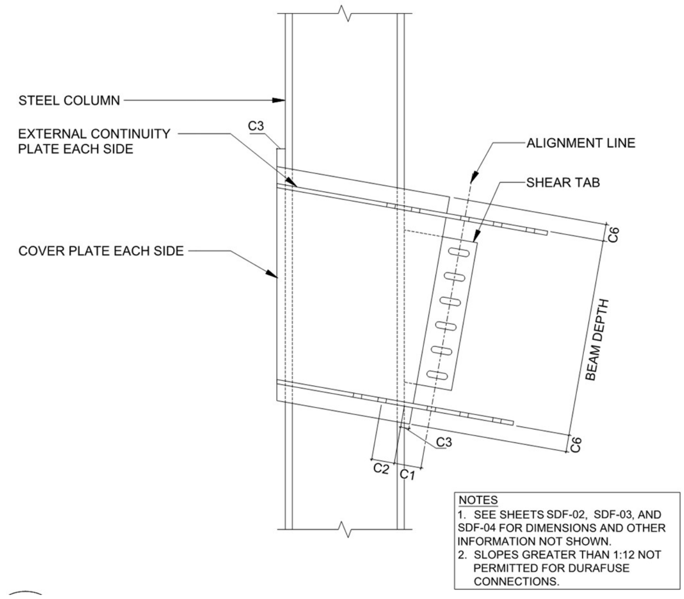


Fig. 20. Connection plates when the beam is sloped.

If DFF beams of different depths frame into the same joint, the detail shown in Fig. 21 is used. In this configuration, the length of W1 increases (cover plate to column weld), but the demands on W1 and the cover plate are no higher than when the external continuity plates are aligned. Similarly for cases with dropped beams (Fig. 22).

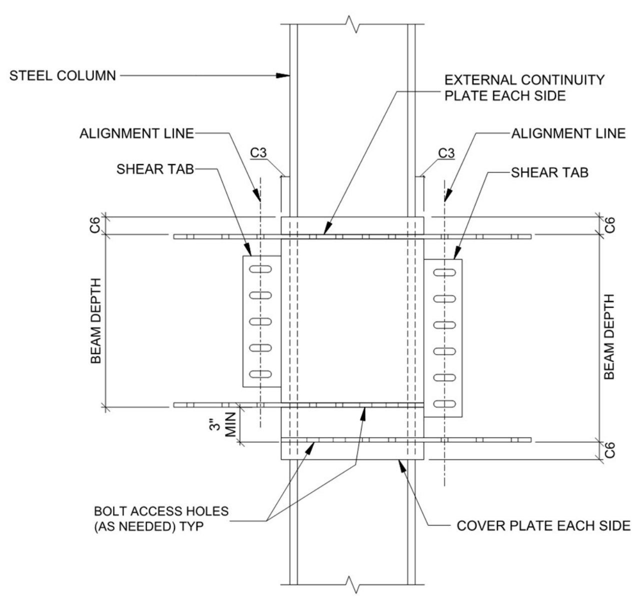


Fig. 21. Column detail for DFF beams of different depth.

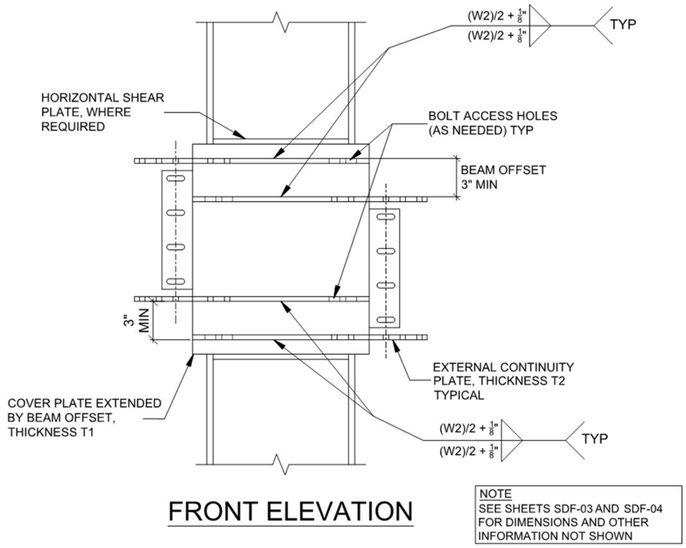


Fig. 22. Column detail for DFF beam that is dropped.

3.10 Example Calculations Summary Sheet

In DFF calculation submittals, each connection ID will have a summary sheet that lists the capacity and demand for all the limit states checked in each step of the design procedure (AISC 358-22 Section 15.6). Table 2 is an example of the summary table. In addition, detailed calculations for each limit state for each connection can be provided.

The calculations summary sheet and detailed calculations are organized by 20 design steps that apply to all connections (AISC 358-22 Section 15.6) followed by miscellaneous calculations that apply in some circumstances. Miscellaneous calculations include: bridge plate design checks (when bridge plates are present), shear plate design checks (when shear plates are present), FR assumption check (if M_{pr} is less than M_p, see 3.3.2.1 of this guide), rigid panel zone assumption check (if rigid panel zones have been assumed in the building model, see 4.4 of this guide), and external continuity plate checks for out-of-plane forces (if there are orthogonal drag loads or orthogonal cantilevers, see 3.6.2 and 3.7 of this guide).

Table 2. Calculation summary table for DFF connection.

Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 7.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5.6	OK
	Column-beam moment ratio	ΣM^*_{pb} = 10142	ΣM^*_{pc} = 5258	OK
2	Maximum probable moment	M_u = 2257 k-in	M_{pr} = 4749 k-in	0.48
3	Cover plate shear yielding	R_u = 112 kips	ϕR_n = 348 kips	0.32
	Cover plate thickness	t_{cp} = 0.5 in	t_{req} = 0.4 in	OK
4	Beam net section check	M_{pe} = 5247 k-in	M_{fr} = 5437 k-in	OK
5	Beam flange bolt shear failure	R_u = 112 kips	ϕR_n = 189.4 kips	0.59
	Bolt slip for wind load	R_u = 41.6 kips	ϕR_n = 70.6 kips	0.59
6	Alignment line location, C1	$C1_{req}$ = 6.4 in	C1 = 2.4 in	OK
7	Weld 1 strength	r_u = 5.5 kip/in	ϕr_n = 10.1 kip/in	0.55
8	Weld 2 strength	r_u = 7.9 kip/in	ϕr_n = 8.4 kip/in	0.94
9	External continuity plate rupture: Mode 1	P_u = 67.2 kips	ϕP_n = 70.1 kips	0.96
	External continuity plate rupture: Mode 2	P_u = 89.6 kips	ϕP_n = 99.3 kips	0.90
	External continuity plate rupture: Mode 3	P_u = 112 kips	ϕP_n = 166.3 kips	0.67
	External continuity plate rupture: Mode 4	R_u = 89.6 kips	ϕR_n = 108 kips	0.83
	External continuity plate rupture: Mode 5	R_u = 112 kips	ϕR_n = 139 kips	0.81
10	Beam shear	V_u = 48 kips	ϕV_n = 217.4 kips	0.22
11	Beam block shear	R_u = 224 kips	ϕR_n = 229.8 kips	0.97
12	Web bolt shear	R_u = 48 kips	ϕR_n = 151.5 kips	0.32
13	Shear tab rupture through net section	f_u = 14.4 ksi	ϕF_u = 48.8 ksi	0.29
	Shear tab weld failure	r_u = 14.4 kips	ϕr_n = 16.7 kips	0.86
15	Top plate for shear yielding	R_u = 112 kips	ϕR_n = 225 kips	0.50
	Top plate for shear rupture	R_u = 112 kips	ϕR_n = 137.1 kips	0.82
16	Top plate for tensile rupture in extensions	P_u = 44.8 kips	ϕP_n = 45.7 kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -0.2 in	P2 = 2.1 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 3.8	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 89.6 kips	ϕP_n = 153 kips	0.59
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 89.6 kips	ϕP_n = 133.1 kips	0.67
19	Fuse yielding region dimension	$F2_{max}$ = 2.2 in	F2 = 2.1 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 0.9	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 89.6 kips	ϕR_n = 135 kips	0.66
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 89.6 kips	ϕR_n = 109.7 kips	0.82
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	K_{req} = 1222350 k/in	K_S = 3042440.7 k/in	OK
	Rigid panel zone assumption	K_{req} = 4074499.9 k/in	β_{PZ} = 10908237.2 k/in	OK
	External continuity plates with out-of-plane forces	$Inter_{Limit}$ = 1.00	Inter = 0.48	OK
		Controlling Demand-Capacity Ratio		0.98

4 Computer Modeling

4.1 Overview

Modeling guidance for DuraFuse Frames for specific software is provided through a series of technical bulletins (www.durafuseframes.com). The general methods that apply, regardless of the software, are summarized in this section.

4.2 Beam and Column Elements

For special moment frames with relatively deep columns and beams, the shear deformations in the columns and beams are significant, relative to the flexural deformations, and should be considered in the model through the use of Timoshenko beam elements (or similar).

When modeling DFF beams, it is conservative to use the same cross-sectional properties of the beams and columns along their entire length, even though some stiffening occurs near the connections due to connection plates.

4.3 Beam-to-Column Connections

The DuraFuse Frames beam-to-column connection is modeled as a fully restrained (FR) connection. This classification is based on the experiments used for prequalification, from which the connection stiffness was derived (Richards 2022). When building the computer model, the beams are considered fixed to the columns.

4.4 Panel Zones

The panel zones in DFF are stiffened by cover plates (Fig. 3). Experiments have demonstrated that, with deeper columns, the stiffened panel zones are essentially rigid relative to the beam and column (Reynolds and Uang 2019a).

DuraFuse Frames panel zone may be modeled as rigid, when the computed panel zone stiffness, converted to a rotational spring, is at least ten times larger than the flexural stiffness of the beam (ASCE 2017). Otherwise, the flexibility of the panel zone shall be represented by rigid end zones for the beam and column elements and a scissor spring (Charney and Marshall 2006).

5 Fabrication and Erection

5.1 Overview

The DuraFuse Frames connections are fabricated by the same fabricator that is providing the structural steel package for the building. DuraFuse Frames grants a project-specific license to the fabricator to use the proprietary DFF connection (upon payment of the DFF license fee by the fabricator).

The DuraFuse Frames connections are erected by the same erector that is providing structural steel erection services for the building.

5.2 Fabrication and Erection

All welders, tack welders, and welding operators shall be qualified in conformance with AWS D1.1, Clause 4, Part C. Welders and welding operators performing welds as described in AWS D1.8 Clause 5.1.1 shall be subjected to Supplemental Welder Qualification Testing in accordance with AWS D1.8 Chapter 5.

Welding shall be performed under a Welding Procedure Specification (WPS) in accordance with AWS D1.1, which shall be prepared for every different welding application including welding position, welding process, electrode manufacturer, filler metal trade name for the electrode type selected, and other essential variables as defined in AWS D1.1. None of the welds in the DuraFuse Frames connection are classified as demand critical.

Erection of DuraFuse Frames shall be in accordance with Chapter M of AISC 360, AISC 303, Chapter I of AISC 341, and Chapter 17A and 22A California Building Code.

5.3 QA/QC

A quality assurance plan conforming to 2022 CBC Section 1704A.2, AISC 360 Chapter N, AISC 341 Chapter J, and AWS D1.8 Clause 7 shall be included in the construction documents by the registered design professional and approved by the building official.

Where lamellar tearing in base metal is observed by visual inspection, the steel fabricator shall repair per AWS D1.1 Section 5.14.

For shear plate welds that encroach on the K-area, visual inspection is required no sooner than 48 hours after welding per AWS D1.8, Section 7.4.

5.4 Protected Zones

The protected zone of the connection consists of only the yielding regions on the fuse plates (as indicated in the drawings). None of the other elements (beam, column, connection plates) experience significant inelastic deformations up to 0.04 rad drift or 0.03 rad inelastic rotation of the connection (Reynolds and Uang 2019a; Reynolds and Uang 2019b).

5.5 Fuse Material Check

As indicated in Section 3.5.3 of this guide, the fuse plates have the additional requirement that the material not have an MTR tensile strength greater than 85 ksi, unless independent testing confirms that the tensile strength is lower. The steel fabricator is responsible to report the MTR strength for the fuses or report independent testing to demonstrate compliance.

The surface roughness of the fuse plates shall be no greater than an ANSI surface roughness of 1000 micro-inches. Roughness exceeding this value or gouges not more than 3/16 in. shall be removed by machining or grinding. The curved cuts on the fuse plate shall be defined by a smooth radius (no faceted curves).

5.6 Welding

All welding shall be performed using E70 electrodes. The weld filler metal and associated welding process for all fillet welds shall meet the requirements of any of the following:

- E70T-6, E7IT-8, or E70TG-K2 for flux-cored arc welding (FCAW)
- E7XT-9 for flux core arc welding (FCAW) with gas shielding
- F7A2-EXXX for submerged arc welding (SAW)
- E7018 stick electrode for shielded metal arc welding (SMAW)
- ER70S-X, E70C-XM, or E70C-XC for gas metal arc welding, except for short circuit transfer.

All weld filler material shall satisfy the requirements specified in Clause 6.3 of the Structural Welding Code-Seismic Supplement (AWS D1.8/D1.8M) including the minimum Charpy V-notch (CV) toughness 20 ft-lb (27 J) at a temperature lower than 0°F (-18°C) as indicated in Section A3.4 of AISC 341-16, and 40 ft-lb impact strength at 70°F as determined by AWS classification methods of manufacturer certification. All weld filler metal shall satisfy a maximum diffusible hydrogen content requirement per AWS D1.1 and D1.8 Section 6.3 of 16 milliliters of hydrogen per 100 grams of weld metal (H16) or less.

5.7 Bolting

Bolts shall be pretensioned high-strength bolts conforming to ASTM F3125 Grade A490 or A490M, or Grade A325 or A325M, or Grade F2280 or F2280M, or Grade F1852 or F1852M. Bolt diameter is limited to a maximum of 1¼ inches (32 mm).

Shims with a maximum overall thickness of ¼ inch (6.4 mm) may be used between the top plates and bars or beams and between the fuse plate and bars or beams. Shims, if required, may be finger shims or may be produced with drilled or punched holes. Experiments have demonstrated the same performance for connections with and without shim plates (Reynolds and Uang 2019a).

5.8 Shape and Fabrication Tolerances

To account for shape tolerances (actual width more than nominal), column flange trimming via grinding is permitted. The roughness of ground surfaces shall be no greater than an ANSI surface roughness of 1000 micro-inches. Transitions shall be 10:1 or greater. The trimming shall not reduce the flange width to less than the nominal design dimension.

To account for shape tolerances (actual width less than nominal but within AISC tolerances) shims are permitted between the cover plates and the column flanges. The shim material shall be the same grade as the cover plate. Fillet weld size shall be increased by the size of the shim.

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.

5.9 Specification Subsection and TIO

5.9.1 Specification Subsection

DuraFuse Frames fall under the requirements of the Structural Steel Framing specification for a project and does not require its own specification subsection. SEOR shall include the following DuraFuse specific subsection within the Structural Steel Framing specification.

1.X Requirements for use of Patented Moment Connection

A. Project includes use of DuraFuseFrames® moment frame connections.

B. Contractor shall be responsible for payment of license fee to DuraFuse Frames.

C. DuraFuse Frames® have requirements beyond this Section affecting submittals, materials, tolerances, details of fabrication and erection, quality control and quality assurance. Conform to these additional requirements designated on Drawings.

1. Where designated requirements are less stringent than the requirements of this Section, the more stringent requirements of this Section shall apply except as otherwise approved by Owner's Representative.

2. Additional requirements are only applicable to DuraFuse Frames® connections and connected members.

5.9.2 TIO

On-site special inspection for DuraFuse Frames shall include:

C-S7: Fuse plate curved cuts shall be defined by a smooth radius (no faceted curves).

C-S8: Fuse plates shall have a surface roughness no greater than ANSI surface roughness of 1000 micro-inches.

			Select with " required OPA
Index #	Stage 1 Required (Select with "X")	ON-SITE SPECIAL INSPECTIONS	Samples of Test & Inspection Reports Included
C-S!		CBC 1705A.2, 1705A.12.2, & 1705A.13.3 Cold-formed steel light frame construction	
C-S6		**Steel** CBC 1705A.13 Special inspections for seismic resistance	
C-S7	X	**Steel** DuraFuse Frames Moment Connection Fuse plates curved cuts shall be defined by a smooth radius (no faceted curves).	
C-S8	X	**Steel** DuraFuse Frames Moment Connection Fuse plates shall have a surface roughness no greater than ANSI surface roughness of 1000 micro-inches.	

Fig. 23. Example TIO.

6 Checklist For Plan Reviewers and FAQ

1) Are the maximum beam and column sizes acceptable?
    a. Verify that the maximum beam depth is limited to a W40x section.
    b. Verify that the maximum beam weight is limited to 232 lbs/ft. Any deviation will be considered as an alternative system by HCAI.
    c. Verify that the maximum column depth is limited to 36”.
2) Are the DuraFuse Frames connection designs acceptable?
    a. For each connection, verify that the summary sheet indicates D/C ratios of 1.0 or less for all limit state checks, or that an OK is listed (for checks that do not have a D/C ratio).
3) Have the beams been braced appropriately to achieve M_p beam?
    a. Verify that the SEOR has provided lateral bracing per AISC 360.
    b. Verify that lateral bracing does not exceed that indicated on DFF calculations.
4) Have the DFF connections been represented accurately in the computer model?
    a. Verify that the beam-to-column connections are modeled as FR.
    b. Verify that if the panel zones are being modeled as rigid, the calculations justify it.
5) Are protected zones clearly marked?
    a. Verify that the fuse plate yield regions are marked as protected zones.
    b. Verify that the beam or other connection plates have not been marked as protected.
6) Are the gravity cantilever beams attached to the external continuity plates acceptable?
    a. Verify that the details match those herein.
    b. Verify that the cantilever loads have been included in connection calculations.
7) Are orthogonal drag beams passing through the DFF connection acceptable?
    a. Verify that the drag loads have been included in connection calculations.
8) Has connection geometry been accounted for in architectural details?
    a. Verify that the slab edge distance is sufficient to accommodate the connection by comparing the required slab edge with that provided.
    b. Check perimeter frames and interior frames that are adjacent to openings.
9) Have the connections been checked for collector/drag loads?
    a. Each connection is designed for a minimum of $0.1A_gF_y$ (10%) beam capacity. Verify that the calculations are updated when collector diagram shows demand greater than 10% of beam capacity at a connection.
    b. The top plates are designed to take all the collector/drag loads.
10) Is there a guide to understand the twenty (20) steps under calculation package?
    a. Refer to AISC 358-22 Chapter 15 commentary which provides a detailed explanation on each step.

7 Standard General Specifications and Drawings

See the following pages for standard specifications and drawings.

COPYRIGHT, INTELLECTUAL PROPERTY, AND LICENSE FEE

1.3. THE USE OF THIS DRAWING IS RESTRICTED FOR USE FOR THE BUILDING SPECIFICALLY IDENTIFIED BELOW:

PROJECT: WEST VALLEY HOSPITAL
LOCATION: WEST VALLEY, UT
DFF TRACKING NUMBER: 230308-00

1.4. DURAFUSE FRAMES, LLC IS NOT RESPONSIBLE FOR THE USE OF THIS DRAWING BY OTHERS WITHOUT ITS KNOWLEDGE AND CONSENT.
1.5. THE DURAFUSE FRAMES SYSTEM IS COVERED BY ONE OR MORE U.S. PAT. NOS. US-2017-0218616-A1, US-2018-0274223-A1, US-2018-0347222-A1, AND FOREIGN COUNTERPARTS.
1.6. THIS DRAWING AND THE INFORMATION HEREON MAY ONLY BE USED AFTER THE PAYMENT OF A LICENSE FEE TO DURAFUSE FRAMES, LLC.
1.7. LICENSE FEE:
1.7.1. PROSPECTIVE STEEL FABRICATORS SHALL REQUEST THE LICENSE FEE IN WRITING FROM DURAFUSE FRAMES, LLC (801-727-4060) (QUOTE@DURAFUSEFRAMES.COM).
1.7.2. PROSPECTIVE STEEL FABRICATORS SHALL INCLUDE THE LICENSE FEE IN THEIR BID PRICE.
1.7.3. UPON CONTRACT AWARD, THE FABRICATOR SHALL SUBMIT A PURCHASE ORDER TO DURAFUSE FRAMES, LLC FOR THE TOTAL FEE, AND SHALL INCLUDE THE FEE IN ITS FIRST CONSTRUCTION DRAW.
1.7.4. PAYMENT OF THE FEE SHALL BE MADE TO:
DURAFUSE FRAMES, LLC
5801 WEST WELLS PARK ROAD
WEST JORDAN, UT 84081
1.8. FABRICATORS ARE ENCOURAGED TO USE THE TEKLA OR SDS2 PLUGINS TO HELP AUTOMATE DURAFUSE FRAMES DETAILING. USE OF THESE TOOLS IS INCLUDED IN THE LICENSE FEE.

DESIGN NOTES

2.1. DESIGN OF THE DURAFUSE FRAMES® SHALL BE IN ACCORDANCE WITH THE FOLLOWING:
- 2022 CALIFORNIA BUILDING CODE
- AISC SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS (ANSI/AISC 360-16)
- AISC SEISMIC PROVISIONS (ANSI/AISC 341-16)
- PREQUALIFIED CONNECTIONS FOR SPECIAL AND INTERMEDIATE STEEL MOMENT FRAMES FOR SEISMIC APPLICATIONS (ANSI/AISC 358-22)
- 2014 RCSC SPECIFICATION FOR STRUCTURAL JOINTS USING HIGH-STRENGTH BOLTS
- STRUCTURAL WELDING CODE-STEEL (AWS D1.1-2015)
- STRUCTURAL WELDING CODE SEISMIC SUPPLEMENT (AWS D1.8-2016)
- IAPMO UES ER 610, INCLUDING THE CALIFORNIA SUPPLEMENT

2.2. USE OF THIS PRODUCT IS SUBJECT TO THE APPROVAL OF HCAI ALTERNATE MEANS OF COMPLIANCE TO MEET THE 2022 CBC SECTION 2205A.5 REQUIREMENTS.
2.3. DURAFUSE FRAMES ENGINEERING DESIGN RESPONSIBILITIES ARE LIMITED TO DESIGN OF THE BEAM-TO-COLUMN CONNECTIONS IN THE MOMENT FRAMES.
2.4. MEMBER SIZES, DIMENSIONS, AND ELEVATIONS ON DURAFUSE FRAMES SHEETS ARE SHOWN FOR COORDINATION ONLY. PROJECT DOCUMENTS SHALL GOVERN.
2.5. PRIOR TO SHOP FABRICATION, THE CONTRACTOR SHALL SUBMIT THE FOLLOWING TO DURAFUSE FRAMES, LLC (VIA THE ENGINEER OF RECORD) FOR REVIEW AND DISPOSITION:
2.5.1. ELECTRONIC COPY OF ALL STRUCTURAL STEEL DRAWINGS PERTAINING TO OR EFFECTING THE SHOP FABRICATION AND ERECTION OF THE DURAFUSE FRAMES, INCLUDING ALL RE-SUBMITTALS. REVIEW TIME FOR DURAFUSE FRAMES SHALL BE AT LEAST THE SPECIFIED REVIEW TIME FOR THE ENGINEER OF RECORD (NOT LESS THAN 7 BUSINESS DAYS).
2.5.2. WELDING PROCEDURE SPECIFICATION (WPS) AND/OR PROCEDURE QUALIFICATION RECORD (PQR) AND CURRENT CERTIFICATE OF COMPLIANCE BY WIRE MANUFACTURER.
2.5.3. QUALITY CONTROL PROGRAM (REQUIRED IF NOT AISC CERTIFIED).

MATERIALS

3.1. STRUCTURAL SHAPES:
3.1.1. ROLLED WIDE-FLANGE BEAMS SHALL CONFORM TO ASTM A992. ROLLED WIDE-FLANGE COLUMNS SHALL CONFORM TO MATERIAL RECOGNIZED IN AISC 341. THE SPECIFIED MINIMUM YIELD STRESS OF COLUMNS, AS SPECIFIED ON THE SCHEDULE, SHALL NOT EXCEED 70 KSI.
3.1.2. ROLLED WIDE-FLANGE SHAPES IN THE SFRS WITH FLANGES 2 INCHES THICK OR THICKER, SHALL HAVE A MINIMUM CHARPY V-NOTCH (CVN) TOUGHNESS OF 20 FT-LBS AT 70°F. COMPLIANCE SHALL BE DEMONSTRATED FOR EACH DIFFERENT SIZE IN THE SFRS BY THE CERTIFIED MILL TEST REPORT (CMTR). ALTERNATIVELY, THE CVN MAY BE DETERMINED USING A COUPON ORIENTED PARALLEL TO THE LONGITUDINAL AXIS OF THE MEMBER, WITH THE LONGINTUDINAL AXIS OF THE TEST SPECIMEN AS CLOSE AS PRACTICABLE TO A POINT MIDWAY BETWEEN THE INNER FLANGE SURFACE AND THE CENTER OF THE FLANGE THICKNESS, AT THE INTERSECTION OF THE WEB MID-THICKNESS.
3.1.3. PLATES FOR BUILT-UP WIDE FLANGE OR BOX COLUMNS SHALL CONFORM TO THE DURAFUSE FRAMES REQUIREMENTS FOR PLATES AND FLAT BAR.
3.1.4. HSS COLUMNS SHALL CONFORM TO ASTM A500 OR A1085, AS SPECIFIED ON THE SCHEDULE.
3.2. PLATES AND FLAT BAR:
3.2.1. ALL CONNECTION STEEL PLATES, WHICH CONSIST OF COVER PLATES, EXTERNAL CONTINUITY PLATES, INTERNAL CONTINUITY PLATES, SHEAR TABS, TOP PLATES, AND FUSE PLATES SHALL BE FABRICATED FROM STRUCTURAL STEEL COMPLYING WITH ASTM A572 GRADE 50 STEEL.
3.2.2. FUSE PLATES SHALL BE FABRICATED FROM PLATES WITH MILL CERTIFIED TENSILE STRENGTH LESS THAN OR EQUAL TO 85 KSI, UNLESS INDEPENDENT MATERIAL TESTING DETERMINES THAT THE TENSILE STRENGTH IS LESS THAN OR EQUAL TO 85 KSI.
3.2.3. PLATES THAT ARE THICKER THAN 2 INCHES SHALL HAVE A MINIMUM CHARPY V-NOTCH TOUGHNESS OF 20 FT-LB AT 70°F IN ACCORDANCE WITH A3.3 OF AISC 341-16. CHARPY TOUGHNESS SHALL BE DOCUMENTED IN THE CERTIFIED MILL TEST REPORT. ALTERNATIVELY, THE CHARPY TOUGHNESS MAY BE DETERMINED USING A COUPON FROM EACH PIECE OF PLATE.
3.2.4. HOT DIP GALVANIZING IS NOT PERMITTED. EXTERIOR APPLICATIONS ARE NOT PERMITTED.
3.3. WELDS:
3.3.1. ALL WELDING SHALL BE PERFORMED USING E70 ELECTRODES. WELD FILLER METAL AND ASSOCIATED WELDING PROCESS FOR ALL FILLET WELDS SHALL MEET THE REQUIREMENTS OF ANY OF THE FOLLOWING:
- E70T-6, E71T-8, OR E70TG-K2 FOR FLUX CORED ARC WELDING (FCAW)
- E7XT-9 FOR FLUX CORE ARC WELDING (FCAW) WITH GAS SHIELDING
- F7A2-EXXX FOR SUBMERGED ARC WELDING (SAW)
- E7018 STICK ELECTRODE FOR SHIELDED METAL ARC WELDING (SMAW)
- ER70S-X, E70C-XM OR E70C-XC FOR GAS METAL ARC WELDING, EXCEPT SHORT CIRCUIT TRANSFER.

3.3.2. ALL WELD FILLER MATERIAL SHALL SATISFY THE REQUIREMENTS IN CLAUSE 6.3 OF AWS D1.8, INCLUDING THE MINIMUM CHARPY V-NOTCH TOUGHNESS OF 20 FT-LB AT A TEMPERATURE LOWER THAN 0°F IN ACCORDANCE WITH A3.4 OF AISC 341-16, AND 40 FT-LB IMPACT STRENGTH AT 70°F AS DETERMINED BY AWS CLASSIFICATION TEST METHODS OR MANUFACTURER CERTIFICATION.
3.3.3. ALL WELD FILLER METAL SHALL SATISFY A MAXIMUM DIFFUSIBLE HYDROGEN CONTENT REQUIREMENT PER AWS D.1.1 AND D1.8 SECTION 6.3 OF 16 MILLILITERS OF HYDROGEN PER 100 GRAMS OF WELD METAL (H16) OR LESS.
3.3.4. REFER TO AWS D1.8 ANNEX B FOR REQUIREMENTS WHEN MIXING FCAW-S WITH OTHER WELD PROCESSES.
3.4. BOLTS:
3.4.1. BOLTS SHALL BE ASTM F3125 GRADE A490, OR GRADE A325, OR GRADE F2280, OR GRADE F1852, AS SPECIFIED ON THE SCHEDULE.
3.4.2. SHIMS WITH A MAXIMUM OVERALL THICKNESS OF ¼ INCH MAY BE USED BETWEEN TOP PLATES AND BARS OR BEAMS AND BETWEEN THE FUSE PLATE AND BAR OR BEAMS. SHIMS MAY BE FINGER SHIMS OR MAY BE PRODUCED WITH DRILLED OR PUNCHED HOLES.
3.5. COMPRESSIVE EXPANSION JOINT MATERIAL:
3.5.1. SPECIFIED COMPRESSIVE STRENGTH SHALL BE NO LESS THAN 10 PSI UNLESS OTHERWISE INDICATED ELSEWHERE FOR THE PROJECT. SPECIFIED COMPRESSIVE STRENGTH SHALL BE NO MORE THAN 100 PSI.
3.5.2. RECOMMENDED MATERIALS AT NON-FIRE RATED ASSEMBLIES ARE RIGID FOAM TYPE EXPANDED POLYSTYRENE (EPS) OR TYPE EXTRUDED POLYSTYRENE (XPS). RECOMMENDED MATERIALS AT FIRE-RATED ASSEMBLIES ARE SPRAYED FIRE-RESISTIVE MATERIALS (SPRAY-APPLIED FIREPROOFING) WITH FIRE RESISTANCE DESIGN PARAMETERS AS REQUIRED FOR THE PROJECT.

FABRICATION

4.1. FABRICATION AND ERECTION OF DURAFUSE FRAMES® SHALL BE IN ACCORDANCE WITH CHAPTER M OF AISC 360, AISC 303, CHAPTER I OF AISC 341, AND 2022 CBC CHAPTER 22A.
4.2. BOLTS SHALL BE ARRANGED SYMMETRICALLY ABOUT THE AXIS OF THE BEAM.
4.3. BOLT HOLES SHALL BE MADE BY DRILLING, SUB-PUNCHING AND REAMING, LASER CUTTING, PLASMA CUTTING, OR WATER-JET CUTTING.
4.4. THE ROUGHNESS OF ALL THERMAL CUT SURFACES SHALL BE NO GREATER THAN AN ANSI SURFACE ROUGHNESS OF 1000 MICRO-INCHES.
4.5. LOOSE SLAG OR CUTTING BY-PRODUCTS SHOULD BE REMOVED FROM PARTS.
4.6. CURVED CUTS SHALL BE DEFINED BY A SMOOTH RADIUS (NO FACETED CURVES).
4.7. TO ACCOUNT FOR SHAPE TOLERANCES (ACTUAL WIDTH MORE THAN NOMINAL), COLUMN FLANGE TRIMMING VIA GRINDING IS PERMITTED.
- ROUGHNESS OF GROUND SURFACES SHALL BE NO GREATER THAN AN ANSI SURFACE ROUGHNESS OF 1000 MICRO-INCHES.
- TRANSITIONS SHALL BE 10:1 OR GREATER.
- TRIMMING SHALL NOT REDUCE THE FLANGE WIDTH TO LESS THAN THE NOMINAL DESIGN DIMENSION.

4.8. TO ACCOUNT FOR SHAPE TOLERANCES WHERE ACTUAL WIDTH IS LESS THAN NOMINAL BUT WITHIN AISC TOLERANCES, SHIMS ARE PERMITTED BETWEEN THE COVER PLATES AND THE COLUMN FLANGES. THE SHIM MATERIAL SHALL BE THE SAME GRADE AS THE COVER PLATE. FILLET WELD SIZE SHALL BE INCREASED BY THE SIZE OF THE SHIM.
4.9. TOLERANCE OF JOINT DIMENSIONS AND DIMENSIONAL TOLERANCE OF WELDED STRUCTURAL MEMBERS SHALL BE IN ACCORDANCE WITH AWS D1.1 SECTION 5.21 AND 5.22.
4.10. ALL WELDERS, TACK WELDERS, AND WELDING OPERATORS SHALL BE QUALIFIED IN CONFORMANCE WITH AWS D1.1, CLAUSE 4, PART C. AWS D1.8 ANNEX D (AS REQUIRED) AND AS APPROVED BY HCAI.
4.11. WELDERS AND WELDING OPERATORS PERFORMING WELDS AS DESCRIBED IN AWS D1.8 CLAUSE 5.1.1 SHALL BE SUBJECTED TO SUPPLEMENTAL WELDER QUALIFICATION TESTING IN ACCORDANCE WITH AWS D1.8 CHAPTER 5.
4.12. WELDING SHALL BE PERFORMED IN ACCORDANCE WITH A WELDING PROCEDURE SPECIFICATION (WPS) IN ACCORDANCE WITH AWS D1.1 WHICH SHALL BE PREPARED FOR EVERY DIFFERENT WELDING POSITION, WELDING PROCESS, ELECTRODE MANUFACTURER, FILLER METAL TRADE NAME FOR THE ELECTRODE TYPE SELECTED, AND OTHER ESSENTIAL VARIABLES AS DEFINED IN AWS D1.1.

INSTALLATION

5.1. BOLTS SHALL BE INSTALLED TO A PRETENSIONED CONDITION FOLLOWING THE REQUIREMENTS IN THE RCSC SPECIFICATION. THE INSPECTOR SHALL ENSURE THAT THE BOLTING CREW PROPERLY FOLLOW THE CORRECT METHODOLOGY OUTLINED BELOW:
- FOR TURN OF THE NUT METHOD: BY VISUAL OBSERVATION ENSURE THAT THE NUT AND PROTRUDING END OF THE BOLT HAVE BEEN MATCH-MARKED SO THAT THE PROPERLY ROTATED TURNED ELEMENT CAN BE VISUALLY OBSERVED RELATIVE TO THE UNTURNED ELEMENT.
- FOR CALIBRATED WRENCH METHOD: BY ROUTINE OBSERVATION ENSURE THAT THE CALIBRATED WRENCH IS BEING APPLIED TO THE TURNED ELEMENT.
- FOR TWIST-OFF-TYPE TENSION CONTROL METHOD: BY ROUTINE OBSERVATION ENSURE THAT THE SPLINED ENDS ARE PROPERLY SEVERED DURING INSTALLATION.

5.2. AN ASTM F436 ORDINARY THICKNESS WASHER SHALL BE USED UNDER EACH NUT AT ALL LOCATIONS IN THE DURAFUSE FRAMES® CONNECTIONS.
5.3. FAYING SURFACES SHALL BE CLASS A AND SHALL NOT HAVE A SURFACE ROUGHNESS THAT ACHIEVES A FRICTION COEFFICIENT IN EXCESS OF 0.5 (CLASS B).
5.4. PROTECTED ZONE:
5.4.1. THE PROTECTED ZONE AT EACH CONNECTION CONSISTS OF SPECIFIC REGIONS OF THE FUSE PLATE.
5.4.2. THE CONTRACTOR SHALL KEEP THE PROTECTED ZONES IDENTIFIED THROUGHOUT CONSTRUCTION AND ALSO AFTER APPLICATION OF FIREPROOFING USING NON DESTRUCTIBLE MEANS.
5.4.3. WELDING, BOLTING, SCREWING, OR SHOT-IN ATTACHMENTS FOR PARTITIONS, DUCT WORK, PIPING, OR OTHER CONSTRUCTION SHALL NOT BE PLACED WITHIN THE PROTECTED ZONE.

FIELD MODIFICATIONS AND REPAIRS

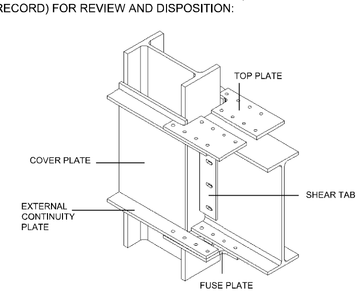
6.1. ROUGHNESS EXCEEDING AN ANSI SURFACE ROUGHNESS OF 1000 MICRO-INCHES OR GOUGES NOT MORE THAN 3/16 INCHES SHALL BE REMOVED BY MACHINING OR GRINDING. GOUGES DEEPER THAN 3/16 INCHES SHALL BE REPAIRED PER AWS D1.1 SECTION 5.14.5.2 AND 5.14.8.
6.2. WITHIN THE PROTECTED ZONE, DISCONTINUITIES CREATED BY FABRICATION OR ERECTION OPERATIONS, BEYOND THOSE REQUIRED FOR INITIAL FABRICATION OF THE PLATE, SUCH AS TACK WELDS, ERECTION AIDS, AIR-ARC GOUGING AND THERMAL CUTTING SHALL BE REPAIRED. REPAIR AND NON DESTRUCTIVE TESTING SHALL BE PER AWS D1.8-2016 SECTION 6.18.

FIREPROOFING

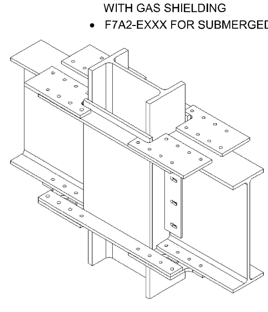
7.1. WHEN THE STEEL STRUCTURE IS REQUIRED TO MEET A CERTAIN CONSTRUCTION RATING, THE DURAFUSE FRAMES® CONNECTIONS SHALL MEET THE RATING.
7.2. WHERE SPRAY FIRE-RESISTANT MATERIAL (SFRM) IS USED, MINIMUM THICKNESS OF SFRM FOR THE DURAFUSE FRAMES® CONNECTIONS SHALL BE SIMILAR TO THAT OF THE BEAM AND COLUMN SECTIONS.
7.3. FINAL FIREPROOFING APPROVAL SHALL BE BY THE HCAI FIRE-SAFETY OFFICER.

QUALITY CONTROL AND QUALITY ASSURANCE

8.1. QUALITY CONTROL AND QUALITY ASSURANCE SHALL BE IN ACCORDANCE WITH CHAPTER N OF AISC 360 AND CHAPTER J OF AISC 341.
8.2. STRUCTURAL STEEL INSPECTION SHALL FOLLOW 2022 CBC TABLE 1705A.2.1 REQUIREMENTS.
8.3. THE FABRICATOR AND ERECTOR SHALL BE RESPONSIBLE FOR QUALITY CONTROL (QC). THE OWNER'S REPRESENTATIVE SHALL BE RESPONSIBLE FOR QUALITY ASSURANCE (QA) INSPECTIONS. QC AND QA REQUIREMENTS SHALL BE IN ACCORDANCE WITH AISC 360 CHAPTER N, AISC 341 CHAPTER J, AND 2022 CBC CHAPTER 17A.
8.4. DURAFUSE FRAMES® FUSE PLATES SHALL HAVE CURVED CUTS DEFINED BY A SMOOTH RADIUS (NO FACETED CURVES).
8.5. DURAFUSE FRAMES® FUSE PLATES SHALL HAVE SURFACE ROUGHNESS NO GREATER THAN ANSI SURFACE ROUGHNESS OF 1000 MICRO-INCHES.
8.6. WHERE LAMELLAR TEARING IN BASE METAL IS OBSERVED BY VISUAL INSPECTION, THE STEEL FABRICATOR SHALL REPAIR PER AWS D1.1 SECTION 5.14.
8.7. WHERE WELDING OF A HORIZONTAL SHEAR PLATE ENCROACHES INTO THE K-AREA (AS DEFINED IN AISC 341), THE WEB SHALL BE VISUALLY INSPECTED FOR CRACKS WITHIN 3 INCHES OF THE WELD. THE INSPECTION SHALL BE PERFORMED NO SOONER THAN 48 HOURS FOLLOWING COMPLETION OF THE WELDING PER AWS D1.8, SECTION 7.4.



ONE-SIDED


TWO-SIDED

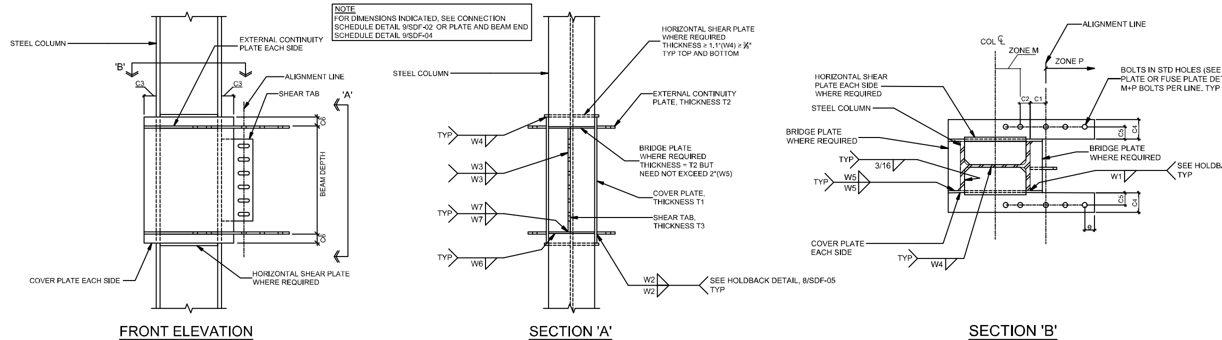
9 **ISOMETRIC CONNECTION VIEW**
SCALE: NTS

PROJECT:	HCAI XXXXXX
NO. / DATE / DESCRIPTION	
ADDITIONAL NOTES:	
STAMP AND SIGNATURE:	

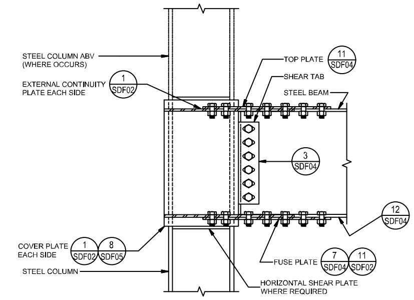
DURAFUSE FRAMES
THE RESILIENT SEISMIC SOLUTION
5801 West Wells Park Rd.
West Jordan, Utah 84081
(801) 727-4060

SHEET TITLE: GENERAL NOTES	
DRAWN BY: XX	DATE: 09/12/2023
CHECKED BY: JM	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00
SHEET NUMBER: SDF-01	

NOTE
FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-02 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04

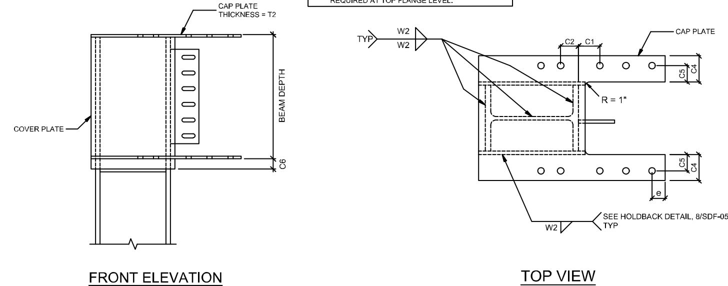


1 ONE SIDED CONNECTION - COLUMN ASSEMBLY
SCALE: NTS



4 ONE SIDED CONNECTION
SCALE: NTS

NOTES
1. FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-02 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04.
2. CONNECTION PER 1/SDF-02 EXCEPT AT TOP FLANGE LEVEL AS SHOWN.
3. SHEAR PLATES AND BRIDGE PLATES ARE NOT REQUIRED AT TOP FLANGE LEVEL.



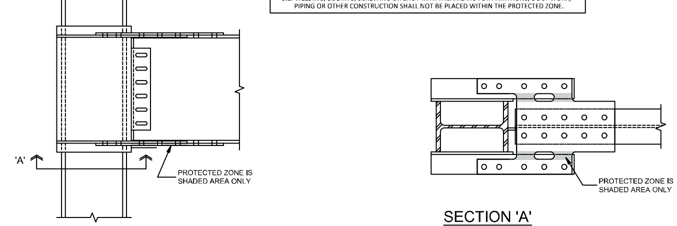
7 TOP-OF-COLUMN CAP PLATE DETAIL FOR ONE SIDED CONNECTION
SCALE: NTS

ID	MEMBER SIZES		PLATE THICKNESS			DIMENSIONS						WELDS							BOLTS PER CONNECTION (EACH SIDE)					
	COLUMN	BEAM	T1	T2	T3	C1	C2	C3	C4	C5	C6	W1	W2	W3	W4	W5	W6	W7	DIA	SPACING	M	P	B	N

NOTES:
1. ALL BOLTS SHALL BE ASTM F3125 GR. A490 OR F2280 PRETENSIONED HIGH-STRENGTH BOLTS W/ THREADS EXCLUDED (X) FROM SHEAR PLANE.
2. ZONE "M" INCLUDES THE BOLTS THAT CONNECT THE TOP PLATE OR FUSE PLATE TO THE EXTERNAL CONTINUITY PLATE BEHIND THE ALIGNMENT LINE.
ZONE "P" INCLUDES THE BOLTS THAT CONNECT THE TOP PLATE OR FUSE PLATE TO THE EXTERNAL CONTINUITY PLATE ON OR BEYOND THE ALIGNMENT LINE.
ZONE "N" INCLUDES THE BOLTS IN THE SHEAR TAB CONNECTION.
ZONE "B" CORRESPONDS TO THE BEAM FLANGE BOLTS (EXCEPT ERECTION AIDS)
3. e = 1-1/8" FOR 3/4" BOLTS, 1-1/4" FOR 7/8" BOLTS, 1-3/8" FOR 1" BOLTS, 1-5/8" FOR 1-1/8" BOLTS, OR 1-3/4" FOR 1-1/4" BOLTS.
4. BOLT SPACING IN TABLE APPLIES TO ALL M, P, AND B BOLTS, UNO.
5. IF W4 EQUALS 0 THEN HORIZONTAL SHEAR PLATES ARE NOT REQUIRED.
6. IF W5, W6, AND W7 EQUAL ZERO THEN BRIDGE PLATES ARE NOT REQUIRED.

9 ONE SIDED CONNECTION SCHEDULE
SCALE: NTS

NOTES
1. THE SHADED AREA IS A PROTECTED ZONE. NO OTHER COMPONENT OF THE CONNECTION, NOR ANY PORTION OF THE BEAMS OR COLUMN ARE A PROTECTED ZONE.
2. PROTECTED ZONE SHALL BE MARKED USING ANY NON-DESTRUCTIVE METHODS PRIOR TO AND AFTER APPLICATION OF SPRAYED ON FIRE-RESISTING MATERIAL.
3. AREA MARKED AS PROTECTED ZONE SHALL COMPLY WITH THE FOLLOWING REQUIREMENTS:
   3.1. WITHIN THE PROTECTED ZONE, DISCONTINUITIES CREATED BY FABRICATION OR ERECTION OPERATIONS, BEYOND THOSE REQUIRED FOR INITIAL FABRICATION OF THE PLATE, SUCH AS TACK WELDS, ERECTION AIDS, AIR-ARC GOUGING AND THERMAL CUTTING SHALL BE REPAIRED. REPAIR METHOD SHALL BE APPROVED BY THE ENGINEER OF RECORD.
   3.2. WELDING, BOLTING, SCREWING OR SHOT-IN ATTACHMENTS FOR PARTITIONS, DUCT WORK, PIPING OR OTHER CONSTRUCTION SHALL NOT BE PLACED WITHIN THE PROTECTED ZONE.



11 ONE SIDED PROTECTED ZONE
SCALE: NTS

PROJECT:
HCAI
XXXXXX

NO.	DATE	DESCRIPTION

ADDITIONAL NOTES:

STAMP AND SIGNATURE:

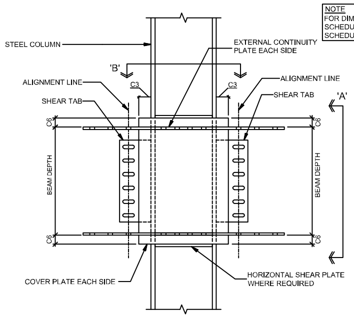
DURAFUSE FRAMES
THE RESILIENT SEISMIC SOLUTION
5801 West Wells Park Rd.
West Jordan, Utah 84081
(801) 727-4060

SHEET TITLE:
ONE-SIDED CONNECTION DETAILS

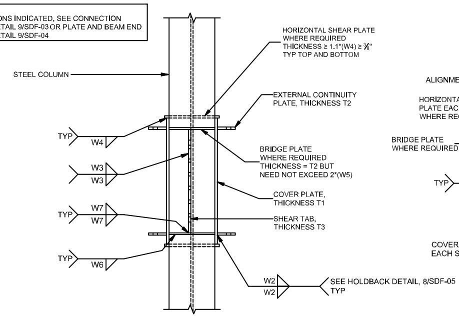
DRAWN BY: XX	DATE: 09/12/2023
CHECKED BY: JM	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

SHEET NUMBER:
SDF-02

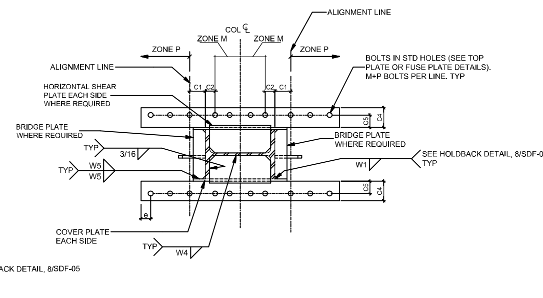
NOTE
FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-03 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04



FRONT ELEVATION

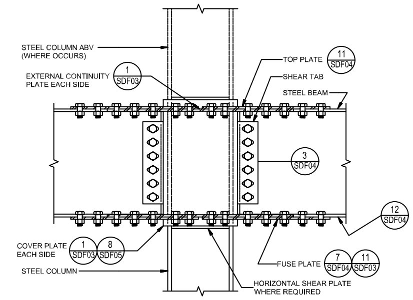


SECTION 'A'



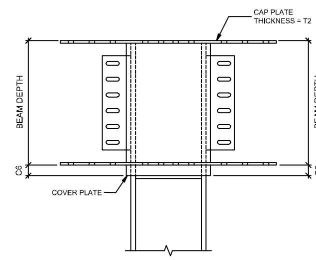
SECTION 'B'

1 TWO SIDED CONNECTION - COLUMN ASSEMBLY
SCALE: NTS

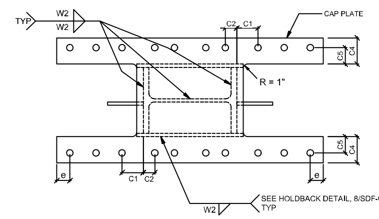


4 TWO SIDED CONNECTION
SCALE: NTS

NOTES
1. FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-03 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04.
2. CONNECTION PER 1/SDF-03 EXCEPT AT TOP FLANGE LEVEL AS SHOWN.
3. SHEAR PLATES AND BRIDGE PLATES ARE NOT REQUIRED AT TOP FLANGE LEVEL.



FRONT ELEVATION



TOP VIEW

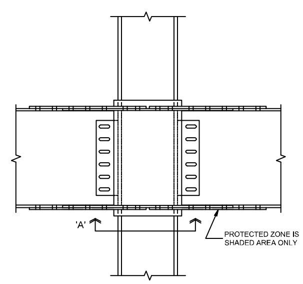
7 TOP-OF-COLUMN DETAIL FOR TWO SIDED CONNECTION
SCALE: NTS

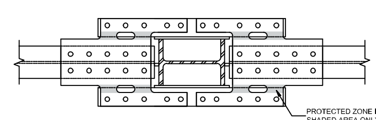
NOTES
1. THE SHADED AREA IS A PROTECTED ZONE. NO OTHER COMPONENT OF THE CONNECTION, NOR ANY PORTION OF THE BEAMS OR COLUMN ARE A PROTECTED ZONE.
2. PROTECTED ZONE SHALL BE MARKED USING ANY NON-DESTRUCTIVE METHODS PRIOR TO AND AFTER APPLICATION OF SPRAYED ON FIRE-RESISTING MATERIAL.
3. AREA MARKED AS PROTECTED ZONE SHALL COMPLY WITH THE FOLLOWING REQUIREMENTS:
   3.1. WITHIN THE PROTECTED ZONE, DISCONTINUITIES CREATED BY FABRICATION OR ERECTION OPERATIONS, BEYOND THOSE REQUIRED FOR INITIAL FABRICATION OF THE PLATE, SUCH AS TACK WELDS, ERECTION AIDS, AIR-ARC GOUGING AND THERMAL CUTTING SHALL BE REPAIRED. REPAIR METHOD SHALL BE APPROVED BY THE ENGINEER OF RECORD.
   3.2. WELDING, BOLTING, SCREWING OR SHOT-IN ATTACHMENTS FOR PARTITIONS, DUCT WORK, PIPING OR OTHER CONSTRUCTION SHALL NOT BE PLACED WITHIN THE PROTECTED ZONE.

ID	MEMBER SIZES		PLATE THICKNESS			DIMENSIONS						WELDS							BOLTS PER CONNECTION (EACH SIDE)					
	COLUMN	BEAM	T1	T2	T3	C1	C2	C3	C4	C5	C6	W1	W2	W3	W4	W5	W6	W7	DIA	SPACING	M	P	B	N

NOTES:
1. ALL BOLTS SHALL BE ASTM F3125 GR. A490 OR F2280 PRETENSIONED HIGH-STRENGTH BOLTS W/ THREADS EXCLUDED (X) FROM SHEAR PLANE.
2. ZONE "M" INCLUDES THE BOLTS THAT CONNECT THE TOP PLATE OR FUSE PLATE TO THE EXTERNAL CONTINUITY PLATE BEHIND THE ALIGNMENT LINE.
ZONE "P" INCLUDES THE BOLTS THAT CONNECT THE TOP PLATE OR FUSE PLATE TO THE EXTERNAL CONTINUITY PLATE ON OR BEYOND THE ALIGNMENT LINE.
ZONE "N" INCLUDES THE BOLTS IN THE SHEAR TAB CONNECTION.
ZONE "B" CORRESPONDS TO THE BEAM FLANGE BOLTS (EXCEPT ERECTION AIDS)
3. e = 1-1/8" FOR 3/4" BOLTS, 1-1/4" FOR 7/8" BOLTS, 1-3/8" FOR 1" BOLTS, 1-5/8" FOR 1-1/8" BOLTS, OR 1-3/4" FOR 1-1/4" BOLTS.
4. BOLT SPACING IN TABLE APPLIES TO ALL M, P, AND B BOLTS, UNO.
5. IF W4 EQUALS 0 THEN HORIZONTAL SHEAR PLATES ARE NOT REQUIRED.
6. IF W5, W6, AND W7 EQUAL ZERO THEN BRIDGE PLATES ARE NOT REQUIRED.

9 TWO SIDED CONNECTION SCHEDULE
SCALE: NTS





SECTION 'A'

11 TWO SIDED PROTECTED ZONE
SCALE: NTS

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SHEET TITLE:
TWO-SIDED CONNECTION DETAILS

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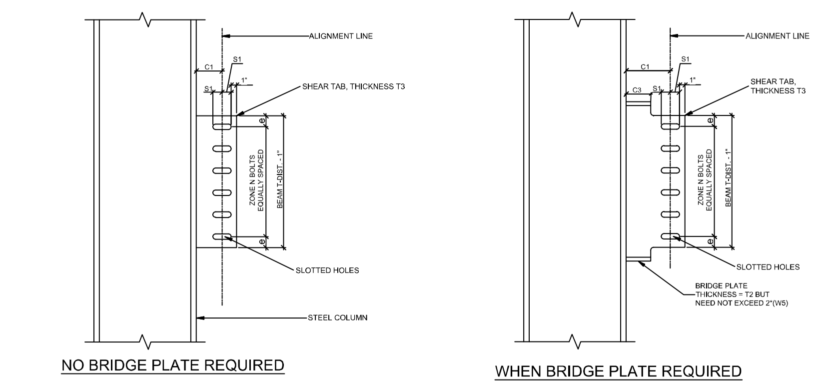
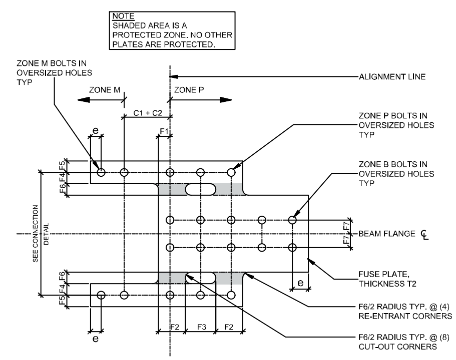
CHECKED BY: JM	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

SHEET NUMBER:
SDF-03

NOTE
FOR DIMENSIONS INDICATED IN TYPICAL DETAILS THIS SHEET, SEE CONNECTION SCHEDULE DETAILS 9/SDF-02 AND 9/SDF-03 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04



3 SHEAR TAB
SCALE: NTS



7 FUSE PLATE
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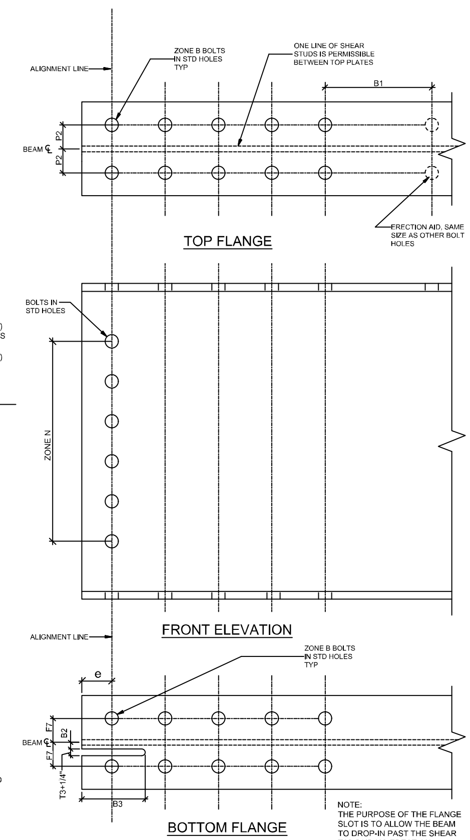

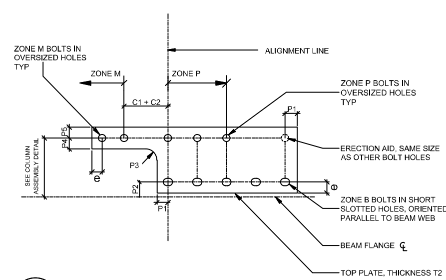

ID	MEMBER SIZES		TOP PLATE DIMENSIONS					FUSE PLATE DIMENSIONS							SHEAR TAB			BEAM DIMENSIONS		
	COLUMN	BEAM	P1	P2	P3	P4	P5	F1	F2	F3	F4	F5	F6	F7	S1	S2	S3	B1	B2	B3

NOTES:
1. SEE CONNECTION SCHEDULE FOR C1, C2, T1, T2, T3, AND NUMBER OF BOLTS IN M, N, P, AND B ZONES.
2. e = 1-1/8" FOR 3/4" BOLTS, 1-1/4" FOR 7/8" BOLTS, 1-3/8" FOR 1" BOLTS, 1-5/8" FOR 1-1/8" BOLTS, OR 1-3/4" FOR 1-1/4" BOLTS.
3. IF B2 AND B3 EQUAL ZERO THEN THERE IS NO SLOT IN THE BEAM FLANGE, AND THE BEAM CAN SWING AROUND THE SHEAR TAB DURING ERECTION.

9 PLATE AND BEAM END SCHEDULE
SCALE: NTS

11 TOP PLATE
SCALE: NTS

12 BEAM END
SCALE: NTS

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SHEET TITLE:
BEAM AND PLATE DETAILS

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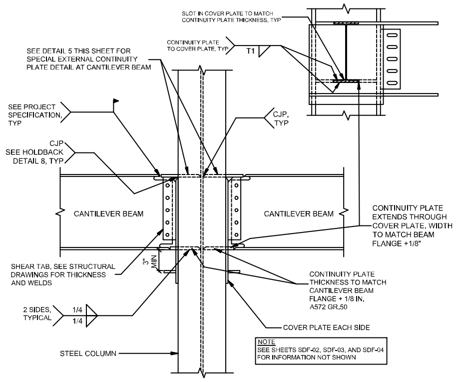
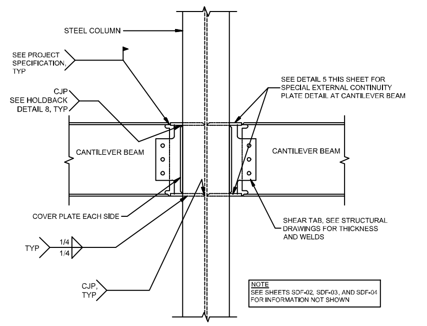
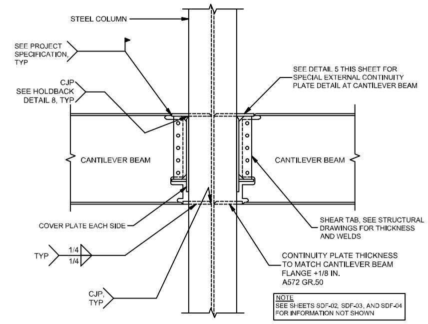
SCALE: NONE	PROJECT NUMBER: XXXXXX-00
SHEET NUMBER: SDF-04	



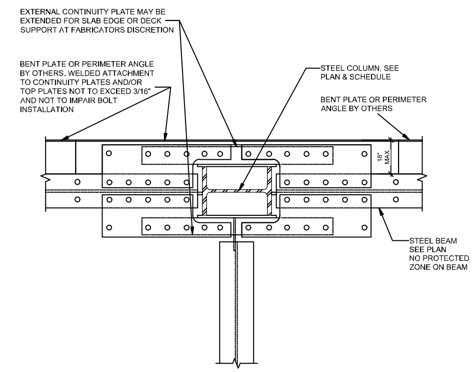
1 CANTILEVER BEAM - CANTILEVER DEPTH < MOMENT FRAME BEAM DEPTH
SCALE: NTS



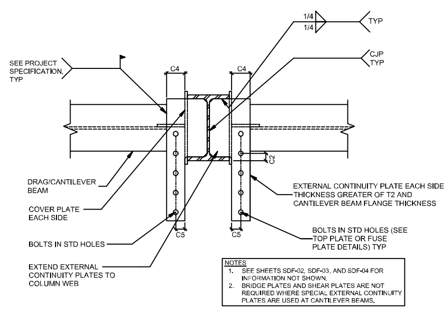
2 CANTILEVER BEAM - CANTILEVER DEPTH = MOMENT FRAME BEAM DEPTH
SCALE: NTS



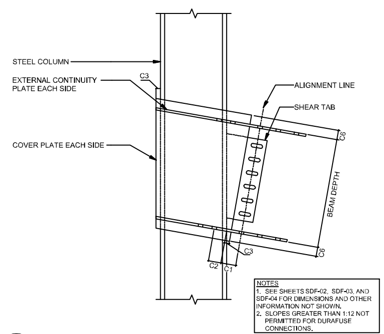
3 CANTILEVER BEAM - CANTILEVER DEPTH > MOMENT FRAME BEAM DEPTH
SCALE: NTS



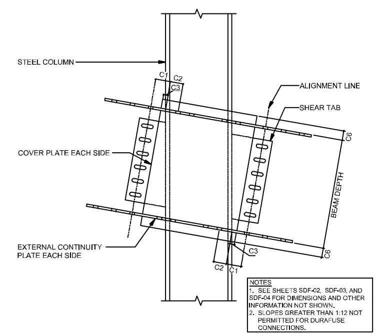
4 SLAB EDGE AND DECK SUPPORT
SCALE: NTS



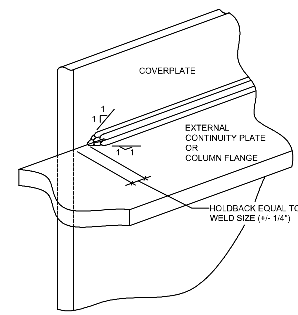
5 SPECIAL EXTERNAL CONTINUITY PLATE AT DRAG/CANTILEVER BEAM
SCALE: NTS



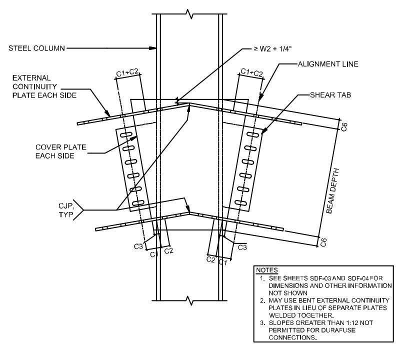
6 ONE SIDED SLOPED CONNECTION
SCALE: NTS



7 TWO SIDED SLOPED CONNECTION
SCALE: NTS



8 HOLDBACK DETAIL
SCALE: NTS



9 TWO SIDED DOUBLE SLOPED CONNECTION
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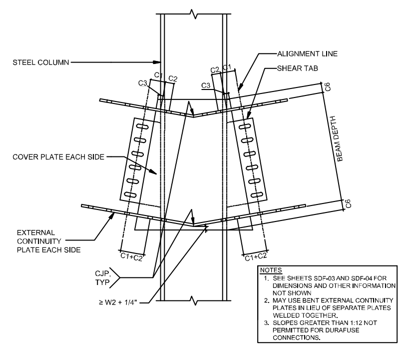

10 TWO SIDED DOUBLE SLOPED CONNECTION
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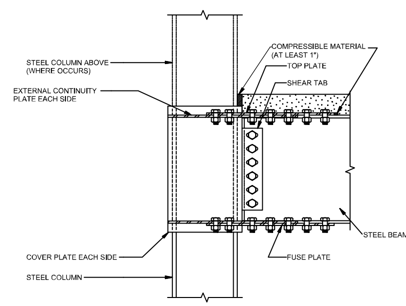
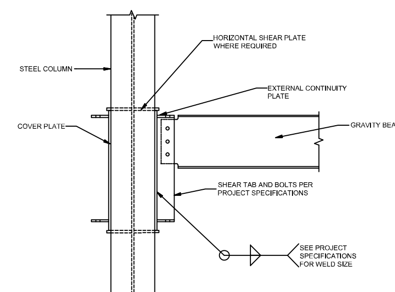


11 COMPRESSIBLE MATERIAL AT SLAB-TO-COLUMN INTERFACE
SCALE: NTS



12 GRAVITY BEAM TO DURAFUSE CONNECTION
SCALE: NTS

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SHEET TITLE:
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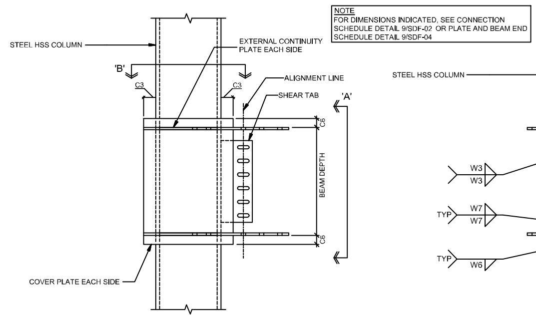
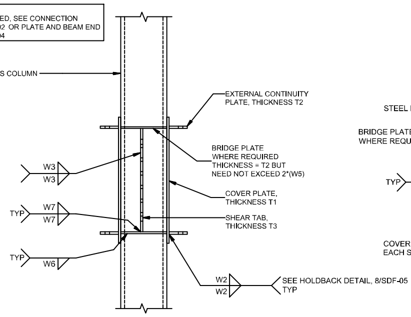
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

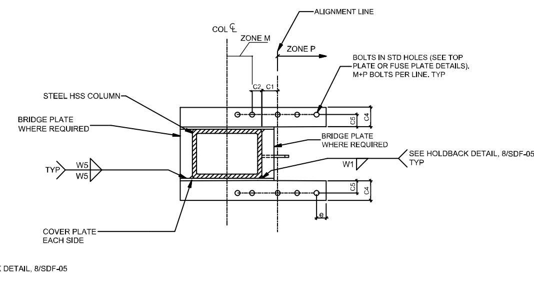
SHEET NUMBER:
SDF-05



FRONT ELEVATION

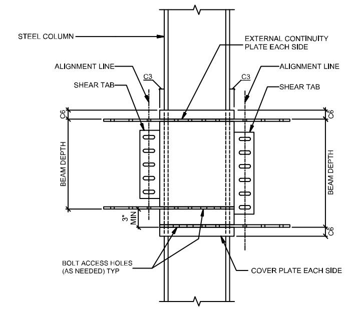


SECTION 'A'

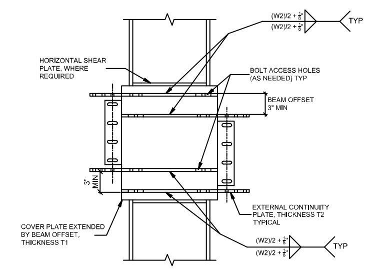


SECTION 'B'

1 ONE SIDED CONNECTION - HSS COLUMN ASSEMBLY
SCALE: NTS



4 TWO SIDED CONNECTION - DIFFERENT BEAMS
SCALE: NTS



FRONT ELEVATION

NOTE
SEE SHEETS SDF-03 AND SDF-04 FOR DIMENSIONS AND OTHER INFORMATION NOT SHOWN

8 DROPPED BEAM CONNECTION - LARGE OFFSET
SCALE: NTS

PROJECT:	
HCAI	XXXXXX

NO.	DATE	DESCRIPTION

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SHEET TITLE:
MISCELLANEOUS DETAILS

DRAWN BY: XX	DATE: 09/12/2023
CHECKED BY: JM	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00
SHEET NUMBER: SDF-06	

8 References

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9 Appendix A – Example Calculations

HCAI CALCULATIONS

CALCULATIONS
SUBMITTAL

Software Version 1.8.68

DURAFUSE FRAMES

THE RESILIENT SEISMIC SOLUTION

9/12/2023

XXXXXX-00

www.durafuseframes.com

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1 GENERAL BUILDING INFORMATION

2 DURAFUSE FRAMES DESIGN INFORMATION

3 DURAFUSE FRAMES DETAILED CONNECTION DESIGN

4 ADDITIONAL EXAMPLE CONNECTIONS

HCAI CALCULATIONS

DuraFuse Connection Calculation Package

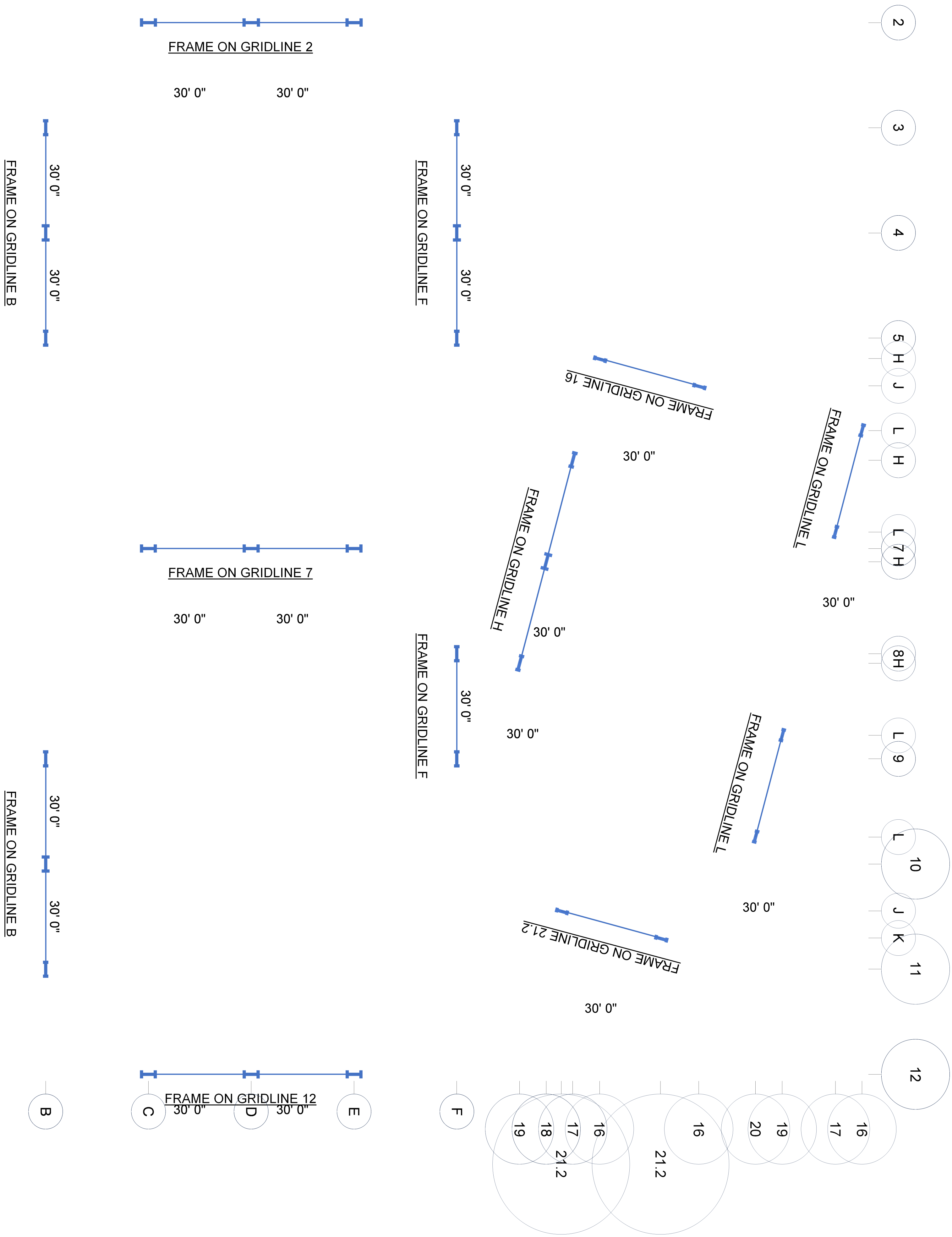
Software Version 1.8.68

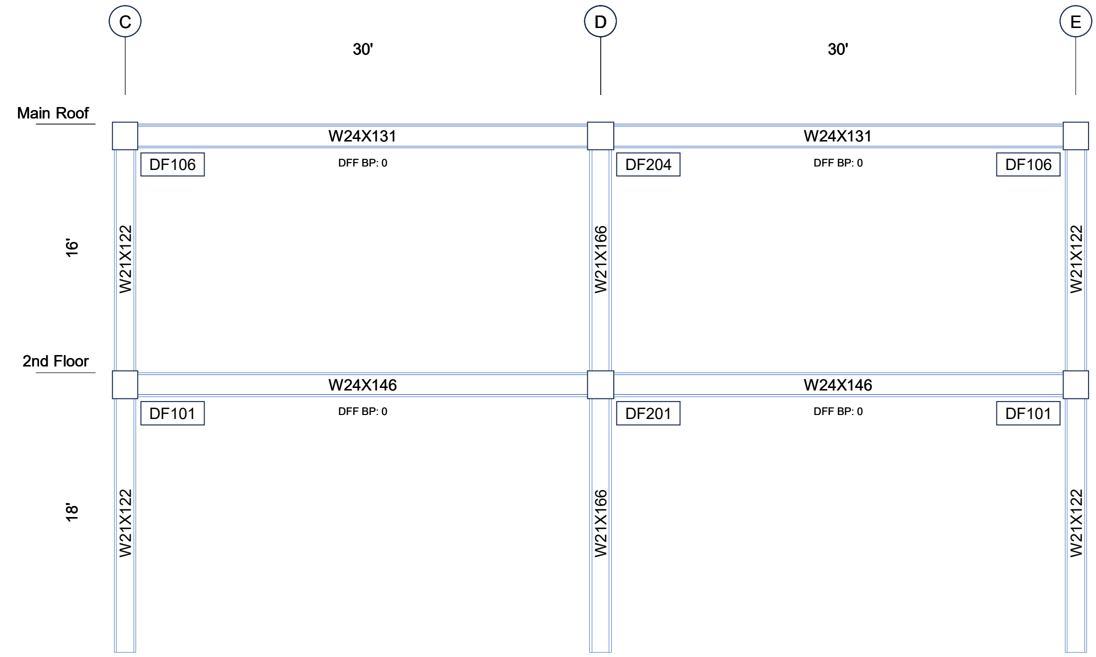
Description

DuraFuse Frames has designed the moment connections for the above listed project. The provided calculations demonstrate satisfaction of the required limit states for each connection. Calculation results are first summarized for all connections, then detailed calculations are presented afterwards for individual connections. DuraFuse Frames is only responsible for the DuraFuse moment connections, and not for the moment frame members, or other moment connections used in the building.

This Calculation Package consists of:

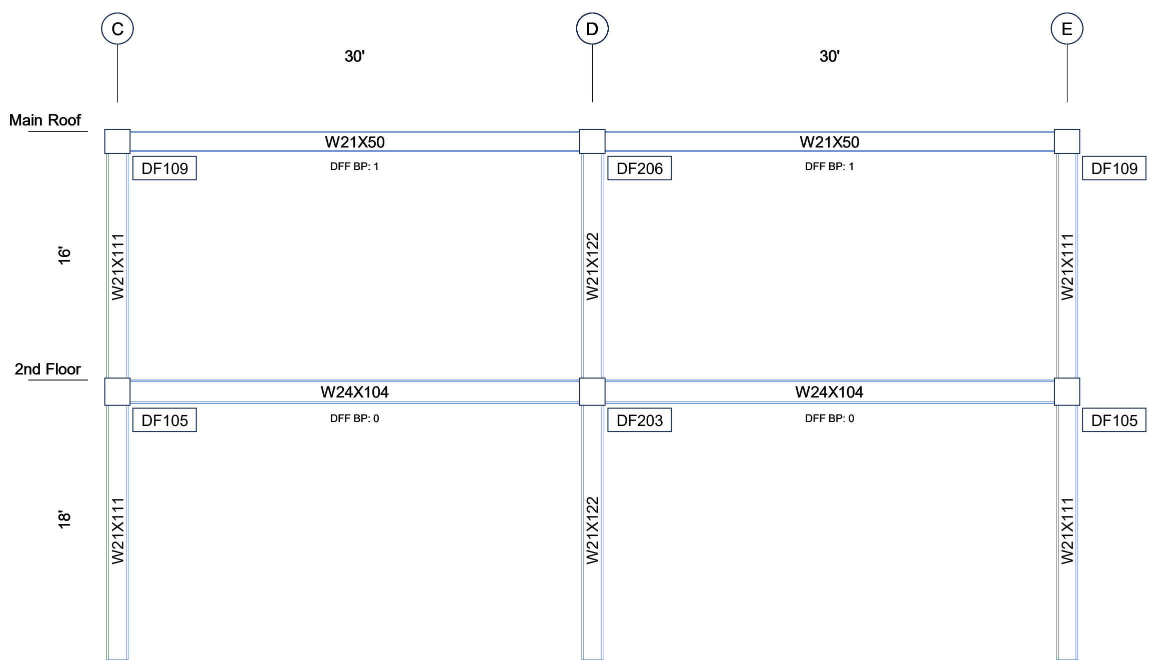
- Demand loads from the EOR
- DuraFuse Connection Diagram
- An Overall Project Limit State Summary
- Individual Connection Limit State Summaries
- Beam Lateral Bracing Calculations
- Selected In-Depth Calculations


FRAME ON GRIDLINE 2
30' 0"
30' 0"
FRAME ON GRIDLINE B
30' 0"
30' 0"
FRAME ON GRIDLINE F
30' 0"
30' 0"
FRAME ON GRIDLINE 16
30' 0"
FRAME ON GRIDLINE L
30' 0"
FRAME ON GRIDLINE H
30' 0"
FRAME ON GRIDLINE 7
30' 0"
30' 0"
FRAME ON GRIDLINE F
30' 0"
30' 0"
FRAME ON GRIDLINE B
30' 0"
30' 0"
FRAME ON GRIDLINE L
30' 0"
FRAME ON GRIDLINE 21.2
30' 0"
FRAME ON GRIDLINE 12
30' 0"
30' 0"
2
3
4
5
H
J
L
H
L
7
H
8H
L
9
L
10
J
K
11
12
B
C
D
E
F
19
18
17
16
21.2
21.2
16
20
19
17
16

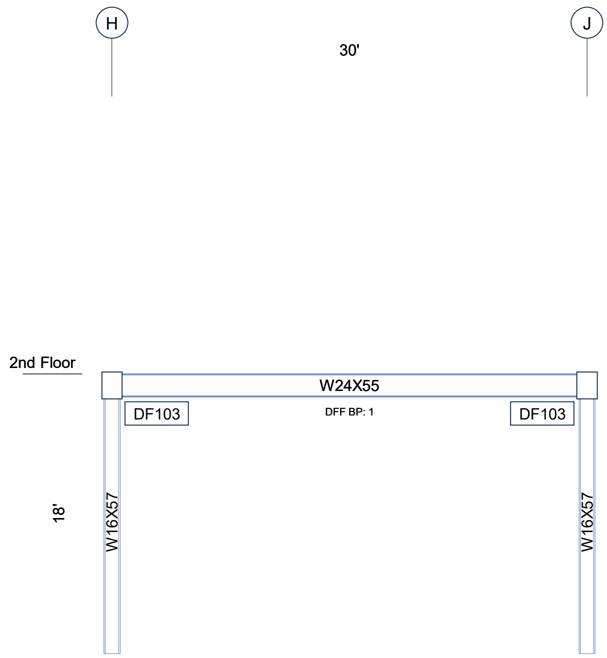


FRAME ON GRIDLINE 2

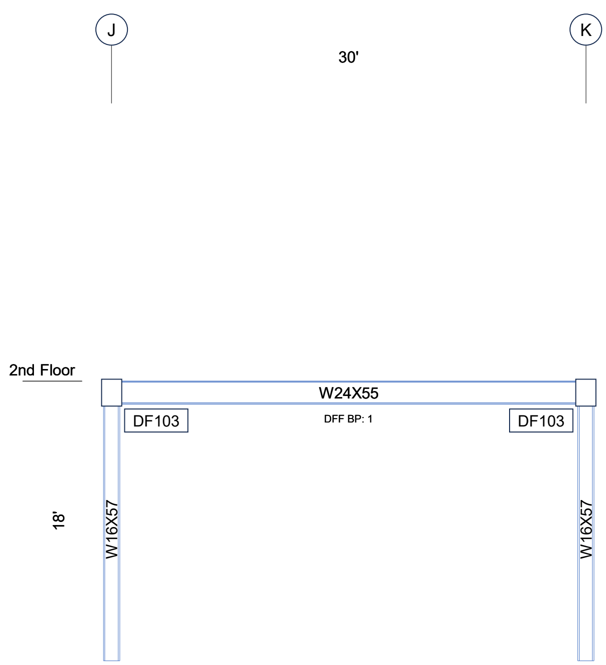
BEAM BRACING KEY:
DFF BP = Number of DuraFuse Frames Beam Brace Points



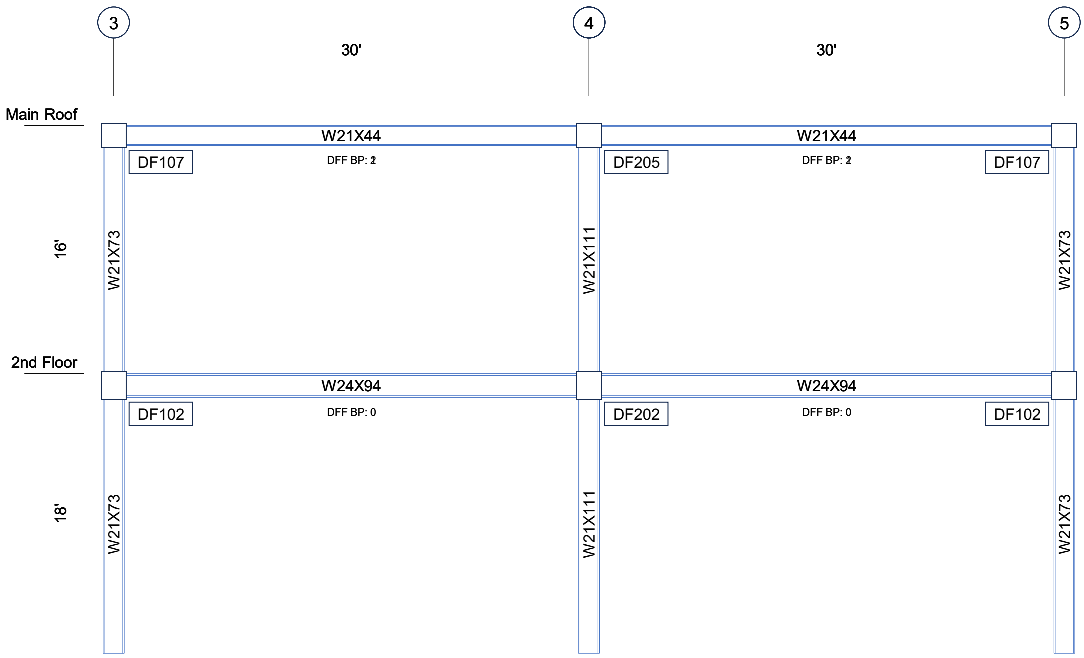
FRAME ON GRIDLINE 12



FRAME ON GRIDLINE 16



FRAME ON GRIDLINE 21.2



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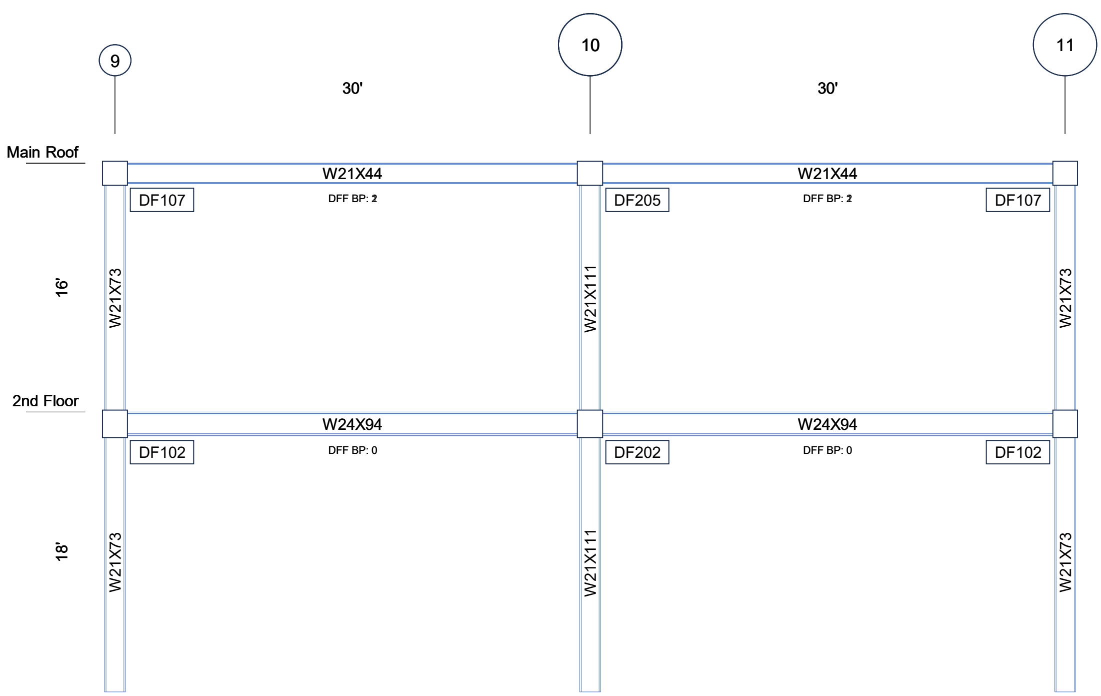

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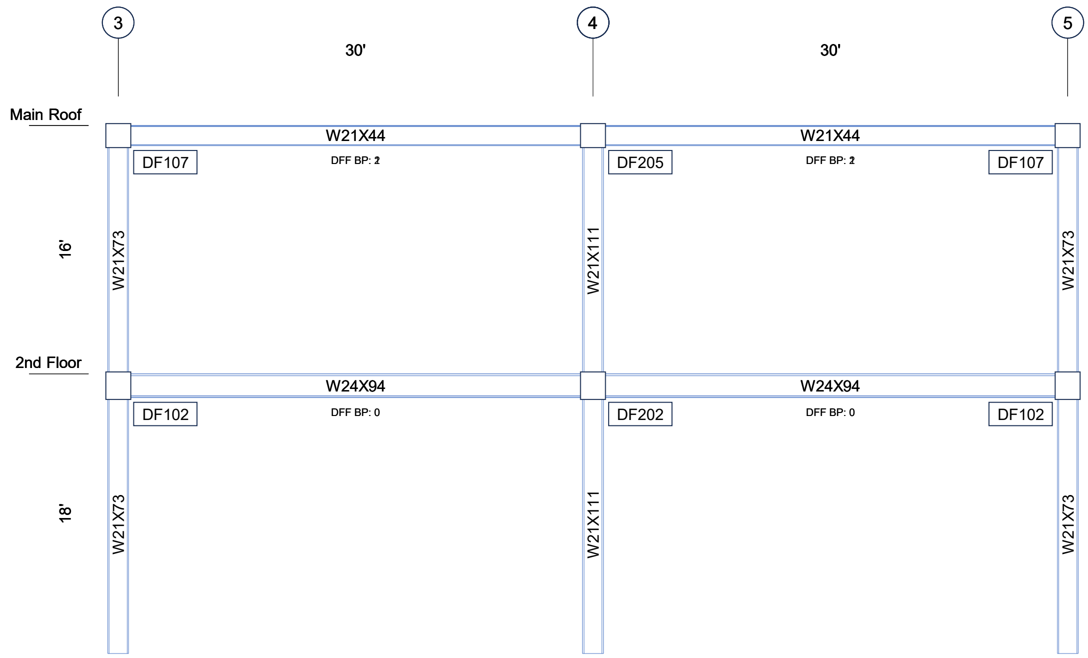

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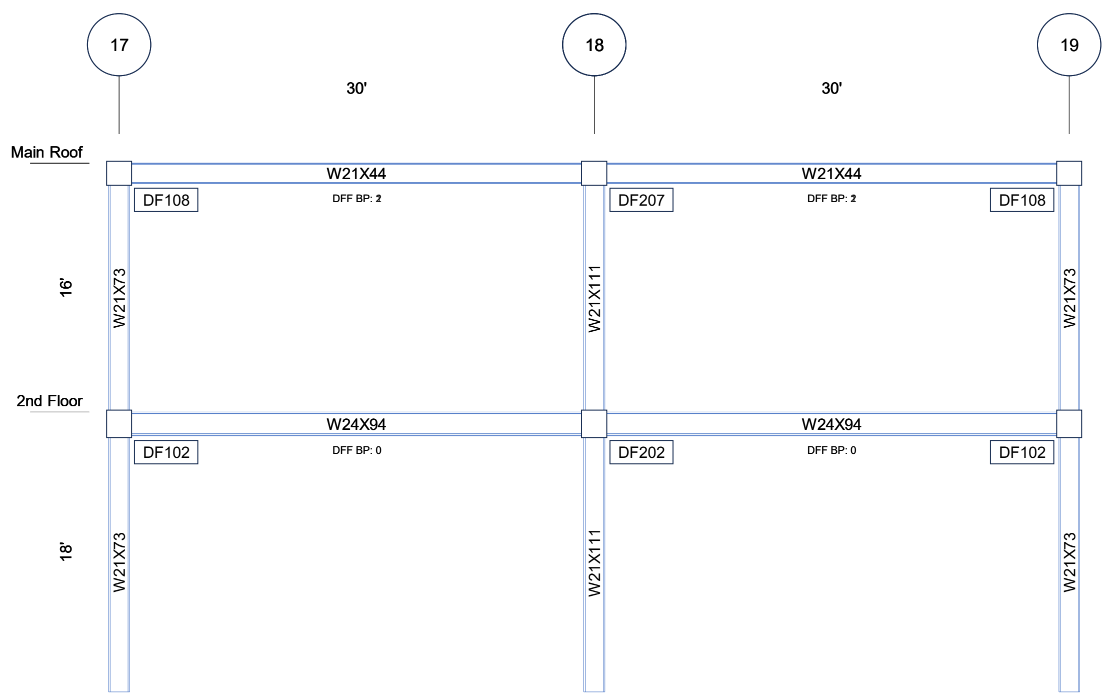

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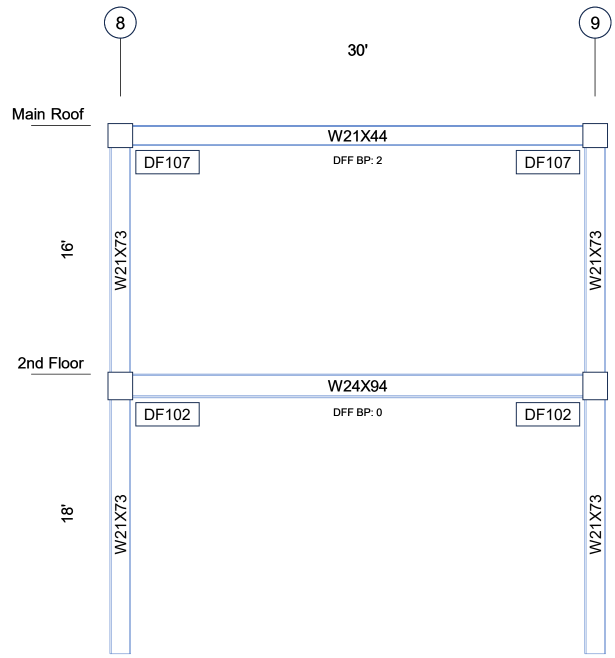

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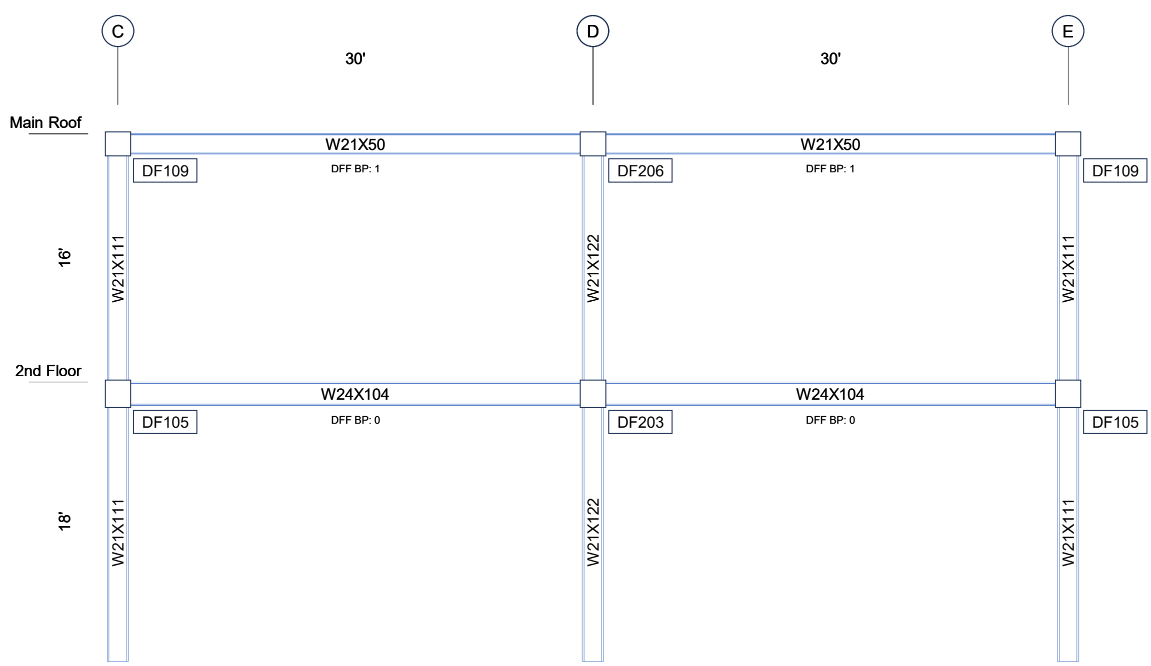

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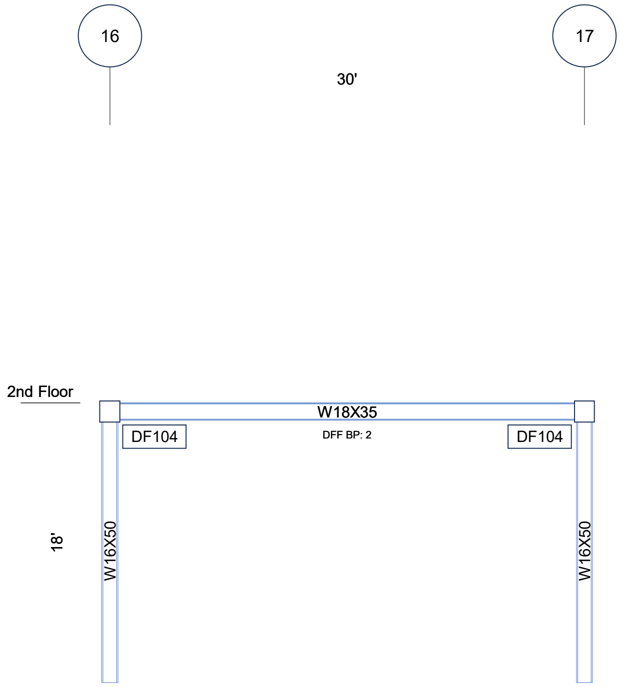

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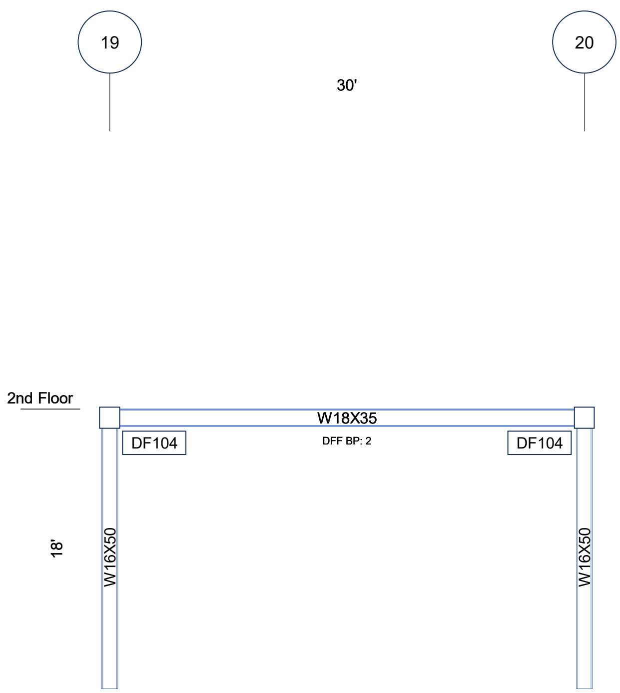


FRAME ON GRIDLINE L

DEMANDS FROM EOR

HCAI Calculations

The following two tables summarize the demands from the EOR that were used for DuraFuse Frames connection design. Applicable demands for the DuraFuse connection include in-plane gravity shear, orthogonal gravity shear, orthogonal gravity moment, column axial compression, in-plane beam moment, in-plane cantilever beam moment, and collector forces. These demands are referenced in the connection design calculations.

The two tables that summarize the demands are: 1) Gravity and Earthquake demand summary and 2) Wind demand summary. Demands are obtained from the Engineer of Record and are factored according to ASCE 7 (see combinations below). Gravity demands are from the maximum of load combinations 1, 2, and 3. Earthquake demands are from load combination 5 and wind demands are from load combination 4. Other load combinations do not govern DuraFuse connection design but must be considered by the Engineer of Record, as required by ASCE 7 and CBC.

An ID has been assigned to each connection in the building (e.g., M101, see Column 1). Connections are then grouped under a global ID, indicated by the bold ID (e.g., **DF101**). These are the DuraFuse connection ID's shown in the schedules and drawings. The final demands used to design each connection ID are the maximum and bold values for grouped designs. The highlighted loads indicate where the controlling load came from. This aproach is conservative because it is combining maximum demands from different joints in the group.

For the columns labled "Orthogonal Gravity Shear" and "Moment", the maximum demand of the two possible orthogonal beams is provided.

Load Combination 1: 1.4D
Load Combination 2: 1.2D + 1.6L + 0.5(Lr or S or R)
Load Combination 3: 1.2D + 1.6(Lr or S or R) + (L or 0.5W)
Load Combination 4: 1.2D + 1.0W + LL + 0.5(LLr or S or R)
Load Combination 5: (1.2 + 0.2SDS) D + ρQE + L + 0.2S

* A minimum of $0.1A_bF_y$ is used in collector design unless the demand from EOR exceeds this minimum value.

GRAVITY & EARTHQUAKE DEMANDS			LC 1,2,3	LC 1,2,3	LC 1,2,3	LC 1,2,3	LC 5	LC 5	LC 1,2,3,5	
Model ID **DF ID**	Framing Level	Grid	DFF Beam 1 Gravity Shear (kips)	Beam 2 Gravity Shear (SMF or gravity beam) (kips)	Orthogonal Gravity Shear (kips)	Orthogonal Gravity Moment (k-ft)	Column Axial Compression (kips)	DFF Beam 1 Moment @ Face of Column (k-ft)	Beam 2 Moment @ Face of Column (SMF or Gravity - Cantilever beam) (k-ft)	In-Plane Collector Force (From EOR) (kips) *
M101	2nd Floor	2 - C	**43.8**	**43.8**	**90.7**		**347.8**	**559.4**		
M102	2nd Floor	2 - E	40.1	40.1	86.2		343.2	557.6		
DF101			**43.8**	**43.8**	**90.7**		**347.8**	**559.4**		
M103	2nd Floor	3 - B	**58.1**	**50.3**	46.5		203.3	423.4		
M104	2nd Floor	3 - F	48.8	45.8	40.8		196.5	406.4		
M105	2nd Floor	5 - B	50.3	50.3	**50.3**		190.3	407.5		
M106	2nd Floor	5 - F	50.1	50.3	44.1		**212.5**	414.6		
M110	2nd Floor	H - 17	15.7	14.7	14.7		145.8	308.6		
M114	2nd Floor	8 - F	51.0	52.1	48.1		204.9	**447.9**		
M115	2nd Floor	H - 19	15.7	15.7	15.7		136.4	303.1		
M117	2nd Floor	9 - B	49.4	49.4	49.4		185.5	390.1		
M118	2nd Floor	9 - F	51.0	46.5	42.4		188.5	445.6		
M122	2nd Floor	11 - B	50.1	50.1	50.1		188.8	398.2		
DF102			**58.1**	**50.3**	**50.3**		**212.5**	**447.9**		
M107	2nd Floor	H - 16	5.4		14.7		**108.8**	67.0		
M108	2nd Floor	J - 16	5.4	23.6	22.6		46.9	**69.4**		
M120	2nd Floor	J - 21.2	**31.5**		**31.5**		35.4	67.8		
M121	2nd Floor	K - 21.2	4.3		26.3		32.2	67.3		
DF103			**31.5**		**31.5**		**108.8**	**69.4**		
M109	2nd Floor	L - 16	**27.6**		**27.6**		**33.9**	61.4		
M111	2nd Floor	L - 17	8.4	13.2	4.6		24.2	61.0		
M116	2nd Floor	L - 19	8.4	13.3	4.6		24.5	**62.7**		
M119	2nd Floor	L - 20	8.4	8.3	4.6		21.5	61.9		
DF104			**27.6**		**27.6**		**33.9**	**62.7**		
M112	2nd Floor	7 - C	**45.9**	**42.4**	**89.5**		**305.7**	452.3		
M113	2nd Floor	7 - E	45.9	45.9	89.5		305.7	**452.4**		
M123	2nd Floor	12 - C	27.8	26.8	89.5		210.3	388.7		
M124	2nd Floor	12 - E	27.8	27.8	89.5		210.7	391.5		
DF105			**45.9**	**42.4**	**89.5**		**305.7**	**452.4**		

GRAVITY & EARTHQUAKE DEMANDS (Continued)

Model ID **DF ID**	Framing Level	Grid	DFF Beam 1 Gravity Shear (kips)	Beam 2 Gravity Shear (SMF or gravity beam) (kips)	Orthogonal Gravity Shear (kips)	Orthogonal Gravity Moment (k-ft)	Column Axial Compression (kips)	DFF Beam 1 Moment @ Face of Column (k-ft)	Beam 2 Moment @ Face of Column (SMF or Gravity - Cantilever beam) (k-ft)	In-Plane Collector Force (From EOR) (kips) *
M125	Main Roof	2 - C	7.3	**4.1**	28.9	115.7	70.3	**225.8**		
M126	Main Roof	2 - E	**7.3**	7.8	**36.7**	**146.9**	**76.5**	223.9		
DF106			**7.3**	**4.1**	**36.7**	**146.9**	**76.5**	**225.8**		
M127	Main Roof	3 - B	16.4		14.4		38.6	132.9		
M128	Main Roof	3 - F	25.2	33.5	7.8		54.9	153.2		
M129	Main Roof	5 - B	16.2	16.2	16.2		39.6	133.3		
M130	Main Roof	5 - F	**34.1**	33.9	7.3		60.7	**167.0**		
M131	Main Roof	H - 17	29.9	19.2	**29.9**	**239.1**	**65.6**	153.9		
M134	Main Roof	8 - F	18.5	19.6	7.3		43.0	138.1		
M135	Main Roof	H - 19	12.5	28.1	28.1	224.8	56.4	141.2		
M136	Main Roof	9 - B	14.8	12.7	12.7		38.2	122.7		
M137	Main Roof	9 - F	16.9	17.1	7.3		35.4	133.6		
M138	Main Roof	11 - B	14.8	14.8	14.8		40.0	119.5		
DF107			**34.1**		**29.9**	**239.1**	**65.6**	**167.0**		
M132	Main Roof	7 - C	**8.7**	**7.3**	22.7		60.4	136.2		
M133	Main Roof	7 - E	8.7	8.7	**24.2**		**63.3**	**136.4**		
M139	Main Roof	12 - C	7.3	7.2	23.0		46.5	119.7		
M140	Main Roof	12 - E	7.3	7.3	23.0		46.7	122.2		
DF108			**8.7**	**7.3**	**24.2**		**63.3**	**136.4**		
M201	2nd Floor	2 - D	**40.1**	**40.1**	**89.5**		**224.7**	**610.0**	**606.5**	
DF201			**40.1**	**40.1**	**89.5**		**224.7**	**610.0**	**606.5**	
M202	2nd Floor	4 - B	**52.7**	**52.7**	**52.7**		135.7	**507.1**	**501.4**	
M203	2nd Floor	4 - F	49.5	50.1	44.1		**156.7**	489.6	489.7	
M205	2nd Floor	H - 18	15.7	15.7	15.7		69.5	315.9	318.3	
M206	2nd Floor	10 - B	45.0	50.1	45.0		121.7	468.4	485.6	
DF202			**52.7**	**52.7**	**52.7**		**156.7**	**507.1**	**501.4**	
M204	2nd Floor	7 - D	**45.9**	**45.9**	**89.5**		**224.1**	**476.1**	**475.9**	
M207	2nd Floor	12 - D	27.8	27.8	89.5		129.0	391.9	393.9	
DF203			**45.9**	**45.9**	**89.5**		**224.1**	**476.1**	**475.9**	
M208	Main Roof	2 - D	**7.8**	**7.3**	**36.6**	**146.4**	**38.0**	**220.4**	**216.5**	
DF204			**7.8**	**7.3**	**36.6**	**146.4**	**38.0**	**220.4**	**216.5**	
M209	Main Roof	4 - B	16.4	17.1	**7.3**		28.5	139.2	145.0	
M210	Main Roof	4 - F	**39.7**	27.0	7.3		**52.2**	**216.5**	**193.8**	
M212	Main Roof	H - 18	15.8	**28.7**	28.7	229.6	40.3	165.4	158.2	
M213	Main Roof	10 - B	14.8	14.8	14.8		26.2	127.6	130.0	
DF205			**39.7**	**28.7**	**7.3**		**52.2**	**216.5**	**193.8**	
M211	Main Roof	7 - D	**8.7**	**8.7**	**24.6**		**38.8**	**132.2**	**132.4**	
M214	Main Roof	12 - D	7.3	7.3	23.1		26.8	122.8	121.2	
DF206			**8.7**	**8.7**	**24.6**		**38.8**	**132.2**	**132.4**	

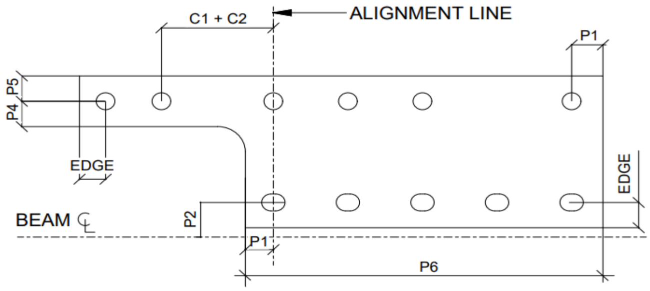
WIND DEMANDS

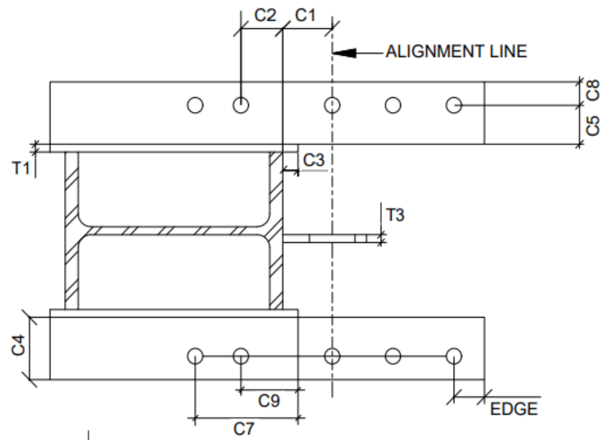
Model ID **DF ID**	Framing Level	Grid	LC 4 Column Axial Compression (kips)	LC 4 DFF Beam 1 Moment @ Face of Column (k-ft)	LC 4 Beam 2 Moment @ Face of Column (SMF or Gravity - Cantilever beam) (k-ft)	LC 4 In-Plane Collector Force (From EOR) (kips) *
M101	2nd Floor	2 - C	201.5	86.9		
M102	2nd Floor	2 - E	196.8	83.6		
DF101			**201.5**	**86.9**		
M103	2nd Floor	3 - B	125.2	143.6		
M104	2nd Floor	3 - F	122.2	129.8		
M105	2nd Floor	5 - B	112.9	127.5		
M106	2nd Floor	5 - F	**138.2**	137.2		
M110	2nd Floor	H - 17	70.8	47.6		
M114	2nd Floor	8 - F	125.5	**159.6**		
M115	2nd Floor	H - 19	60.7	41.5		
M117	2nd Floor	9 - B	111.5	118.6		
M118	2nd Floor	9 - F	109.5	156.2		
M122	2nd Floor	11 - B	114.9	127.0		
DF102			**138.2**	**159.6**		
M107	2nd Floor	H - 16	**101.2**	9.8		
M108	2nd Floor	J - 16	36.3	**12.3**		
M120	2nd Floor	J - 21.2	24.9	9.4		
M121	2nd Floor	K - 21.2	21.8	8.7		
DF103			**101.2**	**12.3**		
M109	2nd Floor	L - 16	**27.2**	25.6		
M111	2nd Floor	L - 17	18.0	25.0		
M116	2nd Floor	L - 19	18.1	**25.7**		
M119	2nd Floor	L - 20	14.9	24.8		
DF104			**27.2**	**25.7**		
M112	2nd Floor	7 - C	**204.1**	**91.2**		
M113	2nd Floor	7 - E	203.9	91.1		
M123	2nd Floor	12 - C	120.1	57.1		
M124	2nd Floor	12 - E	120.4	59.6		
DF105			**204.1**	**91.2**		
M125	Main Roof	2 - C	30.2	**25.5**		
M126	Main Roof	2 - E	**35.9**	21.5		
DF106			**35.9**	**25.5**		
M127	Main Roof	3 - B	22.8	53.5		
M128	Main Roof	3 - F	41.1	79.2		
M129	Main Roof	5 - B	23.9	53.1		
M130	Main Roof	5 - F	**47.8**	**95.1**		
M131	Main Roof	H - 17	44.7	48.7		
M134	Main Roof	8 - F	28.4	62.4		
M135	Main Roof	H - 19	35.1	35.6		
M136	Main Roof	9 - B	24.0	49.9		
M137	Main Roof	9 - F	20.9	56.9		
M138	Main Roof	11 - B	25.9	47.6		
DF107			**47.8**	**95.1**		
M132	Main Roof	7 - C	39.3	23.4		
M133	Main Roof	7 - E	**41.9**	23.4		
M139	Main Roof	12 - C	27.6	22.4		
M140	Main Roof	12 - E	27.8	**24.6**		
DF108			**41.9**	**24.6**		
M201	2nd Floor	2 - D	**215.8**	**160.5**		
DF201			**215.8**	**160.5**		

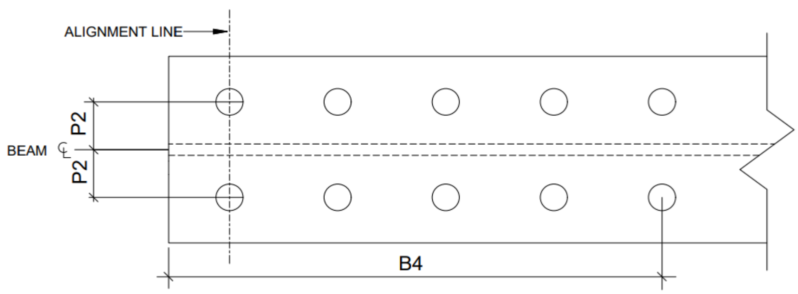
WIND DEMANDS (Continued)

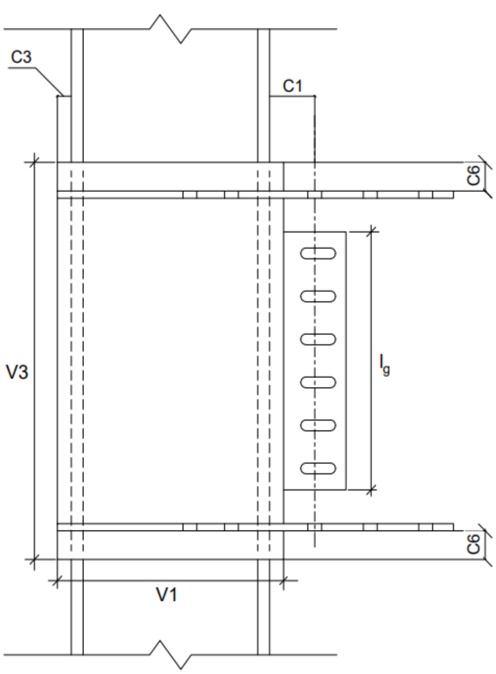
Model ID **DF ID**	Framing Level	Grid	Column Axial Compression (kips)	DFF Beam 1 Moment @ Face of Column (k-ft)	Beam 2 Moment @ Face of Column (SMF or Gravity - Cantilever beam) (k-ft)	In-Plane Collector Force (From EOR) (kips) *
M203	2nd Floor	4 - F	**153.9**	217.4		
M205	2nd Floor	H - 18	72.4	66.6		
M206	2nd Floor	10 - B	118.6	202.9		
DF202			**153.9**	**232.1**		
M204	2nd Floor	7 - D	**218.4**	**144.8**		
M207	2nd Floor	12 - D	125.8	89.2		
DF203			**218.4**	**144.8**		
M208	Main Roof	2 - D	**39.2**	**31.3**		
DF204			**39.2**	**31.3**		
M209	Main Roof	4 - B	30.6	63.4		
M210	Main Roof	4 - F	**54.9**	**148.4**		
M212	Main Roof	H - 18	42.1	64.1		
M213	Main Roof	10 - B	28.0	58.3		
DF205			**54.9**	**148.4**		
M211	Main Roof	7 - D	**41.8**	23.7		
M214	Main Roof	12 - D	28.5	**28.4**		
DF206			**41.8**	**28.4**		

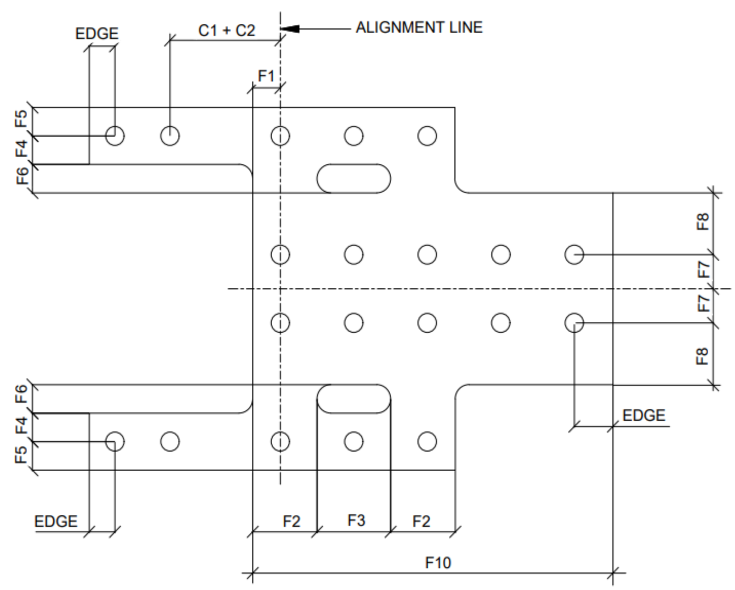
HCAI CALCULATIONS
DIMENSION DIAGRAMS











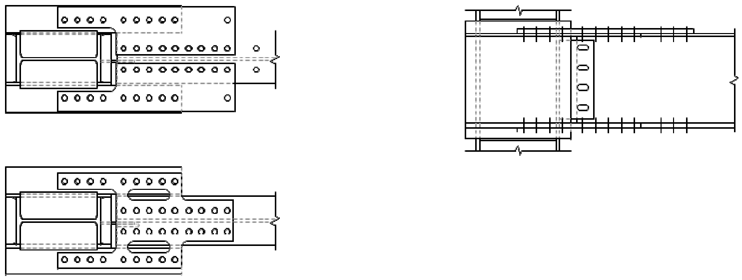
PROJECT SUMMARY
HCAI CALCULATIONS

ID	Count	Column	Beam	Controlling D/C Ratio
DF101	2	W21X122	W24X146	0.99
DF102	10	W21X73	W24X94	0.99
DF103	4	W16X57	W24X55	0.96
DF104	4	W16X50	W18X35	0.99
DF105	4	W21X111	W24X104	1.00
DF106	2	W21X122	W24X131	0.99
DF107	8	W21X73	W21X44	0.98
DF108	2	W21X73	W21X44	0.98
DF109	4	W21X111	W21X50	1.00
DF201	1	W21X166	W24X146	1.00
DF202	4	W21X111	W24X94	1.00
DF203	2	W21X122	W24X104	1.00
DF204	1	W21X166	W24X131	0.98
DF205	3	W21X111	W21X44	0.98
DF206	2	W21X122	W21X50	1.00
DF207	1	W21X111	W21X44	0.98

See following pages for more information

DF101 Limit State Summary

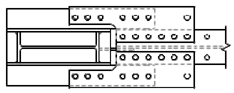
Column: W21X122 - Beam: W24X146

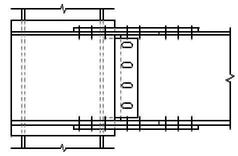


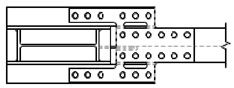
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 5.9	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 6.4	OK
	Column-beam moment ratio	ΣM^*_{pc} = 36076	ΣM^*_{pb} = 22716	OK
2	Maximum probable moment	M_u = 8400 k-in	M_{pr} = 20900 k-in	0.40
3	Cover plate shear yielding	R_u = 501.4 kips	ϕR_n = 623.2 kips	0.80
	Cover plate thickness	t_{cp} = 0.75 in	t_{req} = 0.47 in	OK
4	Beam net section check	M_{pe} = 22990 k-in	M_{fr} = 24827 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 501.4 kips	ϕR_n = 563.6 kips	0.89
	Bolt slip for wind load	R_u = 78.9 kips	ϕR_n = 207.5 kips	0.38
6	Alignment line location, C1	$C1_{req}$ = 6.1 in	C1 = 6.1 in	OK
7	Weld 1 strength	r_u = 9.7 kip/in	ϕr_n = 11.4 kip/in	0.86
8	Weld 2 strength	r_u = 25.4 kip/in	ϕr_n = 27.8 kip/in	0.91
9	External continuity plate rupture: Mode 1	P_u = 278.6 kips	ϕP_n = 318.8 kips	0.87
	External continuity plate rupture: Mode 2	P_u = 334.3 kips	ϕP_n = 337.8 kips	0.99
	External continuity plate rupture: Mode 3	P_u = 390 kips	ϕP_n = 446.4 kips	0.87
	External continuity plate rupture: Mode 4	R_u = 334.3 kips	ϕR_n = 372.6 kips	0.90
	External continuity plate rupture: Mode 5	R_u = 390 kips	ϕR_n = 458 kips	0.85
10	Beam shear	V_u = 167.4 kips	ϕV_n = 481.6 kips	0.35
11	Beam block shear	R_u = 1002.8 kips	ϕR_n = 1437.4 kips	0.70
12	Web bolt shear	R_u = 167.4 kips	ϕR_n = 250.5 kips	0.67
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.35
	Shear tab weld failure	r_u = 10.4 kips	ϕr_n = 11.1 kips	0.93
15	Top plate for shear yielding	R_u = 501.4 kips	ϕR_n = 1037.8 kips	0.48
	Top plate for shear rupture	R_u = 501.4 kips	ϕR_n = 567.6 kips	0.88
16	Top plate for tensile rupture in extensions	P_u = 222.8 kips	ϕP_n = 226.2 kips	0.99
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -6.8 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.4	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 359.7 kips	ϕP_n = 551.8 kips	0.65
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 359.7 kips	ϕP_n = 519.9 kips	0.69
19	Fuse yielding region dimension	$F2_{max}$ = 3.1 in	F2 = 3.1 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 0.9	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 359.7 kips	ϕR_n = 430.3 kips	0.84
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 359.7 kips	ϕR_n = 362 kips	0.99
Misc	Bridge plate tension failure	P_u = 66 kips	ϕP_n = 90 kips	0.73
	Bridge plate to cover plate weld failure	R_u = 66 kips	ϕR_n = 66.8 kips	0.99
	Shear tab to bridge plate weld failure	R_u = 41.5 kips	ϕR_n = 41.8 kips	0.99
	Bridge plate to column flange weld failure	R_u = 41.5 kips	ϕR_n = 62.6 kips	0.66
	Shear plate shear failure	R_u = 165.2 kips	ϕR_n = 296.7 kips	0.56
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.99

DF102 Limit State Summary

Column: W21X73 - Beam: W24X94



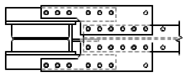


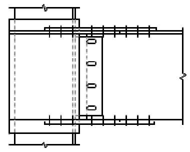


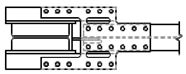
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 5.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5.6	OK
	Column-beam moment ratio	ΣM^*_{pc} = 19918	ΣM^*_{pb} = 14110	OK
2	Maximum probable moment	M_u = 6424 k-in	M_{pr} = 12700 k-in	0.51
3	Cover plate shear yielding	R_u = 313.3 kips	ϕR_n = 612 kips	0.51
	Cover plate thickness	t_{cp} = 0.75 in	t_{req} = 0.47 in	OK
4	Beam net section check	M_{pe} = 13970 k-in	M_{fr} = 14131 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 313.3 kips	ϕR_n = 375.7 kips	0.83
	Bolt slip for wind load	R_u = 63.4 kips	ϕR_n = 138.3 kips	0.46
6	Alignment line location, C1	$C1_{req}$ = 6.1 in	C1 = 6.1 in	OK
7	Weld 1 strength	r_u = 10.5 kip/in	ϕr_n = 11.4 kip/in	0.92
8	Weld 2 strength	r_u = 15.1 kip/in	ϕr_n = 16.7 kip/in	0.91
9	External continuity plate rupture: Mode 1	P_u = 156.7 kips	ϕP_n = 185.9 kips	0.84
	External continuity plate rupture: Mode 2	P_u = 208.9 kips	ϕP_n = 212.4 kips	0.98
	External continuity plate rupture: Mode 3	P_u = 261.1 kips	ϕP_n = 332.2 kips	0.79
	External continuity plate rupture: Mode 4	R_u = 208.9 kips	ϕR_n = 242.1 kips	0.86
	External continuity plate rupture: Mode 5	R_u = 261.1 kips	ϕR_n = 318 kips	0.82
10	Beam shear	V_u = 133.1 kips	ϕV_n = 375.4 kips	0.35
11	Beam block shear	R_u = 626.6 kips	ϕR_n = 673.8 kips	0.93
12	Web bolt shear	R_u = 133.1 kips	ϕR_n = 250.5 kips	0.53
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.43
	Shear tab weld failure	r_u = 9.4 kips	ϕr_n = 11.1 kips	0.85
15	Top plate for shear yielding	R_u = 313.3 kips	ϕR_n = 611.2 kips	0.51
	Top plate for shear rupture	R_u = 313.3 kips	ϕR_n = 332.7 kips	0.94
16	Top plate for tensile rupture in extensions	P_u = 156.7 kips	ϕP_n = 158.4 kips	0.99
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -2.3 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 1.5	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 251 kips	ϕP_n = 396 kips	0.63
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 251 kips	ϕP_n = 339.3 kips	0.74
19	Fuse yielding region dimension	$F2_{max}$ = 2 in	F2 = 2 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 0.8	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 251 kips	ϕR_n = 337.5 kips	0.74
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 251 kips	ϕR_n = 263.2 kips	0.95
Misc	Bridge plate tension failure	P_u = 37.2 kips	ϕP_n = 56.2 kips	0.66
	Bridge plate to cover plate weld failure	R_u = 37.2 kips	ϕR_n = 41.8 kips	0.89
	Shear tab to bridge plate weld failure	R_u = 33.5 kips	ϕR_n = 41.8 kips	0.80
	Bridge plate to column flange weld failure	R_u = 33.5 kips	ϕR_n = 50.1 kips	0.67
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.99

DF103 Limit State Summary

Column: W16X57 - Beam: W24X55



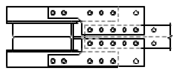


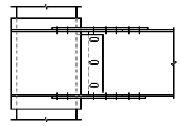


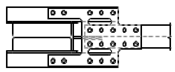
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 6.9	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5	OK
	Column-beam moment ratio	ΣM^*_{pc} = 6214	ΣM^*_{pb} = 6472	NA
2	Maximum probable moment	M_u = 1046 k-in	M_{pr} = 5931 k-in	0.18
3	Cover plate shear yielding	R_u = 158.2 kips	ϕR_n = 414 kips	0.38
	Cover plate thickness	t_{cp} = 0.75 in	t_{req} = 0.417 in	OK
4	Beam net section check	M_{pe} = 7370 k-in	M_{fr} = 7809 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 158.2 kips	ϕR_n = 227.3 kips	0.70
	Bolt slip for wind load	R_u = 27.5 kips	ϕR_n = 84.7 kips	0.32
6	Alignment line location, C1	$C1_{req}$ = 4.2 in	C1 = 2.6 in	OK
7	Weld 1 strength	r_u = 9.1 kip/in	ϕr_n = 9.7 kip/in	0.94
8	Weld 2 strength	r_u = 14.9 kip/in	ϕr_n = 16.7 kip/in	0.89
9	External continuity plate rupture: Mode 1	P_u = 79.1 kips	ϕP_n = 100.5 kips	0.79
	External continuity plate rupture: Mode 2	P_u = 105.5 kips	ϕP_n = 111.7 kips	0.94
	External continuity plate rupture: Mode 3	P_u = 131.9 kips	ϕP_n = 183 kips	0.72
	External continuity plate rupture: Mode 4	R_u = 105.5 kips	ϕR_n = 130.1 kips	0.81
	External continuity plate rupture: Mode 5	R_u = 131.9 kips	ϕR_n = 180.7 kips	0.73
10	Beam shear	V_u = 66.1 kips	ϕV_n = 279.7 kips	0.24
11	Beam block shear	R_u = 316.5 kips	ϕR_n = 346.8 kips	0.91
12	Web bolt shear	R_u = 66.1 kips	ϕR_n = 151.5 kips	0.44
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.07
	Shear tab weld failure	r_u = 13.3 kips	ϕr_n = 13.9 kips	0.95
15	Top plate for shear yielding	R_u = 158.2 kips	ϕR_n = 421.9 kips	0.38
	Top plate for shear rupture	R_u = 158.2 kips	ϕR_n = 263.2 kips	0.60
16	Top plate for tensile rupture in extensions	P_u = 79.1 kips	ϕP_n = 82.3 kips	0.96
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -8.7 in	P2 = 2.4 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 121.8 kips	ϕP_n = 248.7 kips	0.49
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 121.8 kips	ϕP_n = 224.6 kips	0.54
19	Fuse yielding region dimension	$F2_{max}$ = 1.4 in	F2 = 1.4 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 1.1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 121.8 kips	ϕR_n = 194.1 kips	0.63
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 121.8 kips	ϕR_n = 153.6 kips	0.79
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	K_{req} = 1957500 k/in	K_S = 5203652 k/in	OK
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.96

DF104 Limit State Summary

Column: W16X50 - Beam: W18X35



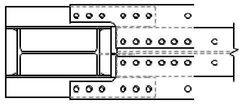


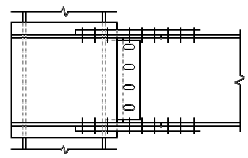


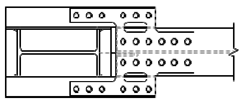
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 7.1	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5.6	OK
	Column-beam moment ratio	ΣM^*_{pc} = 5717	ΣM^*_{pb} = 3707	OK
2	Maximum probable moment	M_u = 906 k-in	M_{pr} = 3325 k-in	0.27
3	Cover plate shear yielding	R_u = 114.5 kips	ϕR_n = 274.5 kips	0.42
	Cover plate thickness	t_{cp} = 0.5 in	t_{req} = 0.354 in	OK
4	Beam net section check	M_{pe} = 3658 k-in	M_{fr} = 3639 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 114.5 kips	ϕR_n = 189.4 kips	0.60
	Bolt slip for wind load	R_u = 19.7 kips	ϕR_n = 70.6 kips	0.28
6	Alignment line location, C1	$C1_{req}$ = 4.4 in	C1 = 2.2 in	OK
7	Weld 1 strength	r_u = 7.1 kip/in	ϕr_n = 7.5 kip/in	0.94
8	Weld 2 strength	r_u = 11 kip/in	ϕr_n = 11.1 kip/in	0.99
9	External continuity plate rupture: Mode 1	P_u = 68.7 kips	ϕP_n = 70.1 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 91.6 kips	ϕP_n = 99.3 kips	0.92
	External continuity plate rupture: Mode 3	P_u = 114.5 kips	ϕP_n = 174.4 kips	0.66
	External continuity plate rupture: Mode 4	R_u = 91.6 kips	ϕR_n = 108 kips	0.85
	External continuity plate rupture: Mode 5	R_u = 114.5 kips	ϕR_n = 141.8 kips	0.81
10	Beam shear	V_u = 46.9 kips	ϕV_n = 159.3 kips	0.29
11	Beam block shear	R_u = 229 kips	ϕR_n = 241.9 kips	0.95
12	Web bolt shear	R_u = 46.9 kips	ϕR_n = 92.1 kips	0.51
	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.15
13	Shear tab weld failure	r_u = 16.1 kips	ϕr_n = 16.7 kips	0.96
15	Top plate for shear yielding	R_u = 114.5 kips	ϕR_n = 240 kips	0.48
	Top plate for shear rupture	R_u = 114.5 kips	ϕR_n = 151.7 kips	0.75
16	Top plate for tensile rupture in extensions	P_u = 45.8 kips	ϕP_n = 48.8 kips	0.94
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -2.6 in	P2 = 1.9 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 3	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 73.1 kips	ϕP_n = 142.2 kips	0.51
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 73.1 kips	ϕP_n = 119 kips	0.61
19	Fuse yielding region dimension	$F2_{max}$ = 1.7 in	F2 = 1.6 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 0.9	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 73.1 kips	ϕR_n = 135 kips	0.54
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 73.1 kips	ϕR_n = 109.7 kips	0.67
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.99

DF105 Limit State Summary

Column: W21X111 - Beam: W24X104



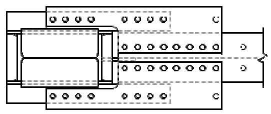


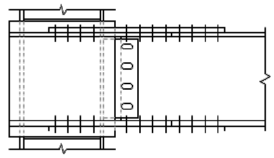


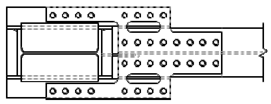
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 8.5	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 7	OK
	Column-beam moment ratio	ΣM^*_{pc} = 32479	ΣM^*_{pb} = 15861	OK
2	Maximum probable moment	M_u = 6733 k-in	M_{pr} = 14450 k-in	0.47
3	Cover plate shear yielding	R_u = 355.5 kips	ϕR_n = 412.5 kips	0.86
	Cover plate thickness	t_{cp} = 0.5 in	t_{req} = 0.471 in	OK
4	Beam net section check	M_{pe} = 15895 k-in	M_{fr} = 17220 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 355.5 kips	ϕR_n = 375.7 kips	0.95
	Bolt slip for wind load	R_u = 75.4 kips	ϕR_n = 138.3 kips	0.55
6	Alignment line location, C1	$C1_{req}$ = 6.1 in	C1 = 6.1 in	OK
7	Weld 1 strength	r_u = 12 kip/in	ϕr_n = 13.2 kip/in	0.91
8	Weld 2 strength	r_u = 17.6 kip/in	ϕr_n = 19.5 kip/in	0.90
9	External continuity plate rupture: Mode 1	P_u = 177.7 kips	ϕP_n = 215.9 kips	0.82
	External continuity plate rupture: Mode 2	P_u = 237 kips	ϕP_n = 241.7 kips	0.98
	External continuity plate rupture: Mode 3	P_u = 296.2 kips	ϕP_n = 367.1 kips	0.81
	External continuity plate rupture: Mode 4	R_u = 237 kips	ϕR_n = 276.1 kips	0.86
	External continuity plate rupture: Mode 5	R_u = 296.2 kips	ϕR_n = 361.5 kips	0.82
10	Beam shear	V_u = 131.3 kips	ϕV_n = 361.5 kips	0.36
11	Beam block shear	R_u = 711 kips	ϕR_n = 713.9 kips	1.00
12	Web bolt shear	R_u = 131.3 kips	ϕR_n = 250.5 kips	0.52
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.42
	Shear tab weld failure	r_u = 9.6 kips	ϕr_n = 11.1 kips	0.86
15	Top plate for shear yielding	R_u = 355.5 kips	ϕR_n = 687.7 kips	0.52
	Top plate for shear rupture	R_u = 355.5 kips	ϕR_n = 374.3 kips	0.95
16	Top plate for tensile rupture in extensions	P_u = 177.7 kips	ϕP_n = 178.2 kips	1.00
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -0.3 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.1	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 286.4 kips	ϕP_n = 546.8 kips	0.52
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 286.4 kips	ϕP_n = 513.3 kips	0.56
19	Fuse yielding region dimension	$F2_{max}$ = 2.2 in	F2 = 2.1 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 1.1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 286.4 kips	ϕR_n = 379.7 kips	0.75
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 286.4 kips	ϕR_n = 296.2 kips	0.97
Misc	Bridge plate tension failure	P_u = 43.5 kips	ϕP_n = 67.5 kips	0.64
	Bridge plate to cover plate weld failure	R_u = 43.5 kips	ϕR_n = 50.1 kips	0.87
	Shear tab to bridge plate weld failure	R_u = 33.4 kips	ϕR_n = 33.4 kips	1.00
	Bridge plate to column flange weld failure	R_u = 33.4 kips	ϕR_n = 50.1 kips	0.67
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		1.00

DF106 Limit State Summary

Column: W21X122 - Beam: W24X131



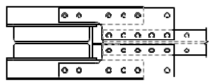


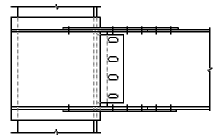


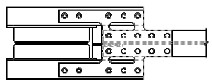
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 6.7	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 6.4	OK
	Column-beam moment ratio	ΣM^*_{pc} = 20938	ΣM^*_{pb} = 19766	OK
2	Maximum probable moment	M_u = 3421 k-in	M_{pr} = 18500 k-in	0.18
3	Cover plate shear yielding	R_u = 446.1 kips	ϕR_n = 623.2 kips	0.72
	Cover plate thickness	t_{cp} = 0.75 in	t_{req} = 0.471 in	OK
4	Beam net section check	M_{pe} = 20350 k-in	M_{fr} = 22012 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 446.1 kips	ϕR_n = 501 kips	0.89
	Bolt slip for wind load	R_u = 33.3 kips	ϕR_n = 184.4 kips	0.18
6	Alignment line location, C1	$C1_{req}$ = 6.1 in	C1 = 6.1 in	OK
7	Weld 1 strength	r_u = 9.6 kip/in	ϕr_n = 11.1 kip/in	0.86
8	Weld 2 strength	r_u = 21.4 kip/in	ϕr_n = 22.3 kip/in	0.96
9	External continuity plate rupture: Mode 1	P_u = 223 kips	ϕP_n = 224.7 kips	0.99
	External continuity plate rupture: Mode 2	P_u = 278.8 kips	ϕP_n = 385.4 kips	0.72
	External continuity plate rupture: Mode 3	P_u = 334.6 kips	ϕP_n = 636 kips	0.53
	External continuity plate rupture: Mode 4	R_u = 278.8 kips	ϕR_n = 382.9 kips	0.73
	External continuity plate rupture: Mode 5	R_u = 334.6 kips	ϕR_n = 477.8 kips	0.70
10	Beam shear	V_u = 116.7 kips	ϕV_n = 444.7 kips	0.26
11	Beam block shear	R_u = 892.1 kips	ϕR_n = 1150.1 kips	0.78
12	Web bolt shear	R_u = 116.7 kips	ϕR_n = 250.5 kips	0.47
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.35
	Shear tab weld failure	r_u = 9.1 kips	ϕr_n = 11.1 kips	0.82
15	Top plate for shear yielding	R_u = 446.1 kips	ϕR_n = 1017.2 kips	0.44
	Top plate for shear rupture	R_u = 446.1 kips	ϕR_n = 553 kips	0.81
16	Top plate for tensile rupture in extensions	P_u = 223 kips	ϕP_n = 228.5 kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -7.7 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 359.2 kips	ϕP_n = 627.2 kips	0.57
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 359.2 kips	ϕP_n = 596 kips	0.60
19	Fuse yielding region dimension	$F2_{max}$ = 2.5 in	F2 = 2.4 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 1.1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 359.2 kips	ϕR_n = 450 kips	0.80
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 359.2 kips	ϕR_n = 365.6 kips	0.98
Misc	Bridge plate tension failure	P_u = 54.2 kips	ϕP_n = 78.8 kips	0.69
	Bridge plate to cover plate weld failure	R_u = 54.2 kips	ϕR_n = 58.5 kips	0.93
	Shear tab to bridge plate weld failure	R_u = 29.2 kips	ϕR_n = 33.4 kips	0.87
	Bridge plate to column flange weld failure	R_u = 29.2 kips	ϕR_n = 62.6 kips	0.47
	Shear plate shear failure	R_u = 165.2 kips	ϕR_n = 296.7 kips	0.56
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.99

DF107 Limit State Summary

Column: W21X73 - Beam: W21X44



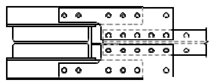


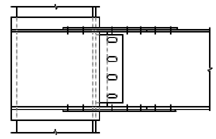


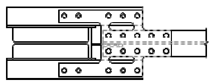
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 7.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5.6	OK
	Column-beam moment ratio	ΣM^*_{pc} = 11221	ΣM^*_{pb} = 5430	OK
2	Maximum probable moment	M_u = 2306 k-in	M_{pr} = 4770 k-in	0.48
3	Cover plate shear yielding	R_u = 141.8 kips	ϕR_n = 348 kips	0.41
	Cover plate thickness	t_{cp} = 0.5 in	t_{req} = 0.439 in	OK
4	Beam net section check	M_{pe} = 5247 k-in	M_{fr} = 5192 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 141.8 kips	ϕR_n = 247.4 kips	0.57
	Bolt slip for wind load	R_u = 38.6 kips	ϕR_n = 92.2 kips	0.42
6	Alignment line location, C1	$C1_{req}$ = 4.4 in	C1 = 2.6 in	OK
7	Weld 1 strength	r_u = 7 kip/in	ϕr_n = 7.3 kip/in	0.96
8	Weld 2 strength	r_u = 10.5 kip/in	ϕr_n = 11.1 kip/in	0.95
9	External continuity plate rupture: Mode 1	P_u = 85 kips	ϕP_n = 86.8 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 113.4 kips	ϕP_n = 133.5 kips	0.85
	External continuity plate rupture: Mode 3	P_u = 141.8 kips	ϕP_n = 247.7 kips	0.57
	External continuity plate rupture: Mode 4	R_u = 113.4 kips	ϕR_n = 136.1 kips	0.83
	External continuity plate rupture: Mode 5	R_u = 141.8 kips	ϕR_n = 178.2 kips	0.80
10	Beam shear	V_u = 62.2 kips	ϕV_n = 217.4 kips	0.29
11	Beam block shear	R_u = 283.5 kips	ϕR_n = 319.2 kips	0.89
12	Web bolt shear	R_u = 62.2 kips	ϕR_n = 163.8 kips	0.38
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.36
	Shear tab weld failure	r_u = 17.1 kips	ϕr_n = 19.5 kips	0.88
15	Top plate for shear yielding	R_u = 141.8 kips	ϕR_n = 285 kips	0.50
	Top plate for shear rupture	R_u = 141.8 kips	ϕR_n = 181.9 kips	0.78
16	Top plate for tensile rupture in extensions	P_u = 56.7 kips	ϕP_n = 59.4 kips	0.95
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -2.8 in	P2 = 2 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 3.8	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 153 kips	0.59
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 122.1 kips	0.74
19	Fuse yielding region dimension	$F2_{max}$ = 2.2 in	F2 = 2.1 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 0.9	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 146.2 kips	0.62
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 113.3 kips	0.79
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.98

DF108 Limit State Summary

Column: W21X73 - Beam: W21X44



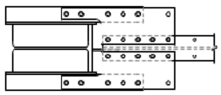


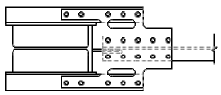


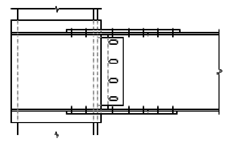
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 7.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5.6	OK
	Column-beam moment ratio	ΣM^*_{pc} = 11211	ΣM^*_{pb} = 5385	OK
2	Maximum probable moment	M_u = 2254 k-in	M_{pr} = 4770 k-in	0.47
3	Cover plate shear yielding	R_u = 141.8 kips	ϕR_n = 348 kips	0.41
	Cover plate thickness	t_{cp} = 0.5 in	t_{req} = 0.439 in	OK
4	Beam net section check	M_{pe} = 5247 k-in	M_{fr} = 5192 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 141.8 kips	ϕR_n = 247.4 kips	0.57
	Bolt slip for wind load	R_u = 20.5 kips	ϕR_n = 92.2 kips	0.22
6	Alignment line location, C1	$C1_{req}$ = 4.4 in	C1 = 2.6 in	OK
7	Weld 1 strength	r_u = 7 kip/in	ϕr_n = 7.3 kip/in	0.96
8	Weld 2 strength	r_u = 10.5 kip/in	ϕr_n = 11.1 kip/in	0.95
9	External continuity plate rupture: Mode 1	P_u = 85 kips	ϕP_n = 86.8 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 113.4 kips	ϕP_n = 133.5 kips	0.85
	External continuity plate rupture: Mode 3	P_u = 141.8 kips	ϕP_n = 247.7 kips	0.57
	External continuity plate rupture: Mode 4	R_u = 113.4 kips	ϕR_n = 136.1 kips	0.83
	External continuity plate rupture: Mode 5	R_u = 141.8 kips	ϕR_n = 178.2 kips	0.80
10	Beam shear	V_u = 58 kips	ϕV_n = 217.4 kips	0.27
11	Beam block shear	R_u = 283.5 kips	ϕR_n = 319.2 kips	0.89
12	Web bolt shear	R_u = 58 kips	ϕR_n = 163.8 kips	0.35
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.35
	Shear tab weld failure	r_u = 16.6 kips	ϕr_n = 19.5 kips	0.85
15	Top plate for shear yielding	R_u = 141.8 kips	ϕR_n = 285 kips	0.50
	Top plate for shear rupture	R_u = 141.8 kips	ϕR_n = 181.9 kips	0.78
16	Top plate for tensile rupture in extensions	P_u = 56.7 kips	ϕP_n = 59.4 kips	0.95
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -2.8 in	P2 = 2 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 3.8	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 153 kips	0.59
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 122.1 kips	0.74
19	Fuse yielding region dimension	$F2_{max}$ = 2.2 in	F2 = 2.1 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 0.9	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 146.2 kips	0.62
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 113.3 kips	0.79
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.98

DF109 Limit State Summary

Column: W21X111 - Beam: W21X50



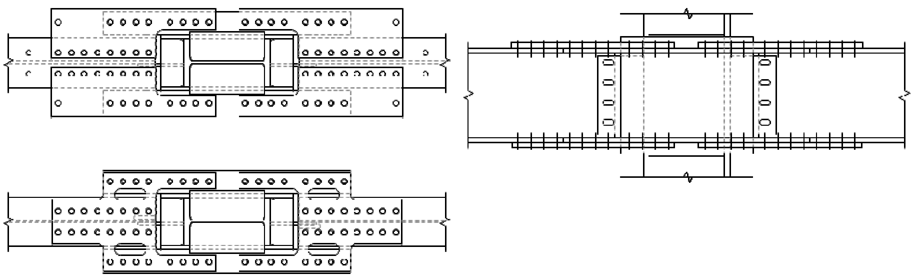




Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 6.1	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 7	OK
	Column-beam moment ratio	ΣM^*_{pc} = 18540	ΣM^*_{pb} = 5943	OK
2	Maximum probable moment	M_u = 2058 k-in	M_{pr} = 5500 k-in	0.37
3	Cover plate shear yielding	R_u = 161.4 kips	ϕR_n = 352.5 kips	0.46
	Cover plate thickness	t_{cp} = 0.5 in	t_{req} = 0.439 in	OK
4	Beam net section check	M_{pe} = 6050 k-in	M_{fr} = 6218 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 161.4 kips	ϕR_n = 189.4 kips	0.85
	Bolt slip for wind load	R_u = 29.5 kips	ϕR_n = 70.6 kips	0.42
6	Alignment line location, C1	$C1_{req}$ = 4.2 in	C1 = 2.4 in	OK
7	Weld 1 strength	r_u = 7.7 kip/in	ϕr_n = 9.1 kip/in	0.85
8	Weld 2 strength	r_u = 11 kip/in	ϕr_n = 11.1 kip/in	0.99
9	External continuity plate rupture: Mode 1	P_u = 96.9 kips	ϕP_n = 99 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 129.1 kips	ϕP_n = 144.1 kips	0.90
	External continuity plate rupture: Mode 3	P_u = 161.4 kips	ϕP_n = 285.5 kips	0.57
	External continuity plate rupture: Mode 4	R_u = 129.1 kips	ϕR_n = 151.8 kips	0.85
	External continuity plate rupture: Mode 5	R_u = 161.4 kips	ϕR_n = 206.2 kips	0.78
10	Beam shear	V_u = 41.2 kips	ϕV_n = 237.1 kips	0.17
11	Beam block shear	R_u = 322.9 kips	ϕR_n = 414.8 kips	0.78
12	Web bolt shear	R_u = 41.2 kips	ϕR_n = 151.5 kips	0.27
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.10
	Shear tab weld failure	r_u = 12.5 kips	ϕr_n = 13.9 kips	0.90
15	Top plate for shear yielding	R_u = 161.4 kips	ϕR_n = 360.9 kips	0.45
	Top plate for shear rupture	R_u = 161.4 kips	ϕR_n = 249.1 kips	0.65
16	Top plate for tensile rupture in extensions	P_u = 64.6 kips	ϕP_n = 64.7 kips	1.00
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -3.1 in	P2 = 2.1 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 3.4	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 102.7 kips	ϕP_n = 296.7 kips	0.35
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 102.7 kips	ϕP_n = 303.5 kips	0.34
19	Fuse yielding region dimension	$F2_{max}$ = 2 in	F2 = 1.9 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 1.1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 102.7 kips	ϕR_n = 161.7 kips	0.63
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 102.7 kips	ϕR_n = 128 kips	0.80
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		1.00

DF201 Limit State Summary

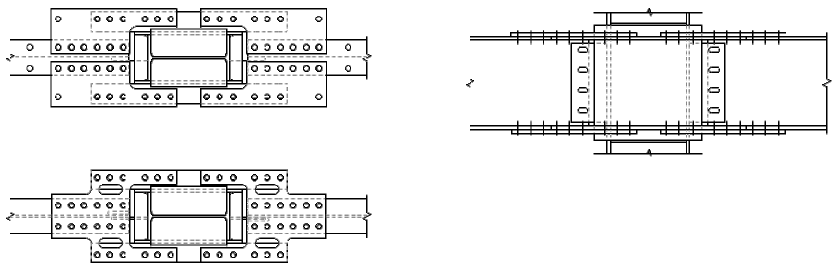
Column: W21X166 - Beam: W24X146



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 5.9	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 4.6	OK
	Column-beam moment ratio	ΣM^*_{pc} = 53951	ΣM^*_{pb} = 45488	OK
2	Maximum probable moment	M_u = 8899 k-in	M_{pr} = 20900 k-in	0.43
3	Cover plate shear yielding	R_u = 902.1 kips	ϕR_n = 1164.4 kips	0.77
	Cover plate thickness	t_{cp} = 1.125 in	t_{req} = 0.47 in	OK
4	Beam net section check	M_{pe} = 22990 k-in	M_{fr} = 24827 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 499.4 kips	ϕR_n = 501 kips	1.00
	Bolt slip for wind load	R_u = 73.2 kips	ϕR_n = 184.4 kips	0.40
6	Alignment line location, C1	$C1_{req}$ = 9.1 in	C1 = 9.1 in	OK
7	Weld 1 strength	r_u = 19.8 kip/in	ϕr_n = 20.7 kip/in	0.96
8	Weld 2 strength	r_u = 30.1 kip/in	ϕr_n = 33.4 kip/in	0.90
9	External continuity plate rupture: Mode 1	P_u = 249.7 kips	ϕP_n = 293.3 kips	0.85
	External continuity plate rupture: Mode 2	P_u = 312.2 kips	ϕP_n = 319.7 kips	0.98
	External continuity plate rupture: Mode 3	P_u = 374.6 kips	ϕP_n = 448.2 kips	0.84
	External continuity plate rupture: Mode 4	R_u = 312.2 kips	ϕR_n = 360.1 kips	0.87
	External continuity plate rupture: Mode 5	R_u = 374.6 kips	ϕR_n = 455 kips	0.82
10	Beam shear	V_u = 163.9 kips	ϕV_n = 481.6 kips	0.34
11	Beam block shear	R_u = 998.9 kips	ϕR_n = 1305.9 kips	0.76
12	Web bolt shear	R_u = 163.9 kips	ϕR_n = 250.5 kips	0.65
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.33
	Shear tab weld failure	r_u = 10.4 kips	ϕr_n = 11.1 kips	0.93
15	Top plate for shear yielding	R_u = 499.4 kips	ϕR_n = 1026.6 kips	0.49
	Top plate for shear rupture	R_u = 499.4 kips	ϕR_n = 562.1 kips	0.89
16	Top plate for tensile rupture in extensions	P_u = 249.7 kips	ϕP_n = 251.4 kips	0.99
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -4.4 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.2	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 402.7 kips	ϕP_n = 655.3 kips	0.61
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 402.7 kips	ϕP_n = 632.5 kips	0.64
19	Fuse yielding region dimension	$F2_{max}$ = 2.8 in	F2 = 2.8 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 402.7 kips	ϕR_n = 492.2 kips	0.82
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 402.7 kips	ϕR_n = 420.5 kips	0.96
Misc	Bridge plate tension failure	P_u = 57 kips	ϕP_n = 140.6 kips	0.41
	Bridge plate to cover plate weld failure	R_u = 57 kips	ϕR_n = 104.4 kips	0.55
	Shear tab to bridge plate weld failure	R_u = 60.6 kips	ϕR_n = 66.8 kips	0.91
	Bridge plate to column flange weld failure	R_u = 60.6 kips	ϕR_n = 86.9 kips	0.70
	Shear plate shear failure	R_u = 247.8 kips	ϕR_n = 370.9 kips	0.67
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		1.00

DF202 Limit State Summary

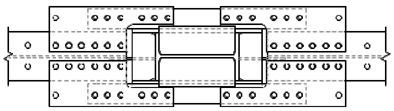
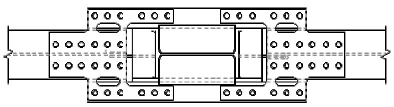
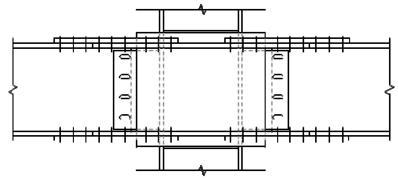
Column: W21X111 - Beam: W24X94



Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 5.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 7	OK
	Column-beam moment ratio	ΣM^*_{pc} = 34530	ΣM^*_{pb} = 28146	OK
2	Maximum probable moment	M_u = 7081 k-in	M_{pr} = 12700 k-in	0.56
3	Cover plate shear yielding	R_u = 564.3 kips	ϕR_n = 630 kips	0.90
	Cover plate thickness	t_{cp} = 0.75 in	t_{req} = 0.47 in	OK
4	Beam net section check	M_{pe} = 13970 k-in	M_{fr} = 14131 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 313.3 kips	ϕR_n = 375.7 kips	0.83
	Bolt slip for wind load	R_u = 76.4 kips	ϕR_n = 138.3 kips	0.55
6	Alignment line location, C1	$C1_{req}$ = 6.4 in	C1 = 3.1 in	OK
7	Weld 1 strength	r_u = 12.2 kip/in	ϕr_n = 13.2 kip/in	0.92
8	Weld 2 strength	r_u = 28.1 kip/in	ϕr_n = 30.6 kip/in	0.92
9	External continuity plate rupture: Mode 1	P_u = 156.7 kips	ϕP_n = 185.9 kips	0.84
	External continuity plate rupture: Mode 2	P_u = 208.9 kips	ϕP_n = 209.3 kips	1.00
	External continuity plate rupture: Mode 3	P_u = 261.1 kips	ϕP_n = 315.3 kips	0.83
	External continuity plate rupture: Mode 4	R_u = 208.9 kips	ϕR_n = 242.1 kips	0.86
	External continuity plate rupture: Mode 5	R_u = 261.1 kips	ϕR_n = 318 kips	0.82
10	Beam shear	V_u = 127.7 kips	ϕV_n = 375.4 kips	0.34
11	Beam block shear	R_u = 626.6 kips	ϕR_n = 673.8 kips	0.93
12	Web bolt shear	R_u = 127.7 kips	ϕR_n = 250.5 kips	0.51
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.40
	Shear tab weld failure	r_u = 9.3 kips	ϕr_n = 11.1 kips	0.83
15	Top plate for shear yielding	R_u = 313.3 kips	ϕR_n = 618.8 kips	0.51
	Top plate for shear rupture	R_u = 313.3 kips	ϕR_n = 340 kips	0.92
16	Top plate for tensile rupture in extensions	P_u = 156.7 kips	ϕP_n = 158.4 kips	0.99
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -0.3 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.4	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 251 kips	ϕP_n = 519.8 kips	0.48
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 251 kips	ϕP_n = 500.2 kips	0.50
19	Fuse yielding region dimension	$F2_{max}$ = 2.2 in	F2 = 2.1 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 1.1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 251 kips	ϕR_n = 337.5 kips	0.74
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 251 kips	ϕR_n = 263.2 kips	0.95
Misc	Bridge plate tension failure	P_u = 41.1 kips	ϕP_n = 101.2 kips	0.41
	Bridge plate to cover plate weld failure	R_u = 41.1 kips	ϕR_n = 75.2 kips	0.55
	Shear tab to bridge plate weld failure	R_u = 33.5 kips	ϕR_n = 36.2 kips	0.93
	Bridge plate to column flange weld failure	R_u = 33.5 kips	ϕR_n = 67.9 kips	0.49
	Shear plate shear failure	R_u = 164.9 kips	ϕR_n = 296.2 kips	0.56
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		1.00

DF203 Limit State Summary

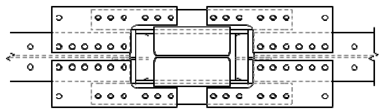
Column: W21X122 - Beam: W24X104

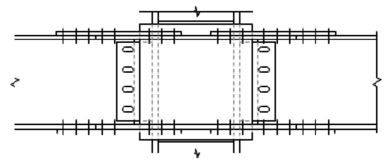




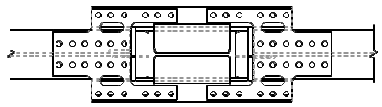
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 8.5	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 6.4	OK
	Column-beam moment ratio	ΣM^*_{pc} = 36861	ΣM^*_{pb} = 31750	OK
2	Maximum probable moment	M_u = 6906 k-in	M_{pr} = 14450 k-in	0.48
3	Cover plate shear yielding	R_u = 641.9 kips	ϕR_n = 884.6 kips	0.73
	Cover plate thickness	t_{cp} = 0.875 in	t_{req} = 0.471 in	OK
4	Beam net section check	M_{pe} = 15895 k-in	M_{fr} = 17220 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 355.5 kips	ϕR_n = 375.7 kips	0.95
	Bolt slip for wind load	R_u = 70.7 kips	ϕR_n = 138.3 kips	0.51
6	Alignment line location, C1	$C1_{req}$ = 9.1 in	C1 = 9.1 in	OK
7	Weld 1 strength	r_u = 13.8 kip/in	ϕr_n = 15.1 kip/in	0.91
8	Weld 2 strength	r_u = 21.7 kip/in	ϕr_n = 22.3 kip/in	0.97
9	External continuity plate rupture: Mode 1	P_u = 177.7 kips	ϕP_n = 215.9 kips	0.82
	External continuity plate rupture: Mode 2	P_u = 237 kips	ϕP_n = 241.7 kips	0.98
	External continuity plate rupture: Mode 3	P_u = 296.2 kips	ϕP_n = 367.1 kips	0.81
	External continuity plate rupture: Mode 4	R_u = 237 kips	ϕR_n = 276.1 kips	0.86
	External continuity plate rupture: Mode 5	R_u = 296.2 kips	ϕR_n = 361.5 kips	0.82
10	Beam shear	V_u = 131.3 kips	ϕV_n = 361.5 kips	0.36
11	Beam block shear	R_u = 711 kips	ϕR_n = 713.9 kips	1.00
12	Web bolt shear	R_u = 131.3 kips	ϕR_n = 250.5 kips	0.52
	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.42
13	Shear tab weld failure	r_u = 9.6 kips	ϕr_n = 11.1 kips	0.86
15	Top plate for shear yielding	R_u = 355.5 kips	ϕR_n = 687.7 kips	0.52
	Top plate for shear rupture	R_u = 355.5 kips	ϕR_n = 374.3 kips	0.95
16	Top plate for tensile rupture in extensions	P_u = 177.7 kips	ϕP_n = 178.2 kips	1.00
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0.2 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.1	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 286.4 kips	ϕP_n = 589.8 kips	0.49
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 286.4 kips	ϕP_n = 569.3 kips	0.50
19	Fuse yielding region dimension	$F2_{max}$ = 2.2 in	F2 = 2.1 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 1.1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 286.4 kips	ϕR_n = 379.7 kips	0.75
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 286.4 kips	ϕR_n = 296.2 kips	0.97
Misc	Bridge plate tension failure	P_u = 38.5 kips	ϕP_n = 112.5 kips	0.34
	Bridge plate to cover plate weld failure	R_u = 38.5 kips	ϕR_n = 83.5 kips	0.46
	Shear tab to bridge plate weld failure	R_u = 49.7 kips	ϕR_n = 50.1 kips	0.99
	Bridge plate to column flange weld failure	R_u = 49.7 kips	ϕR_n = 65.1 kips	0.76
	Shear plate shear failure	R_u = 192.7 kips	ϕR_n = 296.7 kips	0.65
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		1.00

DF204 Limit State Summary

Column: W21X166 - Beam: W24X131



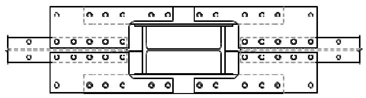


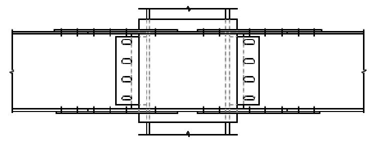


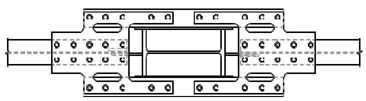
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 6.7	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 4.6	OK
	Column-beam moment ratio	ΣM^*_{pc} = 29925	ΣM^*_{pb} = 30101	NA
2	Maximum probable moment	M_u = 3294 k-in	M_{pr} = 14027 k-in	0.23
3	Cover plate shear yielding	R_u = 634.3 kips	ϕR_n = 767.8 kips	0.83
	Cover plate thickness	t_{cp} = 0.875 in	t_{req} = 0.471 in	OK
4	Beam net section check	M_{pe} = 20350 k-in	M_{fr} = 22012 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 360.6 kips	ϕR_n = 375.7 kips	0.96
	Bolt slip for wind load	R_u = 27 kips	ϕR_n = 138.3 kips	0.19
6	Alignment line location, C1	$C1_{req}$ = 6.5 in	C1 = 6.5 in	OK
7	Weld 1 strength	r_u = 13.8 kip/in	ϕr_n = 14.7 kip/in	0.94
8	Weld 2 strength	r_u = 28.3 kip/in	ϕr_n = 30.6 kip/in	0.93
9	External continuity plate rupture: Mode 1	P_u = 180.3 kips	ϕP_n = 222.8 kips	0.81
	External continuity plate rupture: Mode 2	P_u = 240.4 kips	ϕP_n = 246.6 kips	0.97
	External continuity plate rupture: Mode 3	P_u = 300.5 kips	ϕP_n = 362.3 kips	0.83
	External continuity plate rupture: Mode 4	R_u = 240.4 kips	ϕR_n = 282.9 kips	0.85
	External continuity plate rupture: Mode 5	R_u = 300.5 kips	ϕR_n = 368.3 kips	0.82
10	Beam shear	V_u = 91 kips	ϕV_n = 444.7 kips	0.20
11	Beam block shear	R_u = 721.1 kips	ϕR_n = 918.4 kips	0.79
12	Web bolt shear	R_u = 91 kips	ϕR_n = 250.5 kips	0.36
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.28
	Shear tab weld failure	r_u = 8.4 kips	ϕr_n = 11.1 kips	0.75
15	Top plate for shear yielding	R_u = 360.6 kips	ϕR_n = 696.1 kips	0.52
	Top plate for shear rupture	R_u = 360.6 kips	ϕR_n = 382.5 kips	0.94
16	Top plate for tensile rupture in extensions	P_u = 180.3 kips	ϕP_n = 185.1 kips	0.97
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0.3 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.2	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 273.7 kips	ϕP_n = 589.8 kips	0.46
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 273.7 kips	ϕP_n = 569.3 kips	0.48
19	Fuse yielding region dimension	$F2_{max}$ = 2.2 in	F2 = 2.1 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 1.2	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 273.7 kips	ϕR_n = 367 kips	0.75
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 273.7 kips	ϕR_n = 279.7 kips	0.98
Misc	Bridge plate tension failure	P_u = 45.3 kips	ϕP_n = 120.2 kips	0.38
	Bridge plate to cover plate weld failure	R_u = 45.3 kips	ϕR_n = 89.3 kips	0.51
	Shear tab to bridge plate weld failure	R_u = 24.1 kips	ϕR_n = 37.6 kips	0.64
	Bridge plate to column flange weld failure	R_u = 24.1 kips	ϕR_n = 70.5 kips	0.34
	Shear plate shear failure	R_u = 220.3 kips	ϕR_n = 370.9 kips	0.59
	Fully Restrained (FR) assumption	K_{req} = 5829000 k/in	K_S = 6634070 k/in	OK
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.98

DF205 Limit State Summary

Column: W21X111 - Beam: W21X44



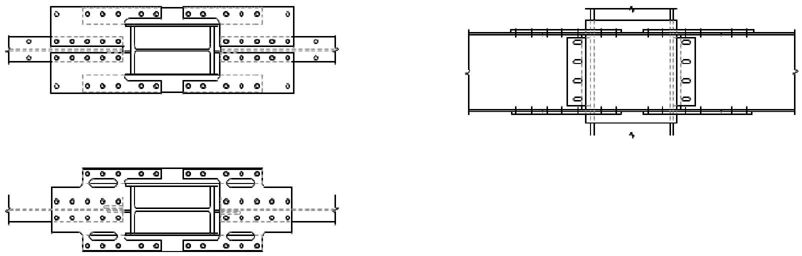




Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 7.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 7	OK
	Column-beam moment ratio	ΣM^*_{pc} = 18499	ΣM^*_{pb} = 11000	OK
2	Maximum probable moment	M_u = 2910 k-in	M_{pr} = 4770 k-in	0.61
3	Cover plate shear yielding	R_u = 254.2 kips	ϕR_n = 382.5 kips	0.66
	Cover plate thickness	t_{cp} = 0.5 in	t_{req} = 0.439 in	OK
4	Beam net section check	M_{pe} = 5247 k-in	M_{fr} = 5192 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 141.8 kips	ϕR_n = 247.4 kips	0.57
	Bolt slip for wind load	R_u = 53.4 kips	ϕR_n = 92.2 kips	0.58
6	Alignment line location, C1	$C1_{req}$ = 5.2 in	C1 = 2.6 in	OK
7	Weld 1 strength	r_u = 13.2 kip/in	ϕr_n = 14.5 kip/in	0.91
8	Weld 2 strength	r_u = 14.3 kip/in	ϕr_n = 16.7 kip/in	0.86
9	External continuity plate rupture: Mode 1	P_u = 85 kips	ϕP_n = 86.8 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 113.4 kips	ϕP_n = 131.3 kips	0.86
	External continuity plate rupture: Mode 3	P_u = 141.8 kips	ϕP_n = 259.5 kips	0.55
	External continuity plate rupture: Mode 4	R_u = 113.4 kips	ϕR_n = 136.1 kips	0.83
	External continuity plate rupture: Mode 5	R_u = 141.8 kips	ϕR_n = 183.9 kips	0.77
10	Beam shear	V_u = 67.9 kips	ϕV_n = 217.4 kips	0.31
11	Beam block shear	R_u = 283.5 kips	ϕR_n = 371.8 kips	0.76
12	Web bolt shear	R_u = 67.9 kips	ϕR_n = 163.8 kips	0.41
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.38
	Shear tab weld failure	r_u = 18.9 kips	ϕr_n = 19.5 kips	0.97
15	Top plate for shear yielding	R_u = 141.8 kips	ϕR_n = 311.2 kips	0.46
	Top plate for shear rupture	R_u = 141.8 kips	ϕR_n = 207.5 kips	0.68
16	Top plate for tensile rupture in extensions	P_u = 56.7 kips	ϕP_n = 59.4 kips	0.95
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -3.4 in	P2 = 2 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 4.2	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 243 kips	0.37
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 239.1 kips	0.38
19	Fuse yielding region dimension	$F2_{max}$ = 2.3 in	F2 = 2.2 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 0.9	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 140.6 kips	0.64
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 106 kips	0.85
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.98

DF206 Limit State Summary

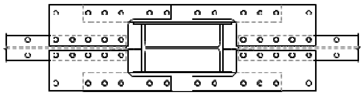
Column: W21X122 - Beam: W21X50

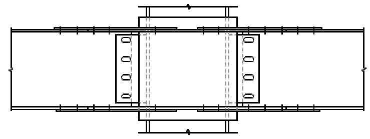


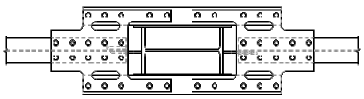
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 6.1	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 6.4	OK
	Column-beam moment ratio	ΣM^*_{pc} = 20583	ΣM^*_{pb} = 11895	OK
2	Maximum probable moment	M_u = 1997 k-in	M_{pr} = 5500 k-in	0.36
3	Cover plate shear yielding	R_u = 289.8 kips	ϕR_n = 355.5 kips	0.82
	Cover plate thickness	t_{cp} = 0.5 in	t_{req} = 0.439 in	OK
4	Beam net section check	M_{pe} = 6050 k-in	M_{fr} = 6218 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 161.4 kips	ϕR_n = 189.4 kips	0.85
	Bolt slip for wind load	R_u = 29.1 kips	ϕR_n = 70.6 kips	0.41
6	Alignment line location, C1	$C1_{req}$ = 3.2 in	C1 = 2.4 in	OK
7	Weld 1 strength	r_u = 15.2 kip/in	ϕr_n = 16.2 kip/in	0.94
8	Weld 2 strength	r_u = 23.7 kip/in	ϕr_n = 25.1 kip/in	0.95
9	External continuity plate rupture: Mode 1	P_u = 96.9 kips	ϕP_n = 99 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 129.1 kips	ϕP_n = 138 kips	0.94
	External continuity plate rupture: Mode 3	P_u = 161.4 kips	ϕP_n = 265.4 kips	0.61
	External continuity plate rupture: Mode 4	R_u = 129.1 kips	ϕR_n = 151.8 kips	0.85
	External continuity plate rupture: Mode 5	R_u = 161.4 kips	ϕR_n = 208 kips	0.78
10	Beam shear	V_u = 41.2 kips	ϕV_n = 237.1 kips	0.17
11	Beam block shear	R_u = 322.9 kips	ϕR_n = 430.5 kips	0.75
12	Web bolt shear	R_u = 41.2 kips	ϕR_n = 151.5 kips	0.27
	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.10
13	Shear tab weld failure	r_u = 11.7 kips	ϕr_n = 13.9 kips	0.84
15	Top plate for shear yielding	R_u = 161.4 kips	ϕR_n = 360.9 kips	0.45
	Top plate for shear rupture	R_u = 161.4 kips	ϕR_n = 249.1 kips	0.65
16	Top plate for tensile rupture in extensions	P_u = 64.6 kips	ϕP_n = 64.7 kips	1.00
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -2.7 in	P2 = 2.1 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 3.4	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 102.7 kips	ϕP_n = 334.7 kips	0.31
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 102.7 kips	ϕP_n = 352.8 kips	0.29
19	Fuse yielding region dimension	$F2_{max}$ = 2 in	F2 = 1.9 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 1.1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 102.7 kips	ϕR_n = 147.7 kips	0.70
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 102.7 kips	ϕR_n = 109.7 kips	0.94
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		1.00

DF207 Limit State Summary

Column: W21X111 - Beam: W21X44







Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 7.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 7	OK
	Column-beam moment ratio	ΣM^{*}_{pc} = 18627	ΣM^{*}_{pb} = 10763	OK
2	Maximum probable moment	M_u = 2388 k-in	M_{pr} = 4770 k-in	0.50
3	Cover plate shear yielding	R_u = 254.2 kips	ϕR_n = 382.5 kips	0.66
	Cover plate thickness	t_{cp} = 0.5 in	t_{req} = 0.439 in	OK
4	Beam net section check	M_{pe} = 5247 k-in	M_{fr} = 5192 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 141.8 kips	ϕR_n = 247.4 kips	0.57
	Bolt slip for wind load	R_u = 28.8 kips	ϕR_n = 92.2 kips	0.31
6	Alignment line location, C1	$C1_{req}$ = 5.2 in	C1 = 2.6 in	OK
7	Weld 1 strength	r_u = 13.2 kip/in	ϕr_n = 14.5 kip/in	0.91
8	Weld 2 strength	r_u = 14.3 kip/in	ϕr_n = 16.7 kip/in	0.86
9	External continuity plate rupture: Mode 1	P_u = 85 kips	ϕP_n = 86.8 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 113.4 kips	ϕP_n = 131.3 kips	0.86
	External continuity plate rupture: Mode 3	P_u = 141.8 kips	ϕP_n = 259.5 kips	0.55
	External continuity plate rupture: Mode 4	R_u = 113.4 kips	ϕR_n = 136.1 kips	0.83
	External continuity plate rupture: Mode 5	R_u = 141.8 kips	ϕR_n = 183.9 kips	0.77
10	Beam shear	V_u = 56.9 kips	ϕV_n = 217.4 kips	0.26
11	Beam block shear	R_u = 283.5 kips	ϕR_n = 371.8 kips	0.76
12	Web bolt shear	R_u = 56.9 kips	ϕR_n = 163.8 kips	0.35
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.35
	Shear tab weld failure	r_u = 17.5 kips	ϕr_n = 19.5 kips	0.90
15	Top plate for shear yielding	R_u = 141.8 kips	ϕR_n = 311.2 kips	0.46
	Top plate for shear rupture	R_u = 141.8 kips	ϕR_n = 207.5 kips	0.68
16	Top plate for tensile rupture in extensions	P_u = 56.7 kips	ϕP_n = 59.4 kips	0.95
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = -3.4 in	P2 = 2 in	OK
18	Fuse plate width-thickness ratio of yeilding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 4.2	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 243 kips	0.37
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 239.1 kips	0.38
19	Fuse yielding region dimension	$F2_{max}$ = 2.3 in	F2 = 2.2 in	OK
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.2	F6 / F2 = 0.9	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 140.6 kips	0.64
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 106 kips	0.85
Misc	Bridge plate tension failure	N/A	N/A	N/A
	Bridge plate to cover plate weld failure	N/A	N/A	N/A
	Shear tab to bridge plate weld failure	N/A	N/A	N/A
	Bridge plate to column flange weld failure	N/A	N/A	N/A
	Shear plate shear failure	N/A	N/A	N/A
	Fully Restrained (FR) assumption	N/A	N/A	N/A
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A
		Controlling Demand-Capacity Ratio		0.98

BEAM LOCAL BUCKLING & LATERAL BRACING REPORT

The purpose of this check is to demonstrate that the beams meet local buckling and bracing requirements requirements specific to DuraFuse Frames, per ER 610. The beam is checked to ensure that it can reach the plastic moment prior to lateral-torsional buckling.

Beam	Col CL to CL Length (ft)	Bar to Bar Length (ft)	λ	λ_p	λ Check	M_u (kip-ft)[b]	Lateral Brace Points Required	Lateral-Torsional Buckling Factor, C_b	Max Unbraced Length (ft)	$\varphi M_{n\text{ - for LTB}}$ (kip-ft)[a]	$M_u/\varphi M_{n\text{ - for LTB}}$ (kip-ft)
W24X146	30.0	24.6	5.92	9.15	OK	1742	0	1.47	32.1	3043	0.57
W24X131	30.0	25.6	6.70	9.15	OK	1542	0	1.66	30.6	2564	0.60
W24X104	30.0	25.8	8.50	9.15	OK	1204	0	1.66	30.8	1897	0.63
W24X94	30.0	25.8	5.18	9.15	OK	1058	0	2.22	25.8	1099	0.96
W24X55	30.0	26.7	6.94	9.15	OK	558	1	1.80	13.5	576	0.97
W21X50	30.0	26.0	6.10	9.15	OK	458	1	1.81	13.5	477	0.96
W21X44	30.0	25.7	7.22	9.15	OK	398	1	1.71	12.5	401	0.99
W18X35	30.0	26.7	7.06	9.15	OK	277	2	1.55	10.0	293	0.94

a. Values reported correspond to equally spaced brace points.
b. M_u equals the plastic moment of the beam (M_p), representing the maximum demand on the beam.

DuraFuse Frames® Calculations

Connection ID DF101

W21X122 Column and W24X146 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	E (ksi)	29000
Shear modulus (AISC 360-22)	G (ksi)	11200

Beams		
Beam material grade	*Gr.*	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1

Columns		
Column material grade	*Gr.*	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1

Plates		
Plate material (AISC 358-22 15.3.3)	*Gr.*	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2

Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	*Gr.*	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks			
Beam size		W24X146	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40	
Beam depth	d_b (in.)	24.7	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	232	
Beam weight	W_b (lb/ft)	146	**OK**
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6	
Beam web width-to-thickness ratio	λ_{web}	33.2	**OK**
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), 0.38(E/Fyb)^0.5	$\lambda_{p,flange}$	9.2	
Beam flange width-to-thickness ratio	λ_{flange}	5.92	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		W21X122	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36	
Column depth	d_c (in.)	21.7	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	122	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	0.96	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	270.9	
Gross area of the column section	A_g (in^2)	35.9	
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.2	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	51.1	
Column web width-to-thickness ratio	λ_{web}	31.3	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	

Column flange width-to-thickness ratio	λ_{flange}	6.45 **OK**

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in^3)	307
Column design axial load	P_u (kips)	270.9
Column area	A_{gc} (in^2)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma R_{yc} Z_c *(F_{yc} - P_u / A_{gc})*(H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	36076
Beam plastic section modulus	Z_b (in^3)	418
Bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900
Clear distance between column faces, $B-d_c$	L_h (in)	338.3
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	43.8
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	167.4
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1816
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	22716
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.59 **OK**

*Not applicable for top stories, see AISC 341 E3.4a(a1))

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

Beam moment demand from load combinations (from EOR)	M_u (k-in)	8400
Beam plastic section modulus	Z_b (in^3)	418
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	20900
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	20900 **OK**
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.7
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	404.6
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	404.6

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Beam area	A_{gb} (in^2)	43.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	96.8
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	501.4
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	91
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.7
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	584.5

Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)

Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75
Cover plate overhange dimension	$C3$ (in)	3
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	27.7
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} d_{cp}$	$R_{n,horiz}$ (kips)	623.3
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	623.3
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	*D/C*	0.80 **OK**

Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)

Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2{*}t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.70

Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} h_{cp}$	$R_{n,vert}$ (kips)	690.8	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	690.8	
Shear yielding demand/capacity ratio, $R_{u,vert}$ / $\phi R_{n,vert}$	D/C	0.85	**OK**

Cover plate thickness check (AISC 341 E3.6e.2)

Beam depth	d_b (in)	24.7	
Beam flange thickness	t_{fb} (in)	1.09	
Column flange thickness	t_{fc} (in)	0.96	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.52	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	19.78	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470	
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.60	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.125	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25	
Beam plastic section modulus	Z_b (in^3)	418	
Beam flange thickness	t_{fb} (in)	1.09	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	1.499	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	70.8	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	347.2	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	22990	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	24827	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.93	**OK**

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.8	
Bolt diameter	d_{bolt} (in)	1.125	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	0.994	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	83.5	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	6.0	
Number of bolts per line (AISC 358 15.2(1))	n_b	9	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	3.843	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	3.2	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	5	**OK**
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4	

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	501.4	
Beam flange thickness	t_{fb} (in)	1.09	
Fuse and top plate thickness	$T2$ (in)	1.125	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	1.1	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	1721.7	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_O$	l_{c1} (in)	1.9	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	1317.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_O/2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1394.9	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	751.5	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	751.5	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	563.6	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.89	**OK**

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	4077
Bolt group demand from wind per line, $M_u/(2(d_b+t_p))$	R_u (kip)	78.9
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	9
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	244.1
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	207.5
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.38 **OK**

Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.7
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$*, 1.0 otherwise*	α	1
Cover plate overhang dimension	*C3* (in)	3
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	6.107
Distance from alignment line to column	*C1* (in)	6.125 **OK**

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per weld, $P_{d,total}/2$	P_d (kips)	96.75
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	91
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.7
Plate thickness (see T2 on drawings)	t_p (in)	1.125
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2{*}t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.7
Gage dimension from the cover plate to the line of bolts on the external continuity plates	*C5* (in)	3.25
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} *(in)*	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.70
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.88
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	6
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.87
Vertical distance from center of rotation to geometric center	e_y (in)	0.54

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d){*}L$	ΣRL (k-in)	18224

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	R*L_R (k-in)
W1RT	18.09	9.00	56.53	162.9
W4TL	18.87	5.32	34.04	100.4
W1LB	18.27	8.75	50.09	159.8
W4BR	17.02	5.23	31.21	89.0
W1RC	10.01	5.64	4.79	56.5
W4TC	15.68	4.09	3.91	64.2
W1LC	11.75	5.79	4.08	68.1
W4BC	14.60	4.05	4.20	59.2
		9.00	56.53	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	9.0

Orientation of Resultant Force	θ_r (degrees)	56.5	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.59	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	9.7	
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c*l_{We1})$	r_{un} (kip/in)	0.0	
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	9.7	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.29	
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75*0.6*70*(1+0.5*\sin^{1.5}(\theta_r))*(p_i(1.9-0.9p_i))^{0.3}*0.707*D_{W1}/16)$	ϕr_n (kips)	11.37	
Cover plate to flange weld tearing check, $r_u/\phi r_n$	D/C	0.86	**OK**

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	404.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5000	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.75	
Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang dimension	*C3* (in)	3	
External continuity plate dimension	*C5* (in)	3.25	
Weld 2 size	D_{W2} (1/16 in)	10	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.63	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc}+t_{cp}+[C3]-l_{hb2}$	$l_{we2,eff}$ (in)	4.09	
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe}+P_d)/(d_c+2[C3])$	r_{uv2} (kip/in)	18.1	
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe}+P_d)[C5]/(l_{we2,eff}(d_c+2[C3]-2l_{hb2}-l_{we2,eff}))$	r_{un2} (kip/in)	17.8	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	25.4	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2\ sides)$	ϕr_n (kip/in)	27.8	
External continuity plate-to-cover plate weld tearing check, $r_u/\phi r_n$	D/C	0.91	**OK**

Failure of external continiuty plate base material check

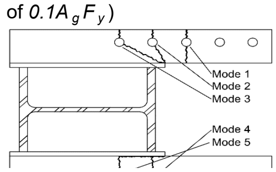
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	404.6	
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	1629.5	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	501.4	
External continuity plate thickness (see step 2)	t_p (in)	1.125	
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c+2[C3])T2$	ϕR_n (kips)	934.9	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c+2*[C3])/2)^2$	ϕM_n (kips)	9711.0	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.70	**OK**

Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

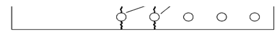
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	404.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.8	
Column fillet dimension	*k* (in)	1.5	
Column fillet dimension	*k1* (in)	1.5	
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	90.5	
Column flange width	b_{fc} (in)	12.4	
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	43.6	
Weld 2 size	D_{W2} (1/16 in)	10	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1678.8	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	809.2	
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl}+R_{nwt})$ or $(0.85R_{nwl}+1.5R_{nwt})$	R_n (kips)	2640.7	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1980.6	
Weld 2 demand/capacity check for cap plate configuration, $R_u/\phi R_n$	D/C	0.25	**OK**

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC *Specification* Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line



Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	193.5	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.8	
Number of bolts ahead of rupture, n_b-n_m	n_{rup}	5.0	
Mode 1 bolt rupture demand, $(V_{fe}+P_d)n_{rup}/n_b$	P_u (kips)	278.6	
External continuity plate thickness (see step 2)	t_p (in)	1.125	
External continuity plate width	*C4* (in)	7.125	



Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, t_p([C4]-($d_{hole,std}$+1/16))	A_n (in^2)	6.539	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	425.0	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	318.8	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.87	**OK**

Mode 2: Rupture through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	334.3	
External continuity plate dimension	C2 (in)	-0.875	
Cover plate overhang dimension	C3 (in)	3	
External continuity plate dimension	C5 (in)	3.25	
External continuity plate dimension, [C2] + [C3]	C9 (in)	2.125	
External continuity plate thickness (see step 2)	t_p (in)	1.125	
External continuity plate width	C4 (in)	7.125	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	Bonus (in)	0.391	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, t_p([C4]-$d_{hole,std}$-1/16) + Bonus	A_n (in^2)	6.930	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	450.4	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	337.8	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.99	**OK**

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	390.0	
Bolt spacing (see schedule)	s (in)	3.375	
External continuity plate dimension	C2 (in)	-0.875	
Cover plate overhang dimension	C3 (in)	3	
External continuity plate dimension	C5 (in)	3.25	
External continuity plate dimension, [C2] + [C3] + s	C7 (in)	5.5	
External continuity plate thickness (see step 2)	t_p (in)	1.125	
External continuity plate width	C4 (in)	7.125	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	Bonus (in)	2.618	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, (t_p)([C4]-$d_{hole,std}$-1/16) + Bonus	A_n (in^2)	9.157	
Nominal tensile rupture capacity, A_nF_{up}	P_n (kips)	595.2	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	446.4	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.87	**OK**

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	334.3	
External continuity plate dimension	C2 (in)	-0.875	
Cover plate overhang dimension	C3 (in)	3	
External continuity plate width	C4 (in)	7.125	
External continuity plate dimension, [C2] + [C3]	C9 (in)	2.125	
External continuity plate thickness (see step 2)	t_p (in)	1.125	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, t_p[C4	A_{gt} (in^2)	8.016	
Gross shear area, t_p[C9]	A_{gv} (in^2)	2.391	
Net tension area, A_{gt} - ((t_p)*($d_{hole,std}$+1/16))	A_{nt} (in^2)	6.539	
Net shear area, A_{gv}	A_{nv} (in^2)	2.391	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	496.8	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	372.6	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.90	**OK**

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	390.0	
Bolt spacing (see schedule)	s (in)	3.375	
External continuity plate dimension	C2 (in)	-0.875	
Cover plate overhang dimension	C3 (in)	3	
External continuity plate width	C4 (in)	7.125	
External continuity plate dimension, [C2] + [C3] + s	C7 (in)	5.5	
External continuity plate thickness (see step 2)	t_p (in)	1.125	

Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, t_p *[C4]*	A_{gt} (in^2)	8.016
Gross shear area, t_p *[C7]*	A_{gv} (in^2)	6.188
Net tension area, $A_{gt}-((t_p)(d_{hole}+1/16))$	A_{nt} (in^2)	6.539
Net shear area, A_{gv}	A_{nv} (in^2)	6.188
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	610.7
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	458.0
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.85 **OK**

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	43.8
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	167.4
Beam depth	d_b (in)	24.7
Beam web thickness	t_{wb} (in)	0.65
Beam web area, $d_b t_{wb}$	A_w (in)	16.055
Beam web width-to-thickness ratio, h / t_w	l_{web}	33.2
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	481.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	*D/C*	0.35 **OK**

*Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2*
C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.8
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	1002.8
Beam flange thickness	t_{fb} (in)	1.09
Beam flange width	b_{fb} (in)	12.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	9
Length of connection on beam	*B4* (in)	28.625
Distance from beam centerline to bolt line	*P2* (in)	2.75
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	8.066
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	62.403
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	6.635
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	38.082
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	1916.5
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1437.4
Demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.70 **OK**

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	167.4
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	0.994
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	3
Provided number of shear tab bolts	n_n	4 **OK**

Shear tab geometry requirements		
Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - 1in	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), T_{beam} - 1in	l_{tab} (in)	19 **OK**

Web bolt limit states check		
Beam web thickness	t_{wb} (in)	0.65
Shear tab thickness	*T3* (in)	0.625

Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	0.6
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\ t_{min}\ F_{up}\ n_n$	R_{nb} (kips)	438.8
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_n -1)$	R_{nt1} (kips)	597.2
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std}/2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	42.7
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	639.8
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75{*}R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.67 **OK**

Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	167.4
Distance from alignment line to column	$C1$ (in)	6.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1025.3

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.625
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam}$ - 1" - $2e_d) / (n_n - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in^2)	11.88
Shear tab net area, $A_{gv} - n_n(w_b + 1/16)t_{tab}$	A_{nv} (in^2)	8.59
Effective net area, $A_e = A_{nv}$	A_e (in^2)	8.59
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up}A_{nv}$	ϕR_{nv} (kips)	251.37
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up}A_e$	ϕR_{nn} (kips)	418.95
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.35 **OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	167.39
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1025.26
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in^2)	11.88
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in^3)	56.41
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp}A_{gv}$	ϕR_{nv} (kips)	356.25
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp}A_{gv}$	ϕR_{nn} (kips)	534.38
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2538.28
Shear tab yielding demand/cacpacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.55 **OK**

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.45
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	167.4
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	7.5
Number of slotted bolt holes in shear tab	n_{vb}	4

Slotted bolt hole spacing	s_{slot} (in)	5.25
Distance from line of bolts to shear tab weld	$C1$ (in)	6.125
Size of weld in 1/16^{ths} of an inch	D_{W3}	4
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1025.3
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.2
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	10.4
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W3}/16(1")(2\ sides)$	ϕr_n (kips)	11.1
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.93 **OK**

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625
Beam depth	d_b (in)	24.7
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.776
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875 **OK**
Provided outside shear tab slot dimension, *[S2] = [S1]*	$S2$ (in)	1.875

Step 15. Check the top plate for shear yielding and shear rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.8
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	501.4

Yielding check		
Top plate thickness (see step 2)	t_p (in)	1.125
Top plate dimension (see schedule)	$P6$ (in)	30.75
Gross shear area of top plate, $(P6)(t_p)$	A_g (in^2)	34.594
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1037.8
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1037.8
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.48 **OK**

Rupture check		
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5
Number of short slotted bolts on top plate (see step 5)	n_b	9
Net shear area of top plate, $A_g - (t_p l_s n_b)$	A_n (in^2)	19.406
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{up}$	R_n (kips)	756.8
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	567.6
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.88 **OK**

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.8
Total number of bolts (see step 5)	n_b	9
Number of bolts in the M region (see step 5)	n_m	4
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	222.8

Yielding check		
Top plate dimension	$P4$ (in)	2
Top plate dimension	$P5$ (in)	3.625
Top plate thickness (see step 2)	t_p (in)	1.125
Gross area, $([P4]+[P5])*(t_p)$	A_g (in^2)	6.328
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	316.4
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	284.8
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.78 **OK**

Rupture check		
Top plate dimension	$P4$ (in)	2

Top plate dimension	$P5$ (in)	3.625	
Top plate thickness (see step 2)	t_p (in)	1.125	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Effective area of rupture, $([P4]+[P5]-d_O-1/16)(t_p)$	A_e (in^2)	4.641	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} A_e$	P_n (kips)	301.6	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	226.2	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.99	**OK**

Step 17

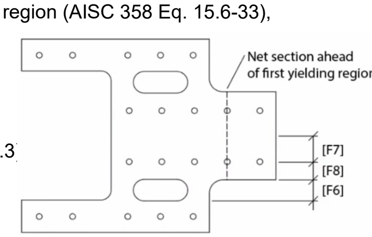
Entering and tightening check			
Beam flange fillet dimension	k_1	1.6	
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.4375	
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3	
Provided P2 dimension	$P2$ (in)	2.75	**OK**

Top plate yielding due to combined flexure and shear check			
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.8	
Top plate thickness (see step 2)	t_p (in)	1.125	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.2	
Inside length of the top plate	$P8$ (in)	30.75	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp} t_p)$	m (in)	16.507	
Dimension, $([P8] - m) / 2$	e (in)	7.122	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-6.8	
Provided P2 dimension	$P2$ (in)	2.75	**OK**

Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC *Specification* Chapter D)

Fuse plate yielding region proportions			
Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4	
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5	
Provided width of yielding region	$F6$ (in)	2.75	**OK**
Top plate thickness (see step 2)	t_p (in)	1.125	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/t_p$	2.4	**OK**

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0	
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0	
Number of bolts in the B region (see step 5)	n_b	9	
Number of bolts in the M region (see step 5)	n_m	4	
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	359.7	
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.7	
Fuse plate thickness (see step 2)	t_p (in)	1.125	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3	d_O (in)	1.4375	
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in^2)	12.263	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	613.1	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	551.8	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.65	**OK**



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Fuse plate dimension	$F7$ (in)	2.75	
Fuse plate dimension	$F8$ (in)	2.7	
Fuse plate thickness (see step 2)	t_p (in)	1.125	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.438	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)t_p$	A_e (in^2)	8.888	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e$ [a]	P_n (kips)	693.2	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	519.9	

Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	*D/C*	0.69 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Step 19. Determine the yielding region depth [F2]

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	404.6
Fuse plate thickness(see step 2)	t_p (in)	1.125
Width of fuse plate cut-out	*F6* (in)	2.75
Coefficient in the strain hardening expression, calibrated from experiments	*A*	1.52
Coefficient in the strain hardening expression, calibrated from experiments	*B*	0.16
Coefficient in the strain hardening expression, calibrated from experiments	*C*	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6]))/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	3.147
Provided yielding region depth	*F2* (in)	3.125 **OK**
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	210.9
Ratio	V_{fe} / V_y	1.9

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, 0.5 $\leq [F6] / [F2] \leq 1.25$	*F6 / F2*	0.88 **OK**

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	n_b	9
Number of bolts in the M region	n_m	4
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	359.7

Yielding check

Fuse plate dimension	*F4* (in)	2
Fuse plate dimension	*F5* (in)	2.25
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in^2)	9.5625
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	478.13
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	430.31
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.84 **OK**

Rupture check

Fuse plate dimension	*F4* (in)	2
Fuse plate dimension	*F5* (in)	2.25
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(t_p)$	A_e (in^2)	6.1875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e$ [a]	R_n (kips)	482.6
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	362.0
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.99 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_d / 2$	P_d (kips)	96.75
External continuity plate dimension	*C5* (in)	3.25
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	66.0

Cover plate overhang	*C3* (in)	3
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	1.000
Bridge plate factored tensile capacity, $0.9*F_{yp}\,t_p\,([C3]\text{-}1")$	ϕP_n (kips)	90.0
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	0.73 **OK**

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	66.0
Weld 5 size	D_{W5} (1/16 in)	8
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16\,([C3]\text{-}1")(2\text{ sides})$	ϕR_n (kips)	66.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.99 **OK**

Shear tab to bridge plate weld failure check

Bay width	*B* (in)	360.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Shear force from gravity	$V_{gravity}$ (kips)	43.8
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Total vertical force at alignment line, $2M_{pr}/(B\text{-}d_c) + V_{gravity}$	V_u (kips)	167.4
Distance from alignment line to column	*C1* (in)	6.125
Weld 7 demand, $V_u\,C1/d_b$	R_u (kips)	41.5
Weld 7 size	D_{W7} (1/16 in)	5
Cover plate overhang	*C3* (in)	3
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(FEXX)(0.707)D_w/16[C3](2\text{ sides})$	ϕR_n (kips)	41.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.99 **OK**

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B\text{-}d_c) + V_{gravity}$	V_u (kips)	167.4
Distance from alignment line to column	*C1* (in)	6.125
Weld 6 demand, $V_u\,[C1]/d_b$	R_u (kips)	41.5
Weld 6 size	D_{W6} (1/16 in)	5
Cover plate overhang	*C3* (in)	3
Column flange width	b_{fc} (in)	12.4
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	6
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	62.6
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.7 **OK**

Shear plate shear failure check (when W1 > 3/4" without shear plates, shear plates are used)

Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Weld 4 size	D_{W4} (1/16 in)	6
Weld 4 effective length, d_c - 2 t_{fc}	l_{eff} (in)	19.78
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16\,(leff)$	R_u (kips)	165.2
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.500
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}\,t_{sp}\,(d_c\text{-}2t_{fc})$	ϕR_n (kips)	296.7
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6 **OK**

HCAI Calculations

DuraFuse Frames® Calculations

Connection ID DF102

W21X73 Column and W24X94 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	E (ksi)	29000
Shear modulus (AISC 360-22)	G (ksi)	11200

Beams		
Beam material grade	*Gr.*	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1

Columns		
Column material grade	*Gr.*	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1

Plates		
Plate material (AISC 358-22 15.3.3)	*Gr.*	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2

Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	*Gr.*	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks			
Beam size		W24X94	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40	
Beam depth	d_b (in.)	24.3	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	232	
Beam weight	W_b (lb/ft)	94	**OK**
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6	
Beam web width-to-thickness ratio	λ_{web}	41.9	**OK**
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), 0.38(E/Fyb)^0.5	$\lambda_{p,flange}$	9.2	
Beam flange width-to-thickness ratio	λ_{flange}	5.18	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		W21X73	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36	
Column depth	d_c (in.)	21.2	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	73	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	0.74	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	173.3	
Gross area of the column section	A_g (in^2)	21.5	
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.2	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	50.9	
Column web width-to-thickness ratio	λ_{web}	41.2	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	

Column flange width-to-thickness ratio	λ_{flange}	5.6	**OK**
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	172	
Column design axial load	P_u (kips)	173.3	
Column area	A_{gc} (in^2)	21.5	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.2	
Beam depth	d_b (in)	24.3	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma R_{yc} Z_c *(F_{yc} - P_u / A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	19918	
Beam plastic section modulus	Z_b (in^3)	254	
Bay width	B (in)	360.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	12700	
Clear distance between column faces, $B-d_c$	L_h (in)	338.8	
Live load factor in seismic load combination	$f1$	0.5	
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	58.1	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	133.1	
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1410	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	14110	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	$Ratio > 1.0$	1.41	**OK**
*Not applicable for top stories, see AISC 341 E3.4a(a1))			

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

Beam moment demand from load combinations (from EOR)	M_u (k-in)	6424	
Beam plastic section modulus	Z_b (in^3)	254	
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	12700	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	12700	**OK**
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1	
Beam depth	d_b (in)	24.3	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	251.0	
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0	
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	251.0	

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	251.0	
Beam area	A_{gb} (in^2)	27.7	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.7	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	62.3	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	313.3	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	50	
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1	
Beam depth	d_b (in)	24.3	
Column depth	d_c (in)	21.2	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	361.9	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75	
Cover plate overhange dimension	$C3$ (in)	3	
Column depth	d_c (in)	21.2	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	27.2	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} d_{cp}$	$R_{n,horiz}$ (kips)	612.0	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	612.0	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.51	**OK**
Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)			
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2{*}t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.30	

Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	681.8	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	681.8	
Shear yielding demand/capacity ratio, $R_{u,vert}$ / $\phi R_{n,vert}$	D/C	0.53	**OK**

Cover plate thickness check (AISC 341 E3.6e.2)

Beam depth	d_b (in)	24.3	
Beam flange thickness	t_{fb} (in)	0.875	
Column flange thickness	t_{fc} (in)	0.74	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.55	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	19.72	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470	
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.60	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.125	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25	
Beam plastic section modulus	Z_b (in^3)	254	
Beam flange thickness	t_{fb} (in)	0.875	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	1.203	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	56.4	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	197.6	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	13970	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	14131	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.99	**OK**

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	251.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.7	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	62.3	
Bolt diameter	d_{bolt} (in)	1.125	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	0.994	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	83.5	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	3.8	
Number of bolts per line (AISC 358 15.2(1))	n_b	6	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	2.681	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	2.5	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	3	**OK**
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	3	

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	313.3	
Beam flange thickness	t_{fb} (in)	0.875	
Fuse and top plate thickness	$T2$ (in)	1	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	0.9	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}t_{min}F_{up}\ n_b$	R_{nb} (kips)	921.4	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_O$	l_{c1} (in)	1.9	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_b - 1)$	R_{nt1} (kips)	661.2	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_O/2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	61.9	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	723.0	
Nominal shear strength of all bolts per line, $F_{nv}A_{bolt}n_b$	R_{nv} (kips)	501.0	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	501.0	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	375.7	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.83	**OK**

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3206
Bolt group demand from wind per line, $M_u/(2(d_b+t_p))$	R_u *(kip)*	63.4
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	6
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	162.7
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	138.3
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.46 **OK**

Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.3
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$*, 1.0 otherwise*	α	1
Cover plate overhang dimension	*C3* (in)	3
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	6.083
Distance from alignment line to column	*C1* (in)	6.125 **OK**

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.7
Drag force per weld, $P_{d,total}/2$	P_d (kips)	62.325
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	50
Column depth	d_c (in)	21.2
Column flange thickness	t_{fc} (in)	0.74
Beam depth	d_b (in)	24.3
Plate thickness (see T2 on drawings)	t_p (in)	1
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2{*}t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.3
Gage dimension from the cover plate to the line of bolts on the external continuity plates	*C5* (in)	2.875
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} *(in)*	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.30
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.75
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	0
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.72
Horizontal distance from center of rotation to geometric center	e_x (in)	0.87
Vertical distance from center of rotation to geometric center	e_y (in)	0.54

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d){*}L$	ΣRL (k-in)	10312

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R{*}L_R$ (k-in)
W1RT	17.79	8.90	56.85	158.3
W4TL	18.69	0.00	34.30	0.0
W1LB	17.96	8.64	50.30	155.2
W4BR	16.84	0.00	31.48	0.0
W1RC	9.76	5.56	4.89	54.3
W4TC	15.48	0.00	3.96	0.0
W1LC	11.50	5.71	4.15	65.7
W4BC	14.40	0.00	4.26	0.0
		8.90	56.85	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	8.9

Orientation of Resultant Force	θ_r (degrees)	56.9	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.59	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	10.5	
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c{*}l_{We1})$	r_{un} (kip/in)	0.0	
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	10.5	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.29	
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75{*}0.6{*}70{*}(1+0.5{*}\sin^{1.5}(\theta_r)){*}(p_i(1.9-0.9p_i))^{0.3}{*}0.707{*}D_{W1}/16)$	ϕr_n (kips)	11.39	
Cover plate to flange weld tearing check, $r_u/\phi r_n$	D/C	0.92	**OK**

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	251.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.6500	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	62.325	
Column depth	d_c (in)	21.2	
Column flange thickness	t_{fc} (in)	0.74	
Cover plate overhang dimension	$C3$ (in)	3	
External continuity plate dimension	$C5$ (in)	2.875	
Weld 2 size	D_{W2} (1/16 in)	6	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.38	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc}+t_{cp}+[C3]-l_{hb2}$	$l_{we2,eff}$ (in)	4.12	
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe}+P_d)/(d_c+2[C3])$	r_{uv2} (kip/in)	11.5	
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe}+P_d)[C5]/(l_{we2,eff}(d_c+2[C3]-2l_{hb2}-l_{we2,eff}))$	r_{un2} (kip/in)	9.8	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	15.1	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2\ \text{sides})$	ϕr_n (kip/in)	16.7	
External continuity plate-to-cover plate weld tearing check, $r_u/\phi r_n$	D/C	0.91	**OK**

Failure of external continiuty plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	251.0	
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	900.8	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	313.3	
External continuity plate thickness (see step 2)	t_p (in)	1.000	
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c+2[C3])T2$	ϕR_n (kips)	816.0	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c+2{*}[C3])/2)^2$	ϕM_n (kips)	8323.2	
Demand/Capacity Ratio, $R_u/\phi R_n+M_u/\phi M_n$	D/C	0.49	**OK**

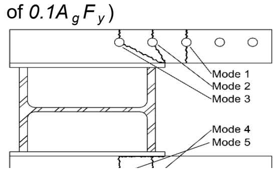
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

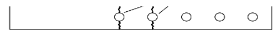
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	251.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.7	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	62.3	
Column fillet dimension	k (in)	1.2	
Column fillet dimension	$k1$ (in)	0.9	
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2{*}l_{hb2})+2{*}(d_c-2k)$	$l_{w2,l}$ (in)	90.3	
Column flange width	b_{fc} (in)	8.3	
Weld 2 length oriented transverse to loading, $2{*}b_{fc}+2{*}(b_{fc}-2k1)$	$l_{w2,t}$ (in)	29.7	
Weld 2 size	D_{W2} (1/16 in)	6	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1006.0	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	330.7	
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl}+R_{nwt})$ or $(0.85R_{nwl}+1.5R_{nwt})$	R_n (kips)	1351.1	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1013.4	
Weld 2 demand/capacity check for cap plate configuration, $R_u/\phi R_n$	D/C	0.31	**OK**

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC *Specification* Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	124.7
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	62.3
Number of bolts ahead of rupture, n_b-n_m	n_{rup}	3.0
Mode 1 bolt rupture demand, $(V_{fe}+P_d)n_{rup}/n_b$	P_u (kips)	156.7
External continuity plate thickness (see step 2)	t_p (in)	1
External continuity plate width	$C4$ (in)	5.125





Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, t_p *([C4]-(*$d_{hole,std}$ *+1/16))*	A_n (in^2)	3.813	
Nominal tensile rupture capacity, $F_{up} A_n$	P_n (kips)	247.8	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	185.9	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.84	**OK**

Mode 2: Rupture through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	208.9	
External continuity plate dimension	*C2 (in)*	-0.5	
Cover plate overhang dimension	*C3* (in)	3	
External continuity plate dimension	*C5* (in)	2.875	
External continuity plate dimension, *[C2]* + *[C3]*	*C9* (in)	2.5	
External continuity plate thickness (see step 2)	t_p (in)	1	
External continuity plate width	*C4* (in)	5.125	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (t_p)/4[C5]$	*Bonus* (in)	0.543	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, t_p *([C4]-*$d_{hole,std}$*-1/16) + Bonus*	A_n (in^2)	4.356	
Nominal tensile rupture capacity, $F_{up} A_n$	P_n (kips)	283.1	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	212.4	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.98	**OK**

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	261.1	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	-0.5	
Cover plate overhang dimension	*C3* (in)	3	
External continuity plate dimension	*C5* (in)	2.875	
External continuity plate dimension, *[C2]* + *[C3]* + *s*	*C7* (in)	5.875	
External continuity plate thickness (see step 2)	t_p (in)	1	
External continuity plate width	*C4* (in)	5.125	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (t_p)/(4[C5])$	*Bonus* (in)	3.001	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, (t_p)*([C4]-*$d_{hole,std}$*-1/16) + Bonus*	A_n (in^2)	6.814	
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	442.9	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	332.2	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.79	**OK**

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	208.9	
External continuity plate dimension	*C2 (in)*	-0.5	
Cover plate overhang dimension	*C3* (in)	3	
External continuity plate width	*C4* (in)	5.125	
External continuity plate dimension, *[C2]* + *[C3]*	*C9* (in)	2.5	
External continuity plate thickness (see step 2)	t_p (in)	1	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, t_p*[C4*	A_{gt} (in^2)	5.125	
Gross shear area, t_p*[C9]*	A_{gv} (in^2)	2.500	
Net tension area, $A_{gt} - ((t_p)*(d_{hole,std} +1/16))$	A_{nt} (in^2)	3.813	
Net shear area, A_{gv}	A_{nv} (in^2)	2.500	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up} A_{nv} + F_{up} A_{nt} < 0.6F_{yp} A_{gv} + F_{up} A_{nt}$	R_n (kips)	322.8	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	242.1	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.86	**OK**

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	261.1	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	-0.5	
Cover plate overhang dimension	*C3* (in)	3	
External continuity plate width	*C4* (in)	5.125	
External continuity plate dimension, *[C2]* + *[C3]* + *s*	*C7* (in)	5.875	
External continuity plate thickness (see step 2)	t_p *(in)*	1	

Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, t_p [C4]	A_{gt} (in^2)	5.125
Gross shear area, t_p [C7]	A_{gv} (in^2)	5.875
Net tension area, $A_{gt}-((t_p)(d_{hole}+1/16))$	A_{nt} (in^2)	3.813
Net shear area, A_{gv}	A_{nv} (in^2)	5.875
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv}+F_{up}A_{nt} < 0.6F_{yp}A_{gv}+F_{up}A_{nt}$	R_n (kips)	424.1
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	318.0
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.82 **OK**

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	58.1
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	12700
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	133.1
Beam depth	d_b (in)	24.3
Beam web thickness	t_{wb} (in)	0.515
Beam web area, $d_b t_{wb}$	A_w (in)	12.5145
Beam web width-to-thickness ratio, h / t_w	l_{web}	41.9
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_wC_v$	ϕV_n (kips)	375.4
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	*D/C*	0.35 **OK**

*Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2*
C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.7
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	62.3
Total force on beam flange, $2(V_{fe}+P_d)$	R_u (kips)	626.6
Beam flange thickness	t_{fb} (in)	0.875
Beam flange width	b_{fb} (in)	9.07
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	6
Length of connection on beam	*B4* (in)	18.5
Distance from beam centerline to bolt line	*P2* (in)	2.75
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	3.124
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	32.375
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	1.975
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	19.742
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv}+F_{ub}A_{nt} < 0.6F_{yb}A_{gv}+F_{ub}A_{nt}$	R_n (kips)	898.3
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	673.8
Demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.93 **OK**

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	133.1
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	0.994
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	2
Provided number of shear tab bolts	n_n	4 **OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - 1in	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), T_{beam} - 1in	l_{tab} (in)	19 **OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.515
Shear tab thickness	*T3* (in)	0.5

Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\ t_{min}\ F_{up}\ n_n$	R_{nb} (kips)	351.0
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_n\ -1)$	R_{nt1} (kips)	477.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d\ - d_{hole,std}\ / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	34.1
Total tearout strength, $R_{nt1}\ + R_{nt2}$	R_{nt} (kips)	511.9
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $min(R_{nb}, R_{nt,}\ R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75{*}R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u\ /\ \phi R_n$	D/C	0.53 **OK**

Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	133.1
Distance from alignment line to column	$C1$ (in)	6.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]{+}d_b/24)$ for roof sloped condition	M_u (k-in)	814.9

Shear tab rupture through net section check		
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam}\ - 1''\ - 2e_d) / (n_n\ - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50
Shear tab net area, $A_{gv} - n_n(w_b + 1/16)t_{tab}$	A_{nv} (in^2)	6.88
Effective net area, $A_e\ = A_{nv}$	A_e (in^2)	6.88
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up}A_{nv}$	ϕR_{nv} (kips)	201.09
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up}A_e$	ϕR_{nn} (kips)	335.16
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.43 **OK**

Shear tab yielding check		
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	133.05
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]{+}d_b/24)$ for roof sloped condition	M_u (k-in)	814.93
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in^3)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp}A_{gv}$	ϕR_{nv} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp}A_{gv}$	ϕR_{nn} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/cacpacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.59 **OK**

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)		
Weld 3 length, equal to l_{tab} except when bridge plates present $d_b\ - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.3
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	133.1
Weld 3 shear demand, $V_u\ /\ l_{tab}$	r_{uv} (k/in)	6.0
Number of slotted bolt holes in shear tab	n_{vb}	4

Slotted bolt hole spacing	s_{slot} (in)	5.25	
Distance from line of bolts to shear tab weld	$C1$ (in)	6.125	
Size of weld in 1/16^{ths} of an inch	D_{W3}	4	
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	814.9	
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.3	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	9.4	
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W3}/16(1")(2\ sides)$	ϕr_n (kips)	11.1	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.85	**OK**

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625	
Beam depth	d_b (in)	24.3	
Length of the shear tab (see step 12)	l_{tab} (in)	19	
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06	
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.764	
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875	**OK**
Provided outside shear tab slot dimension, *[S2] = [S1]*	$S2$ (in)	1.875	

Step 15. Check the top plate for shear yielding and shear rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.7	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	62.3	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	313.3	

Yielding check

Top plate thickness (see step 2)	t_p (in)	1	
Top plate dimension (see schedule)	$P6$ (in)	20.375	
Gross shear area of top plate, $(P6)(t_p)$	A_g (in^2)	20.375	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	611.3	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	611.3	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.51	**OK**

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5	
Number of short slotted bolts on top plate (see step 5)	n_b	6	
Net shear area of top plate, $A_g-(t_p l_s n_b)$	A_n (in^2)	11.375	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{up}$	R_n (kips)	443.6	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	332.7	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.94	**OK**

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.7	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	62.3	
Total number of bolts (see step 5)	n_b	6	
Number of bolts in the M region (see step 5)	n_m	3	
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	156.7	

Yielding check

Top plate dimension	$P4$ (in)	1.875	
Top plate dimension	$P5$ (in)	2.875	
Top plate thickness (see step 2)	t_p (in)	1	
Gross area, $([P4]+[P5])*(t_p)$	A_g (in^2)	4.750	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	237.5	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	213.8	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.73	**OK**

Rupture check

Top plate dimension	$P4$ (in)	1.875

Top plate dimension	$P5$ (in)	2.875	
Top plate thickness (see step 2)	t_p (in)	1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Effective area of rupture, $([P4]+[P5]-d_O-1/16)(t_p)$	A_e (in^2)	3.250	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	211.3	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	158.4	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.99	**OK**

Step 17

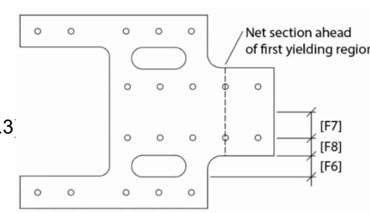
Entering and tightening check			
Beam flange fillet dimension	k_1	1.4	
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.4375	
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	2.875	
Provided P2 dimension	$P2$ (in)	2.75	**OK**

Top plate yielding due to combined flexure and shear check			
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.7	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	62.3	
Top plate thickness (see step 2)	t_p (in)	1	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	7.775	
Inside length of the top plate	$P8$ (in)	20.38	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp}t_p)$	m (in)	11.604	
Dimension, $([P8] - m) / 2$	e (in)	4.385	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-2.3	
Provided P2 dimension	$P2$ (in)	2.75	**OK**

Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC *Specification* Chapter D)

Fuse plate yielding region proportions			
Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4	
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5	
Provided width of yielding region	$F6$ (in)	1.5	**OK**
Top plate thickness (see step 2)	t_p (in)	1	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/t_p$	1.5	**OK**

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0	
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0	
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0	
Number of bolts in the B region (see step 5)	n_b	6	
Number of bolts in the M region (see step 5)	n_m	3	
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	251.0	
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	1.65	
Fuse plate thickness (see step 2)	t_p (in)	1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3	d_O (in)	1.4375	
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in^2)	8.800	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	440.0	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	396.0	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.63	**OK**



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Fuse plate dimension	$F7$ (in)	2.75	
Fuse plate dimension	$F8$ (in)	1.65	
Fuse plate thickness (see step 2)	t_p (in)	1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.438	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)t_p$	A_e (in^2)	5.800	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	P_n (kips)	452.4	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	339.3	

Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	*D/C*	0.74 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Step 19. Determine the yielding region depth [F2]

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	251.0
Fuse plate thickness(see step 2)	t_p (in)	1
Width of fuse plate cut-out	*F6* (in)	1.5
Coefficient in the strain hardening expression, calibrated from experiments	*A*	1.52
Coefficient in the strain hardening expression, calibrated from experiments	*B*	0.16
Coefficient in the strain hardening expression, calibrated from experiments	*C*	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6]))/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.013
Provided yielding region depth	*F2* (in)	2.000 **OK**
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	120.0
Ratio	V_{fe} / V_y	2.1

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, 0.5 *<= [F6] / [F2] <= 1.25*	*F6 / F2*	0.75 **OK**

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	n_b	6
Number of bolts in the M region	n_m	3
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	251.0

Yielding check

Fuse plate dimension	*F4* (in)	1.875
Fuse plate dimension	*F5* (in)	1.875
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in^2)	7.5
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	375.00
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	337.50
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.74 **OK**

Rupture check

Fuse plate dimension	*F4* (in)	1.875
Fuse plate dimension	*F5* (in)	1.875
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(t_p)$	A_e (in^2)	4.5
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e$ [a]	R_n (kips)	351.0
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	263.3
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.95 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	124.7
Drag force per line of bolts, $P_d / 2$	P_d (kips)	62.325
External continuity plate dimension	*C5* (in)	2.875
Column depth	d_c (in)	21.2
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	37.2

Cover plate overhang	*C3* (in)	3
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.625
Bridge plate factored tensile capacity, $0.9*F_{yp}t_p([C3]\text{-}1")$	ϕP_n (kips)	56.3
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	0.66 **OK**

Bridge plate to cover plate weld failure check		
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	37.2
Weld 5 size	D_{W5} (1/16 in)	5
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16([C3]\text{-}1")(2\ sides)$	ϕR_n (kips)	41.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.89 **OK**

Shear tab to bridge plate weld failure check		
Bay width	*B* (in)	360.0
Column depth	d_c (in)	21.2
Beam depth	d_b (in)	24.3
Shear force from gravity	$V_{gravity}$ (kips)	58.1
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	12700
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	133.1
Distance from alignment line to column	*C1* (in)	6.125
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	33.5
Weld 7 size	D_{W7} (1/16 in)	5
Cover plate overhang	*C3* (in)	3
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(FEXX)(0.707)D_w/16[C3](2\ sides)$	ϕR_n (kips)	41.8
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.80 **OK**

Bridge plate to column flange weld failure check		
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	133.1
Distance from alignment line to column	*C1* (in)	6.125
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	33.5
Weld 6 size	D_{W6} (1/16 in)	4
Cover plate overhang	*C3* (in)	3
Column flange width	b_{fc} (in)	8.3
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	6
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	50.1
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.7 **OK**

HCAI Calculations

DuraFuse Frames® Calculations

Connection ID DF201

W21X166 Column and W24X146 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	E (ksi)	29000
Shear modulus (AISC 360-22)	G (ksi)	11200

Beams

Beam material grade	*Gr.*	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1

Columns

Column material grade	*Gr.*	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1

Plates

Plate material (AISC 358-22 15.3.3)	*Gr.*	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2

Bolts

Bolt grade (AISC 358-22 15.5.2(5))	*Gr.*	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks

Beam size		W24X146	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40	
Beam depth	d_b (in.)	24.7	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	232	
Beam weight	W_b (lb/ft)	146	**OK**
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6	
Beam web width-to-thickness ratio	λ_{web}	33.2	**OK**
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), 0.38(E/Fyb)^0.5	$\lambda_{p,flange}$	9.2	
Beam flange width-to-thickness ratio	λ_{flange}	5.92	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks

Column size		W21X166	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36	
Column depth	d_c (in.)	22.5	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	166	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	1.36	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	228.6	
Gross area of the column section	A_g (in^2)	48.8	
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.1	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	53.2	
Column web width-to-thickness ratio	λ_{web}	25	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	

Column flange width-to-thickness ratio	λ_{flange}	4.57	**OK**

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in^3)	432	
Column design axial load	P_u (kips)	228.6	
Column area	A_{gc} (in^2)	48.8	
Story Height	H (in)	216.0	
Column depth	d_c (in)	22.5	
Beam depth	d_b (in)	24.7	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma R_{yc} Z_c *(F_{yc} - P_u / A_{gc})*(H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	53951	
Beam plastic section modulus	Z_b (in^3)	418	
Bay width	B (in)	360.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900	
Clear distance between column faces, $B - d_c$	L_h (in)	337.5	
Live load factor in seismic load combination	$f1$	0.5	
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	40.1	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	163.9	
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1844	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	45488	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	$Ratio > 1.0$	1.19	**OK**

*Not applicable for top stories, see AISC 341 E3.4a(a1))

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

Beam moment demand from load combinations (from EOR)	M_u (k-in)	8899	
Beam plastic section modulus	Z_b (in^3)	418	
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	20900	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	20900	**OK**
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.25	
Beam depth	d_b (in)	24.7	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	402.7	
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0	
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	402.7	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	805.4	

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	805.4	
Beam area	A_{gb} (in^2)	43.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	96.8	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	902.1	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	89	
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.25	
Beam depth	d_b (in)	24.7	
Column depth	d_c (in)	22.5	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	1029.4	

Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)

Cover plate thickness (see T1 in drawings)	t_{cp} (in)	1.125	
Cover plate overhange dimension	$C3$ (in)	6	
Column depth	d_c (in)	22.5	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	34.5	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} d_{cp}$	$R_{n,horiz}$ (kips)	1164.4	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	1164.4	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.77	**OK**

Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)

Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2{*}t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.70	

Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} h_{cp}$	$R_{n,vert}$ (kips)	1036.1	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	1036.1	
Shear yielding demand/capacity ratio, $R_{u,vert}$ / $\phi R_{n,vert}$	D/C	0.99	**OK**

Cover plate thickness check (AISC 341 E3.6e.2)

Beam depth	d_b (in)	24.7	
Beam flange thickness	t_{fb} (in)	1.09	
Column flange thickness	t_{fc} (in)	1.36	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.52	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	19.78	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470	
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	2.39	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.125	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25	
Beam plastic section modulus	Z_b (in^3)	418	
Beam flange thickness	t_{fb} (in)	1.09	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	1.499	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	70.8	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	347.2	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	22990	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	24827	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.93	**OK**

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	402.7	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.8	
Bolt diameter	d_{bolt} (in)	1.125	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	0.994	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	83.5	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	6.0	
Number of bolts per line (AISC 358 15.2(1))	n_b	8	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.25	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	3.442	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	3.0	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	4	**OK**
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4	

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	499.4	
Beam flange thickness	t_{fb} (in)	1.09	
Fuse and top plate thickness	$T2$ (in)	1.25	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	1.1	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	1530.4	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_O$	l_{c1} (in)	1.9	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	1153.1	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1230.1	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	668.0	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	668.0	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	501.0	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	1.00	**OK**

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3797
Bolt group demand from wind per line, $M_u/(2(d_b+t_p))$	R_u *(kip)*	73.2
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	8
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	217.0
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	184.4
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.40 **OK**

Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.7
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$, *1.0 otherwise*	α	1
Cover plate overhang dimension	*C3* (in)	6
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	9.107
Distance from alignment line to column	*C1* (in)	9.125 **OK**

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	805.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per weld, $P_{d,total}/2$	P_d (kips)	96.75
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	89
Column depth	d_c (in)	22.5
Column flange thickness	t_{fc} (in)	1.36
Beam depth	d_b (in)	24.7
Plate thickness (see T2 on drawings)	t_p (in)	1.25
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	1.125
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.7
Gage dimension from the cover plate to the line of bolts on the external continuity plates	*C5* (in)	3.25
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} *(in)*	0.8125
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.08
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	5.38
Weld 1 size	D_{W1} (1/16 in)	11
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	9
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.87
Vertical distance from center of rotation to geometric center	e_y (in)	0.54

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	30777

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	18.07	16.02	54.94	289.3
W4TL	18.87	7.68	34.04	144.9
W1LB	18.30	15.54	48.51	284.4
W4BR	17.02	7.76	31.21	132.1
W1RC	10.41	10.23	4.58	106.5
W4TC	15.68	6.39	3.91	100.1
W1LC	12.15	10.49	3.92	127.5
W4BC	14.60	6.33	4.20	92.5
		16.02	54.94	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	16.0

Orientation of Resultant Force	θ_r (degrees)	54.9
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.58
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	19.8
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c*l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	19.8
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.30
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75*0.6*70*(1+0.5*\sin^{1.5}(\theta_r))*(p_i(1.9-0.9p_i))^{0.3}*0.707*D_{W1}/16)$	ϕr_n (kips)	20.66
Cover plate to flange weld tearing check, $r_u / \phi r_n$	*D/C*	0.96 **OK**

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	805.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.75
Column depth	d_c (in)	22.5
Column flange thickness	t_{fc} (in)	1.36
Cover plate overhang dimension	*C3* (in)	6
External continuity plate dimension	*C5* (in)	3.25
Weld 2 size	D_{W2} (1/16 in)	12
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.75
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc}+t_{cp}+[C3]-l_{hb2}$	$l_{we2,eff}$ (in)	7.74
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe}+P_d)/(d_c+2[C3])$	r_{uv2} (kip/in)	26.1
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe}+P_d)[C5]/(l_{we2,eff}(d_c+2[C3]-2l_{hb2}-l_{we2,eff}))$	r_{un2} (kip/in)	15.0
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	30.1
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2\ sides)$	ϕr_n (kip/in)	33.4
External continuity plate-to-cover plate weld tearing check, $r_u / \phi r_n$	*D/C*	0.90 **OK**

Failure of external continiuty plate base material check

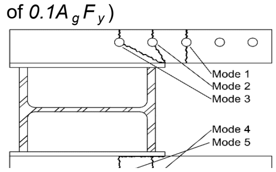
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	805.4
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	2932.0
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	902.1
External continuity plate thickness (see step 2)	t_p (in)	1.250
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c+2[C3])T2$	ϕR_n (kips)	1293.8
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c+2*[C3])/2)^2$	ϕM_n (kips)	16737.9
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.87 **OK**

Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

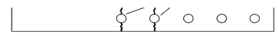
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	805.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.8
Column fillet dimension	*k* (in)	1.9
Column fillet dimension	*k1* (in)	1.6
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	103.6
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	43.35
Weld 2 size	D_{W2} (1/16 in)	12
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	2306.3
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	965.4
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl}+R_{nwt})$ or $(0.85R_{nwl}+1.5R_{nwt})$	R_n (kips)	3408.5
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	2556.4
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	*D/C*	0.35 **OK**

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC *Specification* Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line



Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.8
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	4.0
Mode 1 bolt rupture demand, $(V_{fe}+P_d)n_{rup}/n_b$	P_u (kips)	249.7
External continuity plate thickness (see step 2)	t_p (in)	1.25
External continuity plate width	*C4* (in)	6.125



Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, $t_p([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	6.016	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	391.0	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	293.3	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.85	**OK**

Mode 2: Rupture through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	5	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	312.2	
External continuity plate dimension	*C2 (in)*	-3.625	
Cover plate overhang dimension	*C3* (in)	6	
External continuity plate dimension	*C5* (in)	3.25	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	2.375	
External continuity plate thickness (see step 2)	t_p (in)	1.25	
External continuity plate width	*C4* (in)	6.125	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	*Bonus* (in)	0.542	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	6.558	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	426.3	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	319.7	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.98	**OK**

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	6	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	374.6	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	-3.625	
Cover plate overhang dimension	*C3* (in)	6	
External continuity plate dimension	*C5* (in)	3.25	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5.75	
External continuity plate thickness (see step 2)	t_p (in)	1.25	
External continuity plate width	*C4* (in)	6.125	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	*Bonus* (in)	3.179	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $(t_p)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	9.195	
Nominal tensile rupture capacity, A_nF_{up}	P_n (kips)	597.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	448.2	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.84	**OK**

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	5	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	312.2	
External continuity plate dimension	*C2 (in)*	-3.625	
Cover plate overhang dimension	*C3* (in)	6	
External continuity plate width	*C4* (in)	6.125	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	2.375	
External continuity plate thickness (see step 2)	t_p (in)	1.25	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, $t_p[C4$	A_{gt} (in^2)	7.656	
Gross shear area, $t_p[C9]$	A_{gv} (in^2)	2.969	
Net tension area, $A_{gt} - ((t_p)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	6.016	
Net shear area, A_{gv}	A_{nv} (in^2)	2.969	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	480.1	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	360.1	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.87	**OK**

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	6	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	374.6	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	-3.625	
Cover plate overhang dimension	*C3* (in)	6	
External continuity plate width	*C4* (in)	6.125	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5.75	
External continuity plate thickness (see step 2)	t_p *(in)*	1.25	

Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, t_p *[C4]*	A_{gt} (in^2)	7.656
Gross shear area, t_p *[C7]*	A_{gv} (in^2)	7.188
Net tension area, $A_{gt}-((t_p)(d_{hole}+1/16))$	A_{nt} (in^2)	6.016
Net shear area, A_{gv}	A_{nv} (in^2)	7.188
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	606.6
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	455.0
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.82 **OK**

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Bay width	*B* (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	40.1
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	163.9
Beam depth	d_b (in)	24.7
Beam web thickness	t_{wb} (in)	0.65
Beam web area, $d_b t_{wb}$	A_w (in)	16.055
Beam web width-to-thickness ratio, h / t_w	l_{web}	33.2
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	481.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	*D/C*	0.34 **OK**

*Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2*
C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.8
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	998.9
Beam flange thickness	t_{fb} (in)	1.09
Beam flange width	b_{fb} (in)	12.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	8
Length of connection on beam	*B4* (in)	25.25
Distance from beam centerline to bolt line	*P2* (in)	2.75
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	8.066
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	55.045
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	6.635
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	33.586
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	1741.1
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1305.9
Demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.76 **OK**

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	163.9
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	0.994
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	3
Provided number of shear tab bolts	n_n	4 **OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} *- 1in*	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), T_{beam} *- 1in*	l_{tab} (in)	19 **OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.65
Shear tab thickness	*T3* (in)	0.625

Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	0.6
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\ t_{min}\ F_{up}\ n_n$	R_{nb} (kips)	438.8
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_n\ -1)$	R_{nt1} (kips)	597.2
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d\ - d_{hole,std}\ / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	42.7
Total tearout strength, $R_{nt1}\ + R_{nt2}$	R_{nt} (kips)	639.8
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $min(R_{nb}, R_{nt,}\ R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75{*}R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u\ /\ \phi R_n$	D/C	0.65 **OK**

Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	163.9
Distance from alignment line to column	$C1$ (in)	9.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1495.8

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.625
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam}\ - 1''\ - 2e_d)\ /\ (n_n\ - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in^2)	11.88
Shear tab net area, $A_{gv} - n_n(w_b + 1/16)t_{tab}$	A_{nv} (in^2)	8.59
Effective net area, $A_e\ = A_{nv}$	A_e (in^2)	8.59
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	251.37
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{nn} (kips)	418.95
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{nn})^2 + (V_u/\phi\ R_{nv})^4$	D/C	0.33 **OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	163.92
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1495.81
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in^2)	11.88
Shear tab plastic section modulus, $(l_{tab}/2)^2\ t_{tab}$	Z_{tab} (in^3)	56.41
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	356.25
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{nn} (kips)	534.38
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2538.28
Shear tab yielding demand/cacpacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nn})^2\ + (V_u/\phi R_{nv})^4$	D/C	0.73 **OK**

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b\ - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.2
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	163.9
Weld 3 shear demand, $V_u\ /\ l_{tab}$	r_{uv} (k/in)	7.4
Number of slotted bolt holes in shear tab	n_{vb}	4

Slotted bolt hole spacing	s_{slot} (in)	5.25
Distance from line of bolts to shear tab weld	$C1$ (in)	9.125
Size of weld in 1/16^{ths} of an inch	D_{W3}	4
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1495.8
Normal demand on weld $(3F_b s_{slot} n_{vb}^{\ 2} + 6M_u)/l_{w3}^{\ 2} - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.3
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	10.4
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W3}/16(1")(2\ sides)$	ϕr_n (kips)	11.1
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.93 **OK**

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625
Beam depth	d_b (in)	24.7
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.776
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875 **OK**
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.875

Step 15. Check the top plate for shear yielding and shear rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.8
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	499.4

Yielding check

Top plate thickness (see step 2)	t_p (in)	1.25
Top plate dimension (see schedule)	$P6$ (in)	27.375
Gross shear area of top plate, $(P6)(t_p)$	A_g (in^2)	34.219
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1026.6
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1026.6
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.49 **OK**

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5
Number of short slotted bolts on top plate (see step 5)	n_b	8
Net shear area of top plate, $A_g-(t_p l_s n_b)$	A_n (in^2)	19.219
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	749.5
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	562.1
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.89 **OK**

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.8
Total number of bolts (see step 5)	n_b	8
Number of bolts in the M region (see step 5)	n_m	4
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	249.7

Yielding check

Top plate dimension	$P4$ (in)	2
Top plate dimension	$P5$ (in)	3.625
Top plate thickness (see step 2)	t_p (in)	1.25
Gross area, $([P4]+[P5])*(t_p)$	A_g (in^2)	7.031
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	351.6
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	316.4
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.79 **OK**

Rupture check

Top plate dimension	$P4$ (in)	2

Top plate dimension	$P5$ (in)	3.625	
Top plate thickness (see step 2)	t_p (in)	1.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Effective area of rupture, $([P4]+[P5]-d_O-1/16)(t_p)$	A_e (in^2)	5.156	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	335.2	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	251.4	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.99	**OK**

Step 17

Entering and tightening check

Beam flange fillet dimension	k_1	1.6	
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.4375	
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3	
Provided P2 dimension	$P2$ (in)	2.75	**OK**

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.8	
Top plate thickness (see step 2)	t_p (in)	1.25	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.575	
Inside length of the top plate	$P8$ (in)	27.38	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp}t_p)$	m (in)	14.798	
Dimension, $([P8] - m) / 2$	e (in)	6.288	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-4.4	
Provided P2 dimension	$P2$ (in)	2.75	**OK**

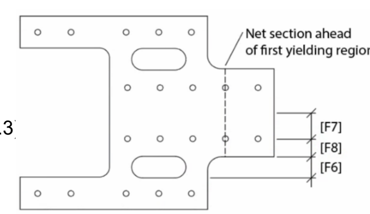
Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC *Specification* Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4	
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5	
Provided width of yielding region	$F6$ (in)	2.75	**OK**
Top plate thickness (see step 2)	t_p (in)	1.25	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/t_p$	2.2	**OK**

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7	
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0	
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0	
Number of bolts in the B region (see step 5)	n_b	8	
Number of bolts in the M region (see step 5)	n_m	4	
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	402.7	
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	3.075	
Fuse plate thickness (see step 2)	t_p (in)	1.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3	d_O (in)	1.4375	
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in^2)	14.563	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	728.1	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	655.3	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.61	**OK**



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.75	
Fuse plate dimension	$F8$ (in)	3.075	
Fuse plate thickness (see step 2)	t_p (in)	1.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.438	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)t_p$	A_e (in^2)	10.813	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	P_n (kips)	843.4	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	632.5	

Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	*D/C*	0.64 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Step 19. Determine the yielding region depth [F2]

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	402.7
Fuse plate thickness(see step 2)	t_p (in)	1.25
Width of fuse plate cut-out	*F6* (in)	2.75
Coefficient in the strain hardening expression, calibrated from experiments	*A*	1.52
Coefficient in the strain hardening expression, calibrated from experiments	*B*	0.16
Coefficient in the strain hardening expression, calibrated from experiments	*C*	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6]))/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.806
Provided yielding region depth	*F2* (in)	2.750 **OK**
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	206.3
Ratio	V_{fe} / V_y	2.0

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, 0.5 <= *[F6] / [F2] <= 1.25*	*F6 / F2*	1.00 **OK**

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	n_b	8
Number of bolts in the M region	n_m	4
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	402.7

Yielding check

Fuse plate dimension	*F4* (in)	2
Fuse plate dimension	*F5* (in)	2.375
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in^2)	10.9375
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	546.88
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	492.19
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.82 **OK**

Rupture check

Fuse plate dimension	*F4* (in)	2
Fuse plate dimension	*F5* (in)	2.375
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(t_p)$	A_e (in^2)	7.1875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e$ [a]	R_n (kips)	560.6
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	420.5
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.96 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.5
Drag force per line of bolts, $P_d / 2$	P_d (kips)	96.75
External continuity plate dimension	*C5* (in)	3.25
Column depth	d_c (in)	22.5
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	57.0

Cover plate overhang	*C3* (in)	6	
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.625	
Bridge plate factored tensile capacity, $0.9 F_{yp} t_p ([C3]-1")$	ϕP_n (kips)	140.6	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	0.41	**OK**

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	57.0	
Weld 5 size	D_{W5} (1/16 in)	5	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16\,([C3]-1")(2\ sides)$	ϕR_n (kips)	104.4	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.55	**OK**

Shear tab to bridge plate weld failure check

Bay width	*B* (in)	360.0	
Column depth	d_c (in)	22.5	
Beam depth	d_b (in)	24.7	
Shear force from gravity	$V_{gravity}$ (kips)	40.1	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	163.9	
Distance from alignment line to column	*C1* (in)	9.125	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	60.6	
Weld 7 size	D_{W7} (1/16 in)	4	
Cover plate overhang	*C3* (in)	6	
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(FEXX)(0.707)D_w/16[C3](2\ sides)$	ϕR_n (kips)	66.8	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.91	**OK**

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	163.9	
Distance from alignment line to column	*C1* (in)	9.125	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	60.6	
Weld 6 size	D_{W6} (1/16 in)	4	
Cover plate overhang	*C3* (in)	6	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	10.4	
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	86.9	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.7	**OK**

Shear plate shear failure check (when W1 > 3/4" without shear plates, shear plates are used)

Column depth	d_c (in)	22.5	
Column flange thickness	t_{fc} (in)	1.36	
Weld 4 size	D_{W4} (1/16 in)	9	
Weld 4 effective length, $d_c - 2t_{fc}$	l_{eff} (in)	19.78	
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16\ (leff)$	R_u (kips)	247.8	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.625	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp} t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	370.9	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.7	**OK**

HCAI Calculations

DuraFuse Frames® Calculations

Connection ID DF150

W21X122 Column and W24X146 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	E (ksi)	29000
Shear modulus (AISC 360-22)	G (ksi)	11200

Beams

Beam material grade	*Gr.*	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1

Columns

Column material grade	*Gr.*	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1

Plates

Plate material (AISC 358-22 15.3.3)	*Gr.*	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2

Bolts

Bolt grade (AISC 358-22 15.5.2(5))	*Gr.*	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks

Beam size		W24X146	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40	
Beam depth	d_b (in.)	24.7	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	232	
Beam weight	W_b (lb/ft)	146	**OK**
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), *3.76(E/F$_{yb}$)$^{0.5}$*	$\lambda_{p,web}$	90.6	
Beam web width-to-thickness ratio	λ_{web}	33.2	**OK**
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), 0.38(E/Fyb)^0.5	$\lambda_{p,flange}$	9.2	
Beam flange width-to-thickness ratio	λ_{flange}	5.92	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks

Column size		W21X122	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36	
Column depth	d_c (in.)	21.7	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	122	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	0.96	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	271.9	
Gross area of the column section	A_g (in^2)	35.9	
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.2	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	51.1	
Column web width-to-thickness ratio	λ_{web}	31.3	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	

Column flange width-to-thickness ratio	λ_{flange}	6.45 **OK**

Column-beam moment ratios (AISC 358 15.4(a))		
Column plastic section modulus	Z_c (in^3)	307
Column design axial load	P_u (kips)	271.9
Column area	A_{gc} (in^2)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma R_{yc} Z_c *(F_{yc} - P_u / A_{gc})*(H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	36056
Beam plastic section modulus	Z_b (in^3)	418
Bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900
Clear distance between column faces, B-d_c	L_h (in)	338.3
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	38.2
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	161.7
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1755
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	22655
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	$Ratio > 1.0$	1.59 **OK**

*Not applicable for top stories, see AISC 341 E3.4a(a1))

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

Beam moment demand from load combinations (from EOR)	M_u (k-in)	8404
Beam plastic section modulus	Z_b (in^3)	418
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	20900
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	20900 **OK**
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.7
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	404.6
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	404.6

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Beam area	A_{gb} (in^2)	43.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	107.5
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	512.1
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	56
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125
Beam depth	d_b (in)	24.7
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	573.5

Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75
Cover plate overhange dimension	$C3$ (in)	3.25
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	28.2
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} d_{cp}$	$R_{n,horiz}$ (kips)	634.5
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	634.5
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.81 **OK**

Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)		
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.70

Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} h_{cp}$	$R_{n,vert}$ (kips)	690.8
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	690.8
Shear yielding demand/capacity ratio, $R_{u,vert}$ / $\phi R_{n,vert}$	D/C	0.83 **OK**

Cover plate thickness check (AISC 341 E3.6e.2)

Beam depth	d_b (in)	24.7
Beam flange thickness	t_{fb} (in)	1.09
Column flange thickness	t_{fc} (in)	0.96
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.52
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	19.78
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.60 **OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.125
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Beam plastic section modulus	Z_b (in^3)	418
Beam flange thickness	t_{fb} (in)	1.09
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	1.499
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	70.8
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	347.2
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	22990
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	24827
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.93 **OK**

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Bolt diameter	d_{bolt} (in)	1.125
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	0.994
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	83.5
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	6.1
Number of bolts per line (AISC 358 15.2(1))	n_b	9 **OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Bolt spacing (see schedule)	s (in)	3.375
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.125
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	3.843
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	3.2
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	5 **OK**
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	512.1
Beam flange thickness	t_{fb} (in)	1.09
Fuse and top plate thickness	$T2$ (in)	1.125
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	1.1
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\ t_{min}\ F_{up}\ n_b$	R_{nb} (kips)	1721.7
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_O$	l_{c1} (in)	1.9
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_b - 1)$	R_{nt1} (kips)	1317.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_O / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	77.0
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1394.9
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	751.5
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	751.5
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	563.6
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.91 **OK**

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3068	
Bolt group demand from wind per line, $M_u/(2(d_b+t_p))$	R_u *(kip)*	59.4	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80	
Mean slip coefficient	μ	0.3	
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13	
Filler factor	h_f	1	
Number of slip planes	n_s	1	
Number of bolts per line	n_b	9	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	244.1	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	207.5	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.29	**OK**

Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06	
Beam depth	d_b (in)	24.7	
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625	
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$*, 1.0 otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	3.25	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	6.357	
Distance from alignment line to column	*C1* (in)	6.375	**OK**

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	107.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	56
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.7
Plate thickness (see T2 on drawings)	t_p (in)	1.125
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.7
Gage dimension from the cover plate to the line of bolts on the external continuity plates	*C5* (in)	3.375
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} *(in)*	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.70
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.88
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	6
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.87
Vertical distance from center of rotation to geometric center	e_y (in)	0.54

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	18224

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	18.09	9.00	56.53	162.9
W4TL	18.87	5.32	34.04	100.4
W1LB	18.27	8.75	50.09	159.8
W4BR	17.02	5.23	31.21	89.0
W1RC	10.01	5.64	4.79	56.5
W4TC	15.68	4.09	3.91	64.2
W1LC	11.75	5.79	4.08	68.1
W4BC	14.60	4.05	4.20	59.2
		9.00	56.53	

Weld strength check according to AISC 360 Chapter J		
Force on Critical Weld Segment	R_{crit} (kips)	9.0

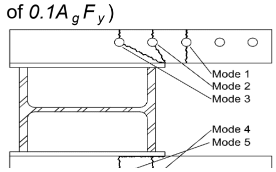
Orientation of Resultant Force	θ_r (degrees)	56.5	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.59	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	9.8	
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c*l_{We1})$	r_{un} (kip/in)	0.0	
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	9.8	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.29	
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75*0.6*70*(1+0.5*\sin^{1.5}(\theta_r))*(p_i(1.9-0.9p_i))^{0.3}*0.707*D_{W1}/16)$	ϕr_n (kips)	11.37	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.87	**OK**

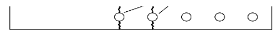
Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	404.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0000	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5	
Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang dimension	*C3* (in)	3.25	
External continuity plate dimension	*C5* (in)	3.375	
Weld 2 size	D_{W2} (1/16 in)	10	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.63	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	4.34	
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe}+P_d) / (d_c+2[C3])$	r_{uv2} (kip/in)	18.2	
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff}(d_c + 2[C3] - 2l_{hb2} - l_{we2,eff}))$	r_{un2} (kip/in)	17.6	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	25.3	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2\ sides)$	ϕr_n (kip/in)	27.8	
External continuity plate-to-cover plate weld tearing check, $r_u / \phi r_n$	D/C	0.91	**OK**
Failure of external continiuty plate base material check			
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	404.6	
External continiuty plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	1728.5	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	512.1	
External continuity plate thickness (see step 2)	t_p (in)	1.125	
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c+2[C3])T2$	ϕR_n (kips)	951.8	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c+2*[C3])/2)^2$	ϕM_n (kips)	10064.8	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.71	**OK**
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)			
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	404.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5	
Column fillet dimension	k (in)	1.5	
Column fillet dimension	$k1$ (in)	1.5	
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3]-2*l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	91.5	
Column flange width	b_{fc} (in)	12.4	
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	43.6	
Weld 2 size	D_{W2} (1/16 in)	10	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1697.4	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	809.2	
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	2656.5	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1992.4	
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.26	**OK**

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC *Specification* Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line		
Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	5.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	284.5
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	*C4* (in)	6.625



Diameter of standard bolt holes (see step 4) 	$d_{hole,std}$ (in)	1.25
Net area of external continuity plate cross section, $t_p([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	5.977
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	388.5
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	291.4
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98 **OK**

Mode 2: Rupture through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	341.4
External continuity plate dimension	$C2$ (in)	2.625
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate dimension	$C5$ (in)	3.375
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	5.875
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	6.625
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	$Bonus$ (in)	2.876
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	8.853
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	575.4
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	431.6
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.79 **OK**

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	398.3
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	2.625
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate dimension	$C5$ (in)	3.375
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	9.25
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	$C4$ (in)	6.625
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	$Bonus$ (in)	7.130
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Area of external continuity plate cross section with bonus, $(t_p)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	13.107
Nominal tensile rupture capacity, A_nF_{up}	P_n (kips)	851.9
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	639.0
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.62 **OK**

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	341.4
External continuity plate dimension	$C2$ (in)	2.625
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate width	$C4$ (in)	6.625
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	5.875
External continuity plate thickness (see step 2)	t_p (in)	1.125
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, $t_p[C4$	A_{gt} (in^2)	7.453
Gross shear area, $t_p[C9]$	A_{gv} (in^2)	6.609
Net tension area, $A_{gt} - ((t_p)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	5.977
Net shear area, A_{gv}	A_{nv} (in^2)	6.609
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	586.8
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	440.1
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.78 **OK**

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	398.3
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	2.625
Cover plate overhang dimension	$C3$ (in)	3.25
External continuity plate width	$C4$ (in)	6.625
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	9.25
External continuity plate thickness (see step 2)	t_p (in)	1.125

Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, t_p *[C4]*	A_{gt} (in^2)	7.453
Gross shear area, t_p *[C7]*	A_{gv} (in^2)	10.406
Net tension area, $A_{gt}-((t_p)(d_{hole}+1/16))$	A_{nt} (in^2)	5.977
Net shear area, A_{gv}	A_{nv} (in^2)	10.406
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	700.7
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	525.5
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.76 **OK**

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Bay width	*B* (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	38.2
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	161.7
Beam depth	d_b (in)	24.7
Beam web thickness	t_{wb} (in)	0.65
Beam web area, $d_b t_{wb}$	A_w (in)	16.055
Beam web width-to-thickness ratio, h / t_w	l_{web}	33.2
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	481.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	*D/C*	0.34 **OK**

*Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2*
C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	1024.3
Beam flange thickness	t_{fb} (in)	1.09
Beam flange width	b_{fb} (in)	12.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	9
Length of connection on beam	*B4* (in)	28.625
Distance from beam centerline to bolt line	*P2* (in)	3
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	7.521
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	62.403
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	6.090
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	38.082
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	1881.1
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1410.8
Demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.73 **OK**

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	161.7
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	0.994
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	3
Provided number of shear tab bolts	n_n	4 **OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} *- 1in*	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), T_{beam} *- 1in*	l_{tab} (in)	19 **OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.65
Shear tab thickness	*T3* (in)	0.5

Governing material thickness, *min*(t_{wb}, *T3*)	t_{min} (in)	0.5	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\ t_{min}\ F_{up}\ n_n$	R_{nb} (kips)	351.0	
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25	
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500	
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	5.333	
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_n - 1)$	R_{nt1} (kips)	477.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std}/2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	34.1	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	511.9	
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0	
Controlling strength, *min*(R_{nb}, $R_{nt,}$ R_{nv})	R_n (kips)	334.0	
Factored shear capacity of all bolts, $0.75{*}R_n$	ϕR_n (kips)	250.5	
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.65	**OK**

Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80	
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3	
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13	
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1	
Number of slip planes required to permit the connection to slip	n_s	1	
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	40.7	
Provided number of shear tab bolts (see step 12)	n_n	4	
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.7	
Distance from alignment line to column	*C1* (in)	6.375	
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1031.1	

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Slotted bolt hole width (standard slot width)	w_b (in)	1.25	
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5	
Shear tab length (see step 12)	l_{tab} (in)	19	
Beam T dimension limit	T_{beam} (in)	20	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5	
Slotted bolt hole spacing, $(T_{beam} - 1'' - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.25	
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab net area, $A_{gv} - n_n(w_b + 1/16)t_{tab}$	A_{nv} (in^2)	6.88	
Effective net area, $A_e = A_{nv}$	A_e (in^2)	6.88	
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	201.09	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{nn} (kips)	335.16	
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	*D/C*	0.65	**OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.75	
Required axial strength of the shear tab (see above)	P_u (kips)	162.72	
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1031.15	
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in^3)	45.13	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	285.00	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{nn} (kips)	427.50	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63	
Shear tab yielding demand/cacpacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	*D/C*	0.76	**OK**

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.45	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.7	
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	7.2	
Number of slotted bolt holes in shear tab	n_{vb}	4	

Slotted bolt hole spacing	s_{slot} (in)	5.25	
Distance from line of bolts to shear tab weld	$C1$ (in)	6.375	
Size of weld in 1/16^{ths} of an inch	D_{W3}	4	
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	1031.1	
Normal demand on weld $(3F_b s_{slot} n_{vb}{}^2 + 6M_u)/l_{w3}{}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.2	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	10.2	
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W3}/16(1")(2\ sides)$	ϕr_n (kips)	11.1	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.92	**OK**

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625	
Beam depth	d_b (in)	24.7	
Length of the shear tab (see step 12)	l_{tab} (in)	19	
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06	
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.776	
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875	**OK**
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.875	

Step 15. Check the top plate for shear yielding and shear rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	512.1	

Yielding check

Top plate thickness (see step 2)	t_p (in)	1.125	
Top plate dimension (see schedule)	$P6$ (in)	31	
Gross shear area of top plate, $(P6)(t_p)$	A_g (in^2)	34.875	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1046.3	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1046.3	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.49	**OK**

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5	
Number of short slotted bolts on top plate (see step 5)	n_b	9	
Net shear area of top plate, $A_g-(t_p l_s n_b)$	A_n (in^2)	19.688	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	767.8	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	575.9	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.89	**OK**

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5	
Total number of bolts (see step 5)	n_b	9	
Number of bolts in the M region (see step 5)	n_m	4	
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	227.6	

Yielding check

Top plate dimension	$P4$ (in)	2.125	
Top plate dimension	$P5$ (in)	3.625	
Top plate thickness (see step 2)	t_p (in)	1.125	
Gross area, $([P4]+[P5])*(t_p)$	A_g (in^2)	6.469	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	323.4	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	291.1	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.78	**OK**

Rupture check

Top plate dimension	$P4$ (in)	2.125	

Top plate dimension	$P5$ (in)	3.625
Top plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Effective area of rupture, $([P4]+[P5]-d_O-1/16)(t_p)$	A_e (in^2)	4.781
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	310.8
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	233.1
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98 **OK**

Step 17

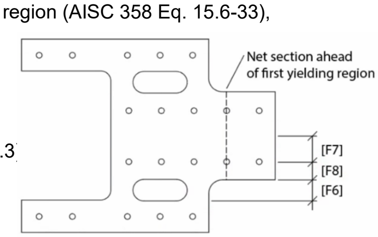
Entering and tightening check		
Beam flange fillet dimension	k_1	1.6
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.4375
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3
Provided P2 dimension	$P2$ (in)	3 **OK**

Top plate yielding due to combined flexure and shear check		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5
Top plate thickness (see step 2)	t_p (in)	1.125
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.325
Inside length of the top plate	$P8$ (in)	31.00
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp}t_p)$	m (in)	16.861
Dimension, $([P8] - m) / 2$	e (in)	7.070
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-6.4
Provided P2 dimension	$P2$ (in)	3 **OK**

Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC *Specification* Chapter D)

Fuse plate yielding region proportions		
Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.75 **OK**
Top plate thickness (see step 2)	t_p (in)	1.125
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.4 **OK**

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	n_b	9
Number of bolts in the M region (see step 5)	n_m	4
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	359.7
Fuse plate dimension, centerline to gage line	$F7$ (in)	3
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.45
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3	d_O (in)	1.4375
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in^2)	12.263
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	613.1
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	551.8
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.65 **OK**



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)		
Fuse plate dimension	$F7$ (in)	3
Fuse plate dimension	$F8$ (in)	2.45
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.438
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)t_p$	A_e (in^2)	8.888
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	P_n (kips)	693.2
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	519.9

Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	*D/C*	0.69 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Step 19. Determine the yielding region depth [F2]

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	404.6
Fuse plate thickness(see step 2)	t_p (in)	1.125
Width of fuse plate cut-out	*F6* (in)	2.75
Coefficient in the strain hardening expression, calibrated from experiments	*A*	1.52
Coefficient in the strain hardening expression, calibrated from experiments	*B*	0.16
Coefficient in the strain hardening expression, calibrated from experiments	*C*	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6]))/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	3.147
Provided yielding region depth	*F2* (in)	3.125 **OK**
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	210.9
Ratio	V_{fe} / V_y	1.9

Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, 0.5 $<= [F6] / [F2] <= 1.25$	*F6 / F2*	0.88 **OK**

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	n_b	9
Number of bolts in the M region	n_m	4
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	359.7

Yielding check		
Fuse plate dimension	*F4* (in)	2.125
Fuse plate dimension	*F5* (in)	2.125
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in^2)	9.5625
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	R_n (kips)	478.13
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	430.31
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.84 **OK**

Rupture check		
Fuse plate dimension	*F4* (in)	2.125
Fuse plate dimension	*F5* (in)	2.125
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(t_p)$	A_e (in^2)	6.1875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	R_n (kips)	482.6
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	362.0
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.99 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	107.5
External continuity plate dimension	*C5* (in)	3.375
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	69.3

Cover plate overhang	$C3$ (in)	3.25	
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	1.000	
Bridge plate factored tensile capacity, $0.9 F_{yp} t_p ([C3]-1")$	ϕP_n (kips)	101.3	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	D/C	0.68	**OK**

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	69.3	
Weld 5 size	D_{W5} (1/16 in)	8	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16\,([C3]-1")(2\ sides)$	ϕR_n (kips)	75.2	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.92	**OK**

Shear tab to bridge plate weld failure check

Bay width	B (in)	360.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.7	
Shear force from gravity	$V_{gravity}$ (kips)	38.2	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	161.7	
Distance from alignment line to column	$C1$ (in)	6.375	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	41.7	
Weld 7 size	D_{W7} (1/16 in)	5	
Cover plate overhang	$C3$ (in)	3.25	
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(FEXX)(0.707)D_w/16[C3](2\ sides)$	ϕR_n (kips)	45.2	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.92	**OK**

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	161.7	
Distance from alignment line to column	$C1$ (in)	6.375	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	41.7	
Weld 6 size	D_{W6} (1/16 in)	5	
Cover plate overhang	$C3$ (in)	3.25	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	6.5	
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	67.9	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6	**OK**

Shear plate shear failure check (when W1 > 3/4" without shear plates, shear plates are used)

Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Weld 4 size	D_{W4} (1/16 in)	6	
Weld 4 effective length, $d_c - 2t_{fc}$	l_{eff} (in)	19.78	
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16\ (leff)$	R_u (kips)	165.2	
Shear plate thickness, $t_{sp} >= 1.1 \cdot W4$ or 3/8"	t_{sp} (in)	0.500	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp} t_{sp} (d_c - 2t_{fc})$	ϕR_n (kips)	296.7	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6	**OK**

Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

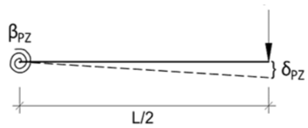
The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

<u>Derivation</u>

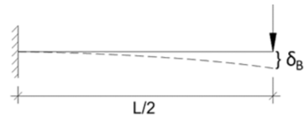
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

<u>Effective Panel Zone Stiffness</u> P <u>Beam Stiffness</u> P

$K_{PZ} = P/\delta_{PZ}$
$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$
$K_{PZ} = \beta_{PZ}/(L/2)^2$



$K_B = P/\delta_B$
$\delta_B = P(L/2)^3/3EI$
$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$
$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$
---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.7	
Beam flange thickness	t_{fb} (in)	1.09	
Column depth	d_c (in)	21.7	
Column flange width	b_{fc} (in)	12.4	
Column web thickness	t_{wc} (in)	0.6	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang	$C3$ (in)	3.3	
Cover plate thickness	t_{cp} (in)	0.75	
Bay width	B (in)	360.0	
Story height	H (in)	216.0	
Panel zone factor, $d_b d_c t_{wc} + 2d_b(d_c + 2[C3])t_{cp}$	V_p (in^3)	1366.4	
Column ratio, $(d_c - t_{fc}) / B$	α	0.058	
Beam ratio, $(d_b - t_{fb}) / H$	β	0.109	
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	22050592	
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	143847	
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	22194439	
Beam moment of inertia	I_{xb} (in^3)	4580	
Required panel zone spring stiffness , *$60EI_{xb}$ / B for one-sided, $120EI_{xb}$ / B for two-sided*	K_{req} (k-in)	22136666	
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	1.0	**OK**

DuraFuse Frames® Calculations

Connection ID DF151

W21X122 Column and W24X146 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	E (ksi)	29000
Shear modulus (AISC 360-22)	G (ksi)	11200

Beams

Beam material grade	*Gr.*	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1

Columns

Column material grade	*Gr.*	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1

Plates

Plate material (AISC 358-22 15.3.3)	*Gr.*	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2

Bolts

Bolt grade (AISC 358-22 15.5.2(5))	*Gr.*	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks

Beam size		W24X146	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40	
Beam depth	d_b (in.)	24.7	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	232	
Beam weight	W_b (lb/ft)	146	**OK**
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6	
Beam web width-to-thickness ratio	λ_{web}	33.2	**OK**
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), 0.38(E/Fyb)^0.5	$\lambda_{p,flange}$	9.2	
Beam flange width-to-thickness ratio	λ_{flange}	5.92	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks

Column size		W21X122	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36	
Column depth	d_c (in.)	21.7	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	122	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	0.96	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	271.9	
Gross area of the column section	A_g (in^2)	35.9	
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.2	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	51.1	
Column web width-to-thickness ratio	λ_{web}	31.3	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	

Column flange width-to-thickness ratio	λ_{flange}	6.45 **OK**

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in^3)	307
Column design axial load	P_u (kips)	271.9
Column area	A_{gc} (in^2)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.7
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma R_{yc} Z_c *(F_{yc} - P_u / A_{gc})*(H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	36056
Beam plastic section modulus	Z_b (in^3)	418
Bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900
Clear distance between column faces, B-d_c	L_h (in)	338.3
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	38.2
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	161.7
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1755
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	22655
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.59 **OK**

*Not applicable for top stories, see AISC 341 E3.4a(a1))

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

Beam moment demand from load combinations (from EOR)	M_u (k-in)	8404
Beam plastic section modulus	Z_b (in^3)	418
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	20900
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	20900 **OK**
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.25
Beam depth	d_b (in)	24.7
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	402.7
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	402.7

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	402.7
Beam area	A_{gb} (in^2)	43.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	107.5
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	510.2
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	56
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.25
Beam depth	d_b (in)	24.7
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	573.8

Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)

Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.875
Cover plate overhange dimension	$C3$ (in)	3
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	27.7
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} d_{cp}$	$R_{n,horiz}$ (kips)	727.1
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	727.1
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.70 **OK**

Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)

Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2^*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.70

Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} h_{cp}$	$R_{n,vert}$ (kips)	805.9	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	805.9	
Shear yielding demand/capacity ratio, $R_{u,vert}$ / $\phi R_{n,vert}$	D/C	0.71	**OK**

Cover plate thickness check (AISC 341 E3.6e.2)

Beam depth	d_b (in)	24.7	
Beam flange thickness	t_{fb} (in)	1.09	
Column flange thickness	t_{fc} (in)	0.96	
Panel zone vertical dimension, d_b - $2t_{fb}$	d_z (in)	22.52	
Panel zone horizontal diemension, d_c - $2t_{fc}$	w_z (in)	19.78	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470	
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.86	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.125	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25	
Beam plastic section modulus	Z_b (in^3)	418	
Beam flange thickness	t_{fb} (in)	1.09	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	1.499	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	70.8	
Beam plastic section modulus of the net section, Z_b - $Z_{bolt\ holes}$	$Z_{b,net}$	347.2	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	22990	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	24827	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.93	**OK**

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	402.7	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5	
Bolt diameter	d_{bolt} (in)	1.125	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	0.994	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	83.5	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	6.1	
Number of bolts per line (AISC 358 15.2(1))	n_b	9	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.25	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	3.442	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	3.0	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	5	**OK**
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4	

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	510.2	
Beam flange thickness	t_{fb} (in)	1.09	
Fuse and top plate thickness	$T2$ (in)	1.25	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	1.1	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	1721.7	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_O$	l_{c1} (in)	1.9	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	1317.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_O/2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1394.9	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	751.5	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	751.5	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	563.6	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.91	**OK**

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3068	
Bolt group demand from wind per line, $M_u/(2(d_b+t_p))$	R_u *(kip)*	59.1	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80	
Mean slip coefficient	μ	0.3	
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13	
Filler factor	h_f	1	
Number of slip planes	n_s	1	
Number of bolts per line	n_b	9	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	244.1	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	207.5	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.28	**OK**

Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06	
Beam depth	d_b (in)	24.7	
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625	
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$*, 1.0 otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	3	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	6.107	
Distance from alignment line to column	*C1* (in)	6.125	**OK**

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	107.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	56
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.7
Plate thickness (see T2 on drawings)	t_p (in)	1.25
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.875
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2^*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	30.7
Gage dimension from the cover plate to the line of bolts on the external continuity plates	*C5* (in)	3.125
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} *(in)*	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.70
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	5.13
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	6
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.87
Vertical distance from center of rotation to geometric center	e_y (in)	0.54

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)^*L$	ΣRL (k-in)	18224

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	R^*L_R (k-in)
W1RT	18.09	9.00	56.53	162.9
W4TL	18.87	5.32	34.04	100.4
W1LB	18.27	8.75	50.09	159.8
W4BR	17.02	5.23	31.21	89.0
W1RC	10.01	5.64	4.79	56.5
W4TC	15.68	4.09	3.91	64.2
W1LC	11.75	5.79	4.08	68.1
W4BC	14.60	4.05	4.20	59.2
		9.00	56.53	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	9.0

Orientation of Resultant Force	θ_r (degrees)	56.5
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.59
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	9.9
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c*l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	9.9
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.29
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75*0.6*70*(1+0.5*\sin^{1.5}(\theta_r))*(p_i(1.9-0.9p_i))^{0.3}*0.707*D_{W1}/16)$	ϕr_n (kips)	11.37
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.87 **OK**

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang dimension	*C3* (in)	3
External continuity plate dimension	*C5* (in)	3.125
Weld 2 size	D_{W2} (1/16 in)	10
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.63
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc}+t_{cp}+[C3]-l_{hb2}$	$l_{we2,eff}$ (in)	4.21
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe}+P_d)/(d_c+2[C3])$	r_{uv2} (kip/in)	18.4
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe}+P_d)[C5]/(l_{we2,eff}(d_c+2[C3]-2l_{hb2}-l_{we2,eff}))$	r_{un2} (kip/in)	17.0
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	25.1
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2\ sides)$	ϕr_n (kip/in)	27.8
External continuity plate-to-cover plate weld tearing check, $r_u / \phi r_n$	D/C	0.90 **OK**

Failure of external continiuty plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	402.7
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	1594.4
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	510.2
External continuity plate thickness (see step 2)	t_p (in)	1.250
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c+2[C3])T2$	ϕR_n (kips)	1038.8
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c+2*[C3])/2)^2$	ϕM_n (kips)	10790.0
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.64 **OK**

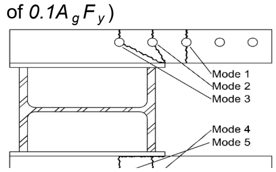
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

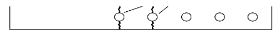
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Column fillet dimension	*k* (in)	1.5
Column fillet dimension	*k1* (in)	1.5
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	90.5
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	43.6
Weld 2 size	D_{W2} (1/16 in)	10
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1678.8
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	809.2
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl}+R_{nwt})$ or $(0.85R_{nwl}+1.5R_{nwt})$	R_n (kips)	2640.7
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1980.6
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.26 **OK**

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC *Specification* Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	5.0
Mode 1 bolt rupture demand, $(V_{fe}+P_d)n_{rup}/n_b$	P_u (kips)	283.4
External continuity plate thickness (see step 2)	t_p (in)	1.25
External continuity plate width	*C4* (in)	6.625





Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, $t_p([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	6.641	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	431.6	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	323.7	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.88	**OK**

Mode 2: Rupture through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	340.1	
External continuity plate dimension	$C2$ (in)	-0.875	
Cover plate overhang dimension	$C3$ (in)	3	
External continuity plate dimension	$C5$ (in)	3.125	
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.125	
External continuity plate thickness (see step 2)	t_p (in)	1.25	
External continuity plate width	$C4$ (in)	6.625	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	$Bonus$ (in)	0.452	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	7.092	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	461.0	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	345.7	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	D/C	0.98	**OK**

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	396.8	
Bolt spacing (see schedule)	s (in)	3.375	
External continuity plate dimension	$C2$ (in)	-0.875	
Cover plate overhang dimension	$C3$ (in)	3	
External continuity plate dimension	$C5$ (in)	3.125	
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.5	
External continuity plate thickness (see step 2)	t_p (in)	1.25	
External continuity plate width	$C4$ (in)	6.625	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	$Bonus$ (in)	3.025	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $(t_p)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	9.666	
Nominal tensile rupture capacity, A_nF_{up}	P_n (kips)	628.3	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	471.2	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.84	**OK**

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	6	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	340.1	
External continuity plate dimension	$C2$ (in)	-0.875	
Cover plate overhang dimension	$C3$ (in)	3	
External continuity plate width	$C4$ (in)	6.625	
External continuity plate dimension, $[C2] + [C3]$	$C9$ (in)	2.125	
External continuity plate thickness (see step 2)	t_p (in)	1.25	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, $t_p[C4$	A_{gt} (in^2)	8.281	
Gross shear area, $t_p[C9]$	A_{gv} (in^2)	2.656	
Net tension area, $A_{gt} - ((t_p)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	6.641	
Net shear area, A_{gv}	A_{nv} (in^2)	2.656	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	511.3	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	383.5	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.89	**OK**

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	7
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	396.8
Bolt spacing (see schedule)	s (in)	3.375
External continuity plate dimension	$C2$ (in)	-0.875
Cover plate overhang dimension	$C3$ (in)	3
External continuity plate width	$C4$ (in)	6.625
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	5.5
External continuity plate thickness (see step 2)	t_p (in)	1.25

Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25
Gross tension area, t_p *[C4]*	A_{gt} (in^2)	8.281
Gross shear area, t_p *[C7]*	A_{gv} (in^2)	6.875
Net tension area, $A_{gt}-((t_p)(d_{hole}+1/16))$	A_{nt} (in^2)	6.641
Net shear area, A_{gv}	A_{nv} (in^2)	6.875
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	637.9
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	478.4
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.83 **OK**

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	38.2
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	161.7
Beam depth	d_b (in)	24.7
Beam web thickness	t_{wb} (in)	0.65
Beam web area, $d_b t_{wb}$	A_w (in)	16.055
Beam web width-to-thickness ratio, h / t_w	l_{web}	33.2
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	481.7
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.34 **OK**

*Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2*
C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	1020.4
Beam flange thickness	t_{fb} (in)	1.09
Beam flange width	b_{fb} (in)	12.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Number of holes along each bolt line (see step 5)	n_b	9
Length of connection on beam	*B4* (in)	28.625
Distance from beam centerline to bolt line	*P2* (in)	3
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	7.521
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	62.403
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	6.090
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	38.082
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	1881.1
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1410.8
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.72 **OK**

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	161.7
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	0.994
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	83.5
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	62.6
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	3
Provided number of shear tab bolts	n_n	4 **OK**

Shear tab geometry requirements		
Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - 1in	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), T_{beam} - 1in	l_{tab} (in)	19 **OK**

Web bolt limit states check		
Beam web thickness	t_{wb} (in)	0.65
Shear tab thickness	*T3* (in)	0.5

Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	0.5	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\ t_{min}\ F_{up}\ n_n$	R_{nb} (kips)	351.0	
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25	
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500	
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	5.333	
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_n\ -1)$	R_{nt1} (kips)	477.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d\ - d_{hole,std}\ / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	34.1	
Total tearout strength, $R_{nt1}\ + R_{nt2}$	R_{nt} (kips)	511.9	
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	334.0	
Factored shear capacity of all bolts, $0.75{*}R_n$	ϕR_n (kips)	250.5	
Shear tab bolt shear demand/capacity ratio, $R_u\ / \phi R_n$	D/C	0.65	**OK**

Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80	
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3	
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13	
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1	
Number of slip planes required to permit the connection to slip	n_s	1	
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	40.7	
Provided number of shear tab bolts (see step 12)	n_n	4	
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.7	
Distance from alignment line to column	$C1$ (in)	6.125	
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	990.7	

Shear tab rupture through net section check			
Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Slotted bolt hole width (standard slot width)	w_b (in)	1.25	
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5	
Shear tab length (see step 12)	l_{tab} (in)	19	
Beam T dimension limit	T_{beam} (in)	20	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5	
Slotted bolt hole spacing, $(T_{beam}\ - 1" - 2e_d) / (n_n\ - 1)$	s_{slot} (in)	5.25	
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab net area, $A_{gv} - n_n(w_b + 1/16)t_{tab}$	A_{nv} (in^2)	6.88	
Effective net area, $A_e\ = A_{nv}$	A_e (in^2)	6.88	
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up}A_{nv}$	ϕR_{nv} (kips)	201.09	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up}A_e$	ϕR_{nn} (kips)	335.16	
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.65	**OK**

Shear tab yielding check			
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.75	
Required axial strength of the shear tab (see above)	P_u (kips)	162.72	
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	990.71	
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in^3)	45.13	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp}A_{gv}$	ϕR_{nv} (kips)	285.00	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp}A_{gv}$	ϕR_{nn} (kips)	427.50	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63	
Shear tab yielding demand/cacpacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.74	**OK**

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)			
Weld 3 length, equal to l_{tab} except when bridge plates present $d_b\ - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.2	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	161.7	
Weld 3 shear demand, $V_u\ / l_{tab}$	r_{uv} (k/in)	7.3	
Number of slotted bolt holes in shear tab	n_{vb}	4	

Slotted bolt hole spacing	s_{slot} (in)	5.25	
Distance from line of bolts to shear tab weld	$C1$ (in)	6.125	
Size of weld in 1/16^{ths} of an inch	D_{W3}	4	
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	990.7	
Normal demand on weld $(3F_b s_{slot} n_{vb}{}^2 + 6M_u)/l_{w3}{}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.3	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	10.3	
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W3}/16(1")(2\ sides)$	ϕr_n (kips)	11.1	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.93	**OK**

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625	
Beam depth	d_b (in)	24.7	
Length of the shear tab (see step 12)	l_{tab} (in)	19	
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06	
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.776	
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875	**OK**
Provided outside shear tab slot dimension, *[S2] = [S1]*	$S2$ (in)	1.875	

Step 15. Check the top plate for shear yielding and shear rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	510.2	

Yielding check

Top plate thickness (see step 2)	t_p (in)	1.25	
Top plate dimension (see schedule)	$P6$ (in)	30.75	
Gross shear area of top plate, $(P6)(t_p)$	A_g (in^2)	38.438	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	1153.1	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	1153.1	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.44	**OK**

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5	
Number of short slotted bolts on top plate (see step 5)	n_b	9	
Net shear area of top plate, $A_g-(t_p l_s n_b)$	A_n (in^2)	21.563	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{up}$	R_n (kips)	840.9	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	630.7	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.81	**OK**

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	107.5	
Total number of bolts (see step 5)	n_b	9	
Number of bolts in the M region (see step 5)	n_m	4	
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	226.8	

Yielding check

Top plate dimension	$P4$ (in)	2	
Top plate dimension	$P5$ (in)	3.25	
Top plate thickness (see step 2)	t_p (in)	1.25	
Gross area, $([P4]+[P5])*(t_p)$	A_g (in^2)	6.563	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	328.1	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	295.3	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.77	**OK**

Rupture check

Top plate dimension	$P4$ (in)	2	

Top plate dimension	$P5$ (in)	3.25	
Top plate thickness (see step 2)	t_p (in)	1.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Effective area of rupture, $([P4]+[P5]-d_O-1/16)(t_p)$	A_e (in^2)	4.688	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	304.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	228.5	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.99	**OK**

Step 17

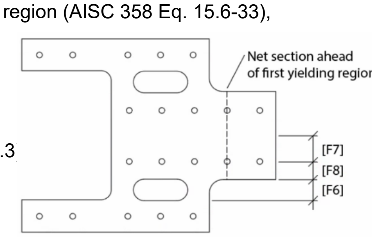
Entering and tightening check			
Beam flange fillet dimension	k_1	1.6	
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.4375	
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3	
Provided P2 dimension	$P2$ (in)	3	**OK**

Top plate yielding due to combined flexure and shear check			
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	107.5	
Top plate thickness (see step 2)	t_p (in)	1.25	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.2	
Inside length of the top plate	$P8$ (in)	30.75	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp}t_p)$	m (in)	15.117	
Dimension, $([P8] - m) / 2$	e (in)	7.817	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-9.6	
Provided P2 dimension	$P2$ (in)	3	**OK**

Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC *Specification* Chapter D)

Fuse plate yielding region proportions			
Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4	
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5	
Provided width of yielding region	$F6$ (in)	2.75	**OK**
Top plate thickness (see step 2)	t_p (in)	1.25	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/t_p$	2.2	**OK**

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7	
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0	
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0	
Number of bolts in the B region (see step 5)	n_b	9	
Number of bolts in the M region (see step 5)	n_m	4	
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	358.0	
Fuse plate dimension, centerline to gage line	$F7$ (in)	3	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.45	
Fuse plate thickness (see step 2)	t_p (in)	1.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3	d_O (in)	1.4375	
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in^2)	13.625	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	681.3	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	613.1	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.58	**OK**



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Fuse plate dimension	$F7$ (in)	3	
Fuse plate dimension	$F8$ (in)	2.45	
Fuse plate thickness (see step 2)	t_p (in)	1.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.438	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)t_p$	A_e (in^2)	9.875	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e{}^a$	P_n (kips)	770.3	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	577.7	

Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	*D/C*	0.62 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Step 19. Determine the yielding region depth [F2]

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	402.7
Fuse plate thickness(see step 2)	t_p (in)	1.25
Width of fuse plate cut-out	*F6* (in)	2.75
Coefficient in the strain hardening expression, calibrated from experiments	*A*	1.52
Coefficient in the strain hardening expression, calibrated from experiments	*B*	0.16
Coefficient in the strain hardening expression, calibrated from experiments	*C*	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6]))/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.806
Provided yielding region depth	*F2* (in)	2.750 **OK**
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	206.3
Ratio	V_{fe} / V_y	2.0

Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, 0.5 $<= [F6] / [F2] <= 1.25$	*F6 / F2*	1.00 **OK**

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	n_b	9
Number of bolts in the M region	n_m	4
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	358.0

Yielding check		
Fuse plate dimension	*F4* (in)	2
Fuse plate dimension	*F5* (in)	2
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in^2)	10
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	R_n (kips)	500.00
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	450.00
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.80 **OK**

Rupture check		
Fuse plate dimension	*F4* (in)	2
Fuse plate dimension	*F5* (in)	2
Fuse plate thickness (see step 2)	t_p (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(t_p)$	A_e (in^2)	6.25
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	R_n (kips)	487.5
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	365.6
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.98 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	107.5
External continuity plate dimension	*C5* (in)	3.125
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	64.5

Cover plate overhang	*C3* (in)	3	
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	1.000	
Bridge plate factored tensile capacity, $0.9*F_{yp}t_p([C3]-1")$	ϕP_n (kips)	90.0	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	0.72	**OK**

Bridge plate to cover plate weld failure check			
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	64.5	
Weld 5 size	D_{W5} (1/16 in)	8	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16\,([C3]-1")(2\ sides)$	ϕR_n (kips)	66.8	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.97	**OK**

Shear tab to bridge plate weld failure check			
Bay width	*B* (in)	360.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.7	
Shear force from gravity	$V_{gravity}$ (kips)	38.2	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	20900	
Total vertical force at alignment line, $2M_{pr}/(B-d_c)+V_{gravity}$	V_u (kips)	161.7	
Distance from alignment line to column	*C1* (in)	6.125	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	40.1	
Weld 7 size	D_{W7} (1/16 in)	5	
Cover plate overhang	*C3* (in)	3	
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(FEXX)(0.707)D_w/16[C3](2\ sides)$	ϕR_n (kips)	41.8	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.96	**OK**

Bridge plate to column flange weld failure check			
Total vertical force at alignment line, $2M_{pr}/(B-d_c)+V_{gravity}$	V_u (kips)	161.7	
Distance from alignment line to column	*C1* (in)	6.125	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	40.1	
Weld 6 size	D_{W6} (1/16 in)	5	
Cover plate overhang	*C3* (in)	3	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc}-2", 2[C3])$	l_{eff} (in)	6	
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	62.6	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**

Shear plate shear failure check (when W1 > 3/4" without shear plates, shear plates are used)

Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Weld 4 size	D_{W4} (1/16 in)	6	
Weld 4 effective length, d_c-2t_{fc}	l_{eff} (in)	19.78	
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16\ (leff)$	R_u (kips)	165.2	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.500	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	296.7	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**

Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

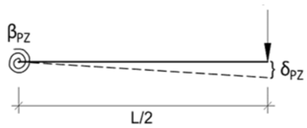
The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

Derivation

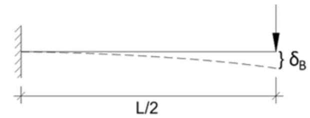
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness P Beam Stiffness P

$K_{PZ} = P/\delta_{PZ}$
$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$
$K_{PZ} = \beta_{PZ}/(L/2)^2$



$K_B = P/\delta_B$
$\delta_B = P(L/2)^3/3EI$
$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$
$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$
---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.7
Beam flange thickness	t_{fb} (in)	1.09
Column depth	d_c (in)	21.7
Column flange width	b_{fc} (in)	12.4
Column web thickness	t_{wc} (in)	0.6
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang	$C3$ (in)	3.0
Cover plate thickness	t_{cp} (in)	0.875
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b(d_c+2[C3])t_{cp}$	V_p (in^3)	1518.9
Column ratio, $(d_c - t_{fc}) / B$	α	0.058
Beam ratio, $(d_b - t_{fb}) / H$	β	0.109
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	24511951
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	143847
Panel zone spring stiffness, $K_{ps}+K_{fs}$	β_{PZ} (k-in)	24655798
Beam moment of inertia	I_{xb} (in^3)	4580
Required panel zone spring stiffness , $60EI_{xb} / B$ for one-sided, $120EI_{xb} / B$ for two-sided	K_{req} (k-in)	22136666
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.9 **OK**

Additional calculations for connections with orthogonal forces from cantilevers or drags

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	402.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	215.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	107.5
Orthogonal drag load (from EOR)	$P_{d,do}$ (kips)	50.0
Moment from orthgonal connection (seismic load combination) (from EOR)	$M_{u,o}$ (k-in)	0
Orthogonal beam size		
Orthogonal beam depth	$d_{b,o}$ (in)	
Orthogonal beam flange thickness	$t_{fb,o}$ (in)	
Flange force associated with $M_{u,o}$, $M_{u,o} / (d_{b,o}-t_{fb,o})$	$P_{d,mo}$ (kips)	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	50.0
Column flange width	b_{fc} (in.)	12.4
Column depth	d_c (in.)	21.7
Column flange thickness	t_{fc} (in.)	0.96
Column web thickness	t_{wc} (in.)	0.60
Plate thickness (see T2 in drawings)	t_p (in)	1.25
External continuity plate gage dimension	$C5$ (in.)	3.1
Plate-to column flange force, $[(\Sigma V_{fe} + P_d) / (1+0.5t_{wc} / t_p)]$	R_{cp} (kips)	411.4
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	206.2
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	50.0

External continuity plate section check (at the coverplate)		
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	1594.4
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	50.0
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	510.2
Plate plastic section modulus, $(d_c - 2t_{fc})^2 t_p/4$	Z (in^3)	122.27
Plate section area, $(d_c - 2t_{fc}) t_p$	A (in^2)	24.73
Moment capacity at section, $0.9F_{yc}Z$	M_c (k-in)	5501.9
Normal capacity at section, $0.9F_{yc}A$	P_c (kips)	1112.6
Shear capacity at section, $0.75(0.6F_{uc})A$	V_c (kips)	723.2
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.54 **OK**

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])\text{-}R_{cp}(b_{fc}/2)$	M_r (k-in)	3209.0	
Normal demand at section, $=P_{d,o}$	P_r (kips)	50.0	
Shear demand at section, $=R_{w1}$	V_r (kips)	206.2	
Corner clip dimension	K_c (in)	1.46	
Plate plastic section modulus, $(d_c\text{-}2t_{fc}\text{-}2K_c)^2 t_p/4$	Z (in^3)	88.8	
Plate section area, $(d_c\text{-}2t_{fc}\text{-}2K_c)t_p$	A (in^2)	21.08	
Moment capacity at section, $0.9F_{yc}Z$	M_c (k-in)	3997.4	
Normal capacity at section, $0.9F_{yc}A$	P_c (kips)	948.4	
Shear capacity at section, $0.75(0.6F_{uc})A$	V_c (kips)	616.4	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.82	**OK**

HCAI Calculations

DuraFuse Frames® Calculations

Connection ID DF152

W21X122 Column and W24X131 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	*E* (ksi)	29000
Shear modulus (AISC 360-22)	*G* (ksi)	11200

Beams		
Beam material grade	*Gr.*	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1

Columns		
Column material grade	*Gr.*	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1

Plates		
Plate material (AISC 358-22 15.3.3)	*Gr.*	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2

Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	*Gr.*	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks			
Beam size		W24X131	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40	
Beam depth	d_b (in.)	24.5	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	232	
Beam weight	W_b (lb/ft)	131	**OK**
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6	
Beam web width-to-thickness ratio	λ_{web}	35.6	**OK**
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), 0.38(E/Fyb)^0.5	$\lambda_{p,flange}$	9.2	
Beam flange width-to-thickness ratio	λ_{flange}	6.7	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		W21X122	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36	
Column depth	d_c (in.)	21.7	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	122	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	0.96	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	53.5	
Gross area of the column section	A_g (in^2)	35.9	
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.0	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	57.2	
Column web width-to-thickness ratio	λ_{web}	31.3	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	

Column flange width-to-thickness ratio	λ_{flange}	6.45	**OK**

Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	307	
Column design axial load	P_u (kips)	53.5	
Column area	A_{gc} (in^2)	35.9	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.5	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma R_{yc} Z_c *(F_{yc} - P_u / A_{gc})*(H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	40633	
Beam plastic section modulus	Z_b (in^3)	370	
Bay width	B (in)	360.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	18500	
Clear distance between column faces, $B-d_c$	L_h (in)	338.3	
Live load factor in seismic load combination	$f1$	0.5	
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	7.0	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	116.4	
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	1263	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	19763	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	$Ratio > 1.0$	2.06	**OK**

*Not applicable for top stories, see AISC 341 E3.4a(a1))

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

Beam moment demand from load combinations (from EOR)	M_u (k-in)	3422	
Beam plastic section modulus	Z_b (in^3)	370	
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	18500	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	18500	**OK**
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125	
Beam depth	d_b (in)	24.5	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	361.0	
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0	
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	361.0	

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	361.0	
Beam area	A_{gb} (in^2)	38.6	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	96.5	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	457.5	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	35	
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1.125	
Beam depth	d_b (in)	24.5	
Column depth	d_c (in)	21.7	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	500.7	

Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.875	
Cover plate overhange dimension	$C3$ (in)	3	
Column depth	d_c (in)	21.7	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	27.7	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} d_{cp}$	$R_{n,horiz}$ (kips)	727.1	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	727.1	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.63	**OK**

Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)			
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	22.25	

Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} h_{cp}$	$R_{n,vert}$ (kips)	584.1	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	584.1	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.86	**OK**

Cover plate thickness check (AISC 341 E3.6e.2)

Beam depth	d_b (in)	24.5	
Beam flange thickness	t_{fb} (in)	0.96	
Column flange thickness	t_{fc} (in)	0.96	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.58	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	19.78	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.471	
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	1.86	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.125	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25	
Beam plastic section modulus	Z_b (in^3)	370	
Beam flange thickness	t_{fb} (in)	0.96	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	1.320	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	62.1	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	307.9	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	20350	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	22012	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.92	**OK**

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	361.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.5	
Bolt diameter	d_{bolt} (in)	1.125	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	0.994	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	83.5	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	5.5	
Number of bolts per line (AISC 358 15.2(1))	n_b	8	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up} R_{tp} t_p)]$	$F2_a$ (in)	3.428	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	3.0	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	4	**OK**
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), $n_b - n_p$	n_m	4	

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	457.5	
Beam flange thickness	t_{fb} (in)	0.96	
Fuse and top plate thickness	$T2$ (in)	1.125	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	1.0	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	1347.8	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_O$	l_{c1} (in)	1.9	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	1015.6	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	67.9	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1083.4	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	668.0	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	668.0	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	501.0	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.91	**OK**

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1451
Bolt group demand from wind per line, $M_u/(2(d_b+t_p))$	R_u (kip)	28.3
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	80
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	8
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	217.0
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	184.4
Wind slip demand/capacity ratio, R_u / ϕR_n	D/C	0.15 **OK**

Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.5
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.625
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$*, 1.0 otherwise*	α	1
Cover plate overhang dimension	$C3$ (in)	3
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	6.095
Distance from alignment line to column	$C1$ (in)	6.125 **OK**

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	96.5
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	35
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.5
Plate thickness (see T2 on drawings)	t_p (in)	1.125
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.875
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	22.25
Gage dimension from the cover plate to the line of bolts on the external continuity plates	$C5$ (in)	3
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.5625
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	21.13
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	5.00
Weld 1 size	D_{W1} (1/16 in)	7
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	7
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.87
Vertical distance from center of rotation to geometric center	e_y (in)	0.54

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$ — ΣRL (k-in) 16547

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	14.77	7.12	47.50	105.2
W4TL	18.79	5.90	34.21	110.8
W1LB	15.29	6.82	39.93	104.3
W4BR	16.94	6.01	31.38	101.7
W1RC	10.01	4.93	4.30	49.3
W4TC	15.58	5.00	3.94	77.8
W1LC	11.75	5.05	3.67	59.3
W4BC	14.50	4.96	4.23	71.9
		7.12	47.50	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	7.1

Orientation of Resultant Force	θ_r (degrees)	47.5
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.42
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	10.7
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c*l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	10.7
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.29
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75*0.6*70*(1+0.5*\sin^{1.5}(\theta_r))*(p_i(1.9-0.9p_i))^{0.3}*0.707*D_{W1}/16)$	ϕr_n (kips)	12.64
Cover plate to flange weld tearing check, $r_u/\phi r_n$	*D/C*	0.84 **OK**

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.5
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang dimension	*C3* (in)	3
External continuity plate dimension	*C5* (in)	3
Weld 2 size	D_{W2} (1/16 in)	8
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.50
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc}+t_{cp}+[C3]-l_{hb2}$	$l_{we2,eff}$ (in)	4.34
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe}+P_d)/(d_c+2[C3])$	r_{uv2} (kip/in)	16.5
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe}+P_d)[C5]/(l_{we2,eff}(d_c+2[C3]-2l_{hb2}-l_{we2,eff}))$	r_{un2} (kip/in)	14.2
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	21.8
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2\ sides)$	ϕr_n (kip/in)	22.3
External continuity plate-to-cover plate weld tearing check, $r_u/\phi r_n$	*D/C*	0.98 **OK**

Failure of external continiuty plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	361.0
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	1372.4
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	457.5
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c+2[C3])T2$	ϕR_n (kips)	934.9
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c+2*[C3])/2)^2$	ϕM_n (kips)	9711.0
Demand/Capacity Ratio, $R_u/\phi R_n+M_u/\phi M_n$	*D/C*	0.63 **OK**

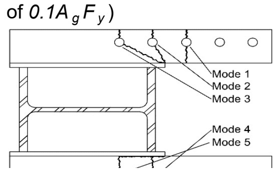
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

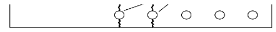
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.5
Column fillet dimension	*k* (in)	1.5
Column fillet dimension	*k1* (in)	1.5
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	91.0
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	43.6
Weld 2 size	D_{W2} (1/16 in)	8
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1350.5
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	647.3
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl}+R_{nwt})$ or $(0.85R_{nwl}+1.5R_{nwt})$	R_n (kips)	2118.9
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1589.2
Weld 2 demand/capacity check for cap plate configuration, $R_u/\phi R_n$	*D/C*	0.29 **OK**

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC *Specification* Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.5
Number of bolts ahead of rupture, n_b-n_m	n_{rup}	4.0
Mode 1 bolt rupture demand, $(V_{fe}+P_d)n_{rup}/n_b$	P_u (kips)	228.7
External continuity plate thickness (see step 2)	t_p (in)	1.125
External continuity plate width	*C4* (in)	6.25





Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, $t_p([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	5.555	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	361.1	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	270.8	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.84	**OK**

Mode 2: Rupture through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	5	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	285.9	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	3	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	2.125	
External continuity plate thickness (see step 2)	t_p (in)	1.125	
External continuity plate width	*C4* (in)	6.25	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	*Bonus* (in)	0.423	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	5.978	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	388.6	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	291.4	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.98	**OK**

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	6	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	343.1	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	3	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5.5	
External continuity plate thickness (see step 2)	t_p (in)	1.125	
External continuity plate width	*C4* (in)	6.25	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	*Bonus* (in)	2.836	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $(t_p)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	8.391	
Nominal tensile rupture capacity, A_nF_{up}	P_n (kips)	545.4	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	409.0	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.84	**OK**

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	5	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	285.9	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	3	
External continuity plate width	*C4* (in)	6.25	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	2.125	
External continuity plate thickness (see step 2)	t_p (in)	1.125	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, $t_p[C4$	A_{gt} (in^2)	7.031	
Gross shear area, $t_p[C9]$	A_{gv} (in^2)	2.391	
Net tension area, $A_{gt} - ((t_p)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	5.555	
Net shear area, A_{gv}	A_{nv} (in^2)	2.391	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	432.8	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	324.6	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.88	**OK**

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	6	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	343.1	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	3	
External continuity plate width	*C4* (in)	6.25	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5.5	
External continuity plate thickness (see step 2)	t_p *(in)*	1.125	

Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, t_p *[C4]*	A_{gt} (in^2)	7.031	
Gross shear area, t_p *[C7]*	A_{gv} (in^2)	6.188	
Net tension area, $A_{gt}-((t_p)(d_{hole}+1/16))$	A_{nt} (in^2)	5.555	
Net shear area, A_{gv}	A_{nv} (in^2)	6.188	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	546.7	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	410.0	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.84	**OK**

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Bay width	B (in)	360.0	
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	7.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	18500	
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	116.4	
Beam depth	d_b (in)	24.5	
Beam web thickness	t_{wb} (in)	0.605	
Beam web area, $d_b t_{wb}$	A_w (in)	14.8225	
Beam web width-to-thickness ratio, h / t_w	l_{web}	35.6	
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34	
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1	
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	444.7	
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	*D/C*	0.26	**OK**

*Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2*
C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	96.5	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	915.0	
Beam flange thickness	t_{fb} (in)	0.96	
Beam flange width	b_{fb} (in)	12.9	
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25	
Number of holes along each bolt line (see step 5)	n_b	8	
Length of connection on beam	*B4* (in)	25.25	
Distance from beam centerline to bolt line	*P2* (in)	2.75	
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	7.104	
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	48.480	
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	5.844	
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	29.580	
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	1533.5	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1150.1	
Demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.80	**OK**

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	116.4	
Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	0.994	
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	83.5	
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	62.6	
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	2	
Provided number of shear tab bolts	n_n	4	**OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20	
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - 1in	$l_{tab,req}$ (in)	19	
Length of shear tab (see drawings), T_{beam} - 1in	l_{tab} (in)	19	**OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.605	
Shear tab thickness	*T3* (in)	0.5	

Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\ t_{min}\ F_{up}\ n_n$	R_{nb} (kips)	351.0
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.25
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.500
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	5.333
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.1
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_n - 1)$	R_{nt1} (kips)	477.8
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.9
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	34.1
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	511.9
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	334.0
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	334.0
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	250.5
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.46 **OK**

Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	80
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	40.7
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	162.7
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	116.4
Distance from alignment line to column	$C1$ (in)	6.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	712.8

Shear tab rupture through net section check		
Bolt diameter (see step 4)	d_{bolt} (in)	1.125
Slotted bolt hole width (standard slot width)	w_b (in)	1.25
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.5
Slotted bolt hole spacing, $(T_{beam} - 1'' - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.25
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in^2)	6.88
Effective net area, $A_e = A_{nv}$	A_e (in^2)	6.88
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	201.09
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{nn} (kips)	335.16
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.35 **OK**

Shear tab yielding check		
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	116.37
Required axial strength of the shear tab (see above)	P_u (kips)	162.72
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	712.77
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in^3)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{nn} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/cacpacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.52 **OK**

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)		
Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.25
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	116.4
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	5.2
Number of slotted bolt holes in shear tab	n_{vb}	4

Slotted bolt hole spacing	s_{slot} (in)	5.25	
Distance from line of bolts to shear tab weld	$C1$ (in)	6.125	
Size of weld in 1/16^{ths} of an inch	D_{W3}	4	
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	712.8	
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	7.3	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	9.0	
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W3}/16(1")(2\ sides)$	ϕr_n (kips)	11.1	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.81	**OK**

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.625	
Beam depth	d_b (in)	24.5	
Length of the shear tab (see step 12)	l_{tab} (in)	19	
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06	
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.77	
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.875	**OK**
Provided outside shear tab slot dimension, *[S2] = [S1]*	$S2$ (in)	1.875	

Step 15. Check the top plate for shear yielding and shear rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.5	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	457.5	

Yielding check

Top plate thickness (see step 2)	t_p (in)	1.125	
Top plate dimension (see schedule)	$P6$ (in)	27.125	
Gross shear area of top plate, $(P6)(t_p)$	A_g (in^2)	30.516	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	915.5	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	915.5	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.50	**OK**

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.5	
Number of short slotted bolts on top plate (see step 5)	n_b	8	
Net shear area of top plate, $A_g-(t_p l_s n_b)$	A_n (in^2)	17.016	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{up}$	R_n (kips)	663.6	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	497.7	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.92	**OK**

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.5	
Total number of bolts (see step 5)	n_b	8	
Number of bolts in the M region (see step 5)	n_m	4	
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	228.7	

Yielding check

Top plate dimension	$P4$ (in)	1.875	
Top plate dimension	$P5$ (in)	3.875	
Top plate thickness (see step 2)	t_p (in)	1.125	
Gross area, $([P4]+[P5])*(t_p)$	A_g (in^2)	6.469	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	323.4	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	291.1	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.79	**OK**

Rupture check

Top plate dimension	$P4$ (in)	1.875	

Top plate dimension	$P5$ (in)	3.875
Top plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Effective area of rupture, $([P4]+[P5]-d_O-1/16)(t_p)$	A_e (in^2)	4.781
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	310.8
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	233.1
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98 **OK**

Step 17

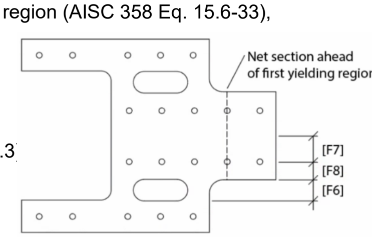
Entering and tightening check		
Beam flange fillet dimension	k_1	1.5
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.4375
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	2.9375
Provided P2 dimension	$P2$ (in)	2.75 **OK**

Top plate yielding due to combined flexure and shear check		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	96.5
Top plate thickness (see step 2)	t_p (in)	1.125
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	10.075
Inside length of the top plate	$P8$ (in)	27.13
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp}t_p)$	m (in)	15.061
Dimension, $([P8] - m) / 2$	e (in)	6.032
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-4.0
Provided P2 dimension	$P2$ (in)	2.75 **OK**

Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC *Specification* Chapter D)

Fuse plate yielding region proportions		
Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.5 **OK**
Top plate thickness (see step 2)	t_p (in)	1.125
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.2 **OK**

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	n_b	8
Number of bolts in the M region (see step 5)	n_m	4
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	361.0
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.95
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3	d_O (in)	1.4375
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in^2)	12.825
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	641.3
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	577.1
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.63 **OK**



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)		
Fuse plate dimension	$F7$ (in)	2.75
Fuse plate dimension	$F8$ (in)	2.95
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.438
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)t_p$	A_e (in^2)	9.450
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	P_n (kips)	737.1
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	552.8

Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	*D/C*	0.65 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Step 19. Determine the yielding region depth [F2]

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	361.0
Fuse plate thickness(see step 2)	t_p (in)	1.125
Width of fuse plate cut-out	*F6* (in)	2.5
Coefficient in the strain hardening expression, calibrated from experiments	*A*	1.52
Coefficient in the strain hardening expression, calibrated from experiments	*B*	0.16
Coefficient in the strain hardening expression, calibrated from experiments	*C*	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6]))/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.770
Provided yielding region depth	*F2* (in)	2.750 **OK**
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	185.6
Ratio	V_{fe} / V_y	1.9

Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, 0.5 <= *[F6] / [F2] <= 1.25*	*F6 / F2*	0.91 **OK**

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	n_b	8
Number of bolts in the M region	n_m	4
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	361.0

Yielding check		
Fuse plate dimension	*F4* (in)	1.875
Fuse plate dimension	*F5* (in)	2.375
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in^2)	9.5625
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	R_n (kips)	478.13
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	430.31
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.84 **OK**

Rupture check		
Fuse plate dimension	*F4* (in)	1.875
Fuse plate dimension	*F5* (in)	2.375
Fuse plate thickness (see step 2)	t_p (in)	1.125
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(t_p)$	A_e (in^2)	6.1875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	R_n (kips)	482.6
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	362.0
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	1.00 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	96.5
External continuity plate dimension	*C5* (in)	3
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	55.6

Cover plate overhang	$C3$ (in)	3	
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.875	
Bridge plate factored tensile capacity, $0.9*F_{yp}\,t_p([C3]-1")$	ϕP_n (kips)	78.8	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	D/C	0.71	**OK**

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	55.6	
Weld 5 size	D_{W5} (1/16 in)	7	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16\,([C3]-1")(2\ sides)$	ϕR_n (kips)	58.5	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.95	**OK**

Shear tab to bridge plate weld failure check

Bay width	B (in)	360.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.5	
Shear force from gravity	$V_{gravity}$ (kips)	7.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	18500	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	116.4	
Distance from alignment line to column	$C1$ (in)	6.125	
Weld 7 demand, $V_u\,C1/d_b$	R_u (kips)	29.1	
Weld 7 size	D_{W7} (1/16 in)	4	
Cover plate overhang	$C3$ (in)	3	
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(FEXX)(0.707)D_w/16[C3](2\ sides)$	ϕR_n (kips)	33.4	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.87	**OK**

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	116.4	
Distance from alignment line to column	$C1$ (in)	6.125	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	29.1	
Weld 6 size	D_{W6} (1/16 in)	5	
Cover plate overhang	$C3$ (in)	3	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc}$ - 2", $2[C3])$	l_{eff} (in)	6	
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	62.6	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.5	**OK**

Shear plate shear failure check (when W1 > 3/4" without shear plates, shear plates are used)

Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Weld 4 size	D_{W4} (1/16 in)	7	
Weld 4 effective length, $d_c - 2\,t_{fc}$	l_{eff} (in)	19.78	
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16\ (leff)$	R_u (kips)	192.7	
Shear plate thickness, t_{sp} >= 1.1*W4 or 3/8"	t_{sp} (in)	0.500	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}\,t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	296.7	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6	**OK**

Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

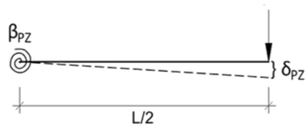
Derivation

Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

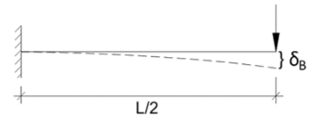
Effective Panel Zone Stiffness P

Beam Stiffness P

$K_{PZ} = P/\delta_{PZ}$
$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$
$K_{PZ} = \beta_{PZ}/(L/2)^2$



$K_B = P/\delta_B$
$\delta_B = P(L/2)^3/3EI$
$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$
$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$
---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.5	
Beam flange thickness	t_{fb} (in)	0.96	
Column depth	d_c (in)	21.7	
Column flange width	b_{fc} (in)	12.4	
Column web thickness	t_{wc} (in)	0.6	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang	*C3* (in)	3.0	
Cover plate thickness	t_{cp} (in)	0.875	
Bay width	*B* (in)	360.0	
Story height	*H* (in)	216.0	
Panel zone factor, $d_b d_c t_{wc} + 2d_b(d_c+2[C3])t_{cp}$	V_p (in^3)	1506.6	
Column ratio, $(d_c - t_{fc})/B$	α	0.058	
Beam ratio, $(d_b - t_{fb})/H$	β	0.109	
Web and cover plate stiffness, $GV_p/(1-\alpha-\beta)^2$	K_{ps} (k-in)	24294569	
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2)/(1-\alpha-\beta)^2$	K_{fs} (k-in)	143735	
Panel zone spring stiffness, $K_{ps}+K_{fs}$	β_{PZ} (k-in)	24438304	
Beam moment of inertia	I_{xb} (in^3)	4020	
Required panel zone spring stiffness , *$60EI_{xb}/B$ for one-sided, $120EI_{xb}/B$ for two-sided*	K_{req} (k-in)	19429999	
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.8	**OK**

Additional calculations for connections with orthogonal forces from cantilevers or drags

Connection demands (one side)			
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	361.0	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	193.0	
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	96.5	
Orthogonal drag load (from EOR)	$P_{d,do}$ (kips)	0.0	
Moment from orthgonal connection (seismic load combination) (from EOR)	$M_{u,o}$ (k-in)	391	
Orthogonal beam size		W24X55	
Orthogonal beam depth	$d_{b,o}$ (in)	23.6	
Orthogonal beam flange thickness	$t_{fb,o}$ (in)	0.51	
Flange force associated with $M_{u,o}$, $M_{u,o}/(d_{b,o}-t_{fb,o})$	$P_{d,mo}$ (kips)	16.9	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	16.9	
Column flange width	b_{fc} (in.)	12.4	
Column depth	d_c (in.)	21.7	
Column flange thickness	t_{fc} (in.)	0.96	
Column web thickness	t_{wc} (in.)	0.60	
Plate thickness (see T2 in drawings)	t_p (in)	1.13	
External continuity plate gage dimension	*C5* (in.)	3.0	
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)/(1+0.5t_{wc}/t_p)]$	R_{cp} (kips)	361.2	
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	192.8	
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	16.9	

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	1372.4	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	16.9	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	457.5	
Plate plastic section modulus, $(d_c - 2t_{fc})^2 t_p/4$	*Z* (in^3)	110.04	
Plate section area, $(d_c - 2t_{fc})t_p$	*A* (in^2)	22.25	
Moment capacity at section, $0.9F_{yc}Z$	M_c (k-in)	4951.7	
Normal capacity at section, $0.9F_{yc}A$	P_c (kips)	1001.4	
Shear capacity at section, $0.75(0.6F_{uc})A$	V_c (kips)	650.9	
Interaction equation, $(M_r/M_c)+(P_r/P_c)^2+(V_r/V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.52	**OK**

External continuity plate section check (at the column web)

Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])\text{-}R_{cp}(b_{fc}/2)$	M_r (k-in)	2857.4
Normal demand at section, $=P_{d,o}$	P_r (kips)	16.9
Shear demand at section, $=R_{w1}$	V_r (kips)	192.8
Corner clip dimension	K_c (in)	1.46
Plate plastic section modulus, $(d_c\text{-}2t_{fc}\text{-}2K_c)^2 t_p/4$	Z (in^3)	79.9
Plate section area, $(d_c\text{-}2t_{fc}\text{-}2K_c)t_p$	A (in^2)	18.97
Moment capacity at section, $0.9F_{yc}Z$	M_c (k-in)	3597.7
Normal capacity at section, $0.9F_{yc}A$	P_c (kips)	853.5
Shear capacity at section, $0.75(0.6F_{uc})A$	V_c (kips)	554.8
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.81 **OK**

HCAI Calculations

DuraFuse Frames® Calculations

Connection ID DF250

W21X122 Column and W24X94 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	E (ksi)	29000
Shear modulus (AISC 360-22)	G (ksi)	11200

Beams		
Beam material grade	*Gr.*	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1

Columns		
Column material grade	*Gr.*	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1

Plates		
Plate material (AISC 358-22 15.3.3)	*Gr.*	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2

Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	*Gr.*	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks			
Beam size		W24X94	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40	
Beam depth	d_b (in.)	24.3	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	232	
Beam weight	W_b (lb/ft)	94	**OK**
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6	
Beam web width-to-thickness ratio	λ_{web}	41.9	**OK**
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), 0.38(E/Fyb)^0.5	$\lambda_{p,flange}$	9.2	
Beam flange width-to-thickness ratio	λ_{flange}	5.18	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		W21X122	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36	
Column depth	d_c (in.)	21.7	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	122	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	0.96	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	53.5	
Gross area of the column section	A_g (in^2)	35.9	
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.0	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	57.2	
Column web width-to-thickness ratio	λ_{web}	31.3	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	

Column flange width-to-thickness ratio	λ_{flange}	6.45 **OK**

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in^3)	307
Column design axial load	P_u (kips)	53.5
Column area	A_{gc} (in^2)	35.9
Story Height	H (in)	216.0
Column depth	d_c (in)	21.7
Beam depth	d_b (in)	24.3
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma R_{yc} Z_c *(F_{yc} - P_u / A_{gc})*(H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	20293
Beam plastic section modulus	Z_b (in^3)	254
Bay width	B (in)	360.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	9535
Clear distance between column faces, $B-d_c$	L_h (in)	338.3
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	7.0
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	63.4
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	688
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	20445
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	$Ratio > 1.0$	0.99 **NA**

*Not applicable for top stories, see AISC 341 E3.4a(a1))

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

Beam moment demand from load combinations (from EOR)	M_u (k-in)	3422
Beam plastic section modulus	Z_b (in^3)	254
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	12700
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	9535 **OK**
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.875
Beam depth	d_b (in)	24.3
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	189.4
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	189.4
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	378.7

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
Beam area	A_{gb} (in^2)	27.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	56.0
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	434.7
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	35
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.875
Beam depth	d_b (in)	24.3
Column depth	d_c (in)	21.7
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	489.4

Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)

Cover plate thickness (see T1 in drawings)	t_{cp} (in)	1
Cover plate overhange dimension	$C3$ (in)	3.5
Column depth	d_c (in)	21.7
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	28.7
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} d_{cp}$	$R_{n,horiz}$ (kips)	861.0
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	861.0
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.50 **OK**

Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)

Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	26.43

Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} h_{cp}$	$R_{n,vert}$ (kips)	792.8
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	792.8
Shear yielding demand/capacity ratio, $R_{u,vert}$ / $\phi R_{n,vert}$	D/C	0.62 **OK**

Cover plate thickness check (AISC 341 E3.6e.2)

Beam depth	d_b (in)	24.3
Beam flange thickness	t_{fb} (in)	0.875
Column flange thickness	t_{fc} (in)	0.96
Panel zone vertical dimension, d_b - $2t_{fb}$	d_z (in)	22.55
Panel zone horizontal diemension, d_c - $2t_{fc}$	w_z (in)	19.78
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470
Thickness check, t_{cp} / t_{req}	$t_{cp} / t_{req} > 1.0$	2.13 **OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.125
Beam plastic section modulus	Z_b (in^3)	254
Beam flange thickness	t_{fb} (in)	0.875
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	1.094
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	51.2
Beam plastic section modulus of the net section, Z_b - $Z_{bolt\ holes}$	$Z_{b,net}$	202.8
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	13970
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	14497
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.96 **OK**

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0
Bolt diameter	d_{bolt} (in)	1
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	0.785
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	66.0
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	3.7
Number of bolts per line (AISC 358 15.2(1))	n_b	5 **OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.375
Bolt spacing (see schedule)	s (in)	3.25
Plate thickness, t_p (see T2 in drawings)	t_p (in)	0.875
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up} R_{tp} t_p)]$	$F2_a$ (in)	2.312
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	2.5
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	3 **OK**
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), n_b - n_p	n_m	2

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	245.4
Beam flange thickness	t_{fb} (in)	0.875
Fuse and top plate thickness	$T2$ (in)	0.875
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	0.9
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	682.5
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, $s - d_O$	l_{c1} (in)	2.0
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b - 1)$	R_{nt1} (kips)	546.0
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, $e_d - d_O / 2$	l_{c2} (in)	0.8
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	51.2
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	597.2
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	329.9
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	329.9
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	247.4
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.99 **OK**

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1451
Bolt group demand from wind per line, $M_u/(2(d_b+t_p))$	R_u *(kip)*	28.8
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	64
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	5
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	108.5
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	92.2
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.31 **OK**

Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	24.3
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.375
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$*, 1.0 otherwise*	α	0
Cover plate overhang dimension	*C3* (in)	3.5
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	2.833
Distance from alignment line to column	*C1* (in)	6.875 **OK**

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	56
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	35
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	24.3
Plate thickness (see T2 on drawings)	t_p (in)	0.875
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	1
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2{*}t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	26.425
Gage dimension from the cover plate to the line of bolts on the external continuity plates	*C5* (in)	2.75
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} *(in)*	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	25.43
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.88
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	6
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.87
Vertical distance from center of rotation to geometric center	e_y (in)	0.54

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d){*}L$	ΣRL (k-in)	16066

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R{*}L_R$ (k-in)
W1RT	16.39	7.53	52.49	123.4
W4TL	18.71	5.25	34.38	98.1
W1LB	16.72	7.27	45.48	121.6
W4BR	16.85	5.23	31.56	88.1
W1RC	10.01	4.97	4.55	49.7
W4TC	15.48	4.19	3.96	64.9
W1LC	11.75	5.10	3.88	59.9
W4BC	14.40	4.15	4.26	59.8
		7.53	52.49	

Weld strength check according to AISC 360 Chapter J		
Force on Critical Weld Segment	R_{crit} (kips)	7.5

Orientation of Resultant Force	θ_r (degrees)	52.5
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.51
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	9.4
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c*l_{We1})$	r_{un} (kip/in)	0.0
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	9.4
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.31
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75*0.6*70*(1+0.5*\sin^{1.5}(\theta_r))*(p_i(1.9-0.9p_i))^{0.3}*0.707*D_{W1}/16)$	ϕr_n (kips)	11.11
Cover plate to flange weld tearing check, $r_u/\phi r_n$	D/C	0.85 **OK**

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0000
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang dimension	C3 (in)	3.5
External continuity plate dimension	C5 (in)	2.75
Weld 2 size	D_{W2} (1/16 in)	7
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.44
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc}+t_{cp}+[C3]-l_{hb2}$	$l_{we2,eff}$ (in)	5.02
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe}+P_d)/(d_c+2[C3])$	r_{uv2} (kip/in)	15.1
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe}+P_d)[C5]/(l_{we2,eff}(d_c+2[C3]-2l_{hb2}-l_{we2,eff}))$	r_{un2} (kip/in)	10.4
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	18.4
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2\ sides)$	ϕr_n (kip/in)	19.5
External continuity plate-to-cover plate weld tearing check, $r_u/\phi r_n$	D/C	0.94 **OK**

Failure of external continiuty plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	1195.6
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	434.7
External continuity plate thickness (see step 2)	t_p (in)	0.875
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c+2[C3])T2$	ϕR_n (kips)	753.4
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c+2*[C3])/2)^2$	ϕM_n (kips)	8108.2
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.72 **OK**

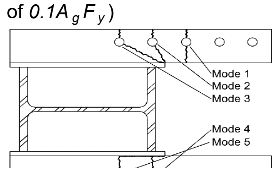
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

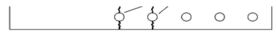
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	378.7
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0
Column fillet dimension	k (in)	1.5
Column fillet dimension	$k1$ (in)	1.5
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	93.2
Column flange width	b_{fc} (in)	12.4
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	43.6
Weld 2 size	D_{W2} (1/16 in)	7
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1210.9
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	566.4
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl}+R_{nwt})$ or $(0.85R_{nwl}+1.5R_{nwt})$	R_n (kips)	1878.9
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1409.2
Weld 2 demand/capacity check for cap plate configuration, $R_u/\phi R_n$	D/C	0.31 **OK**

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC *Specification* Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0
Number of bolts ahead of rupture, n_b-n_m	n_{rup}	3.0
Mode 1 bolt rupture demand, $(V_{fe}+P_d)n_{rup}/n_b$	P_u (kips)	147.2
External continuity plate thickness (see step 2)	t_p (in)	0.875
External continuity plate width	C4 (in)	4.75





Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Net area of external continuity plate cross section, t_p *([C4]-(*$d_{hole,std}$ *+1/16))*	A_n (in^2)	3.117
Nominal tensile rupture capacity, $F_{up} A_n$	P_n (kips)	202.6
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	152.0
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.97 **OK**

Mode 2: Rupture through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	196.3
External continuity plate dimension	*C2 (in)*	0.375
Cover plate overhang dimension	*C3* (in)	3.5
External continuity plate dimension	*C5* (in)	2.75
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	3.875
External continuity plate thickness (see step 2)	t_p (in)	0.875
External continuity plate width	*C4* (in)	4.75
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (t_p)/4[C5]$	*Bonus* (in)	1.194
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Area of external continuity plate cross section with bonus, t_p *([C4]-*$d_{hole,std}$*-1/16) + Bonus*	A_n (in^2)	4.312
Nominal tensile rupture capacity, $F_{up} A_n$	P_n (kips)	280.3
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	210.2
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.93 **OK**

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	245.4
Bolt spacing (see schedule)	*s* (in)	3.25
External continuity plate dimension	*C2 (in)*	0.375
Cover plate overhang dimension	*C3* (in)	3.5
External continuity plate dimension	*C5* (in)	2.75
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	7.125
External continuity plate thickness (see step 2)	t_p (in)	0.875
External continuity plate width	*C4* (in)	4.75
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (t_p)/(4[C5])$	*Bonus* (in)	4.038
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Area of external continuity plate cross section with bonus, (t_p)*([C4]-*$d_{hole,std}$*-1/16) + Bonus*	A_n (in^2)	7.155
Nominal tensile rupture capacity, $A_n F_{up}$	P_n (kips)	465.1
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	348.8
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.70 **OK**

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	196.3
External continuity plate dimension	*C2 (in)*	0.375
Cover plate overhang dimension	*C3* (in)	3.5
External continuity plate width	*C4* (in)	4.75
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	3.875
External continuity plate thickness (see step 2)	t_p (in)	0.875
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Gross tension area, t_p*[C4*	A_{gt} (in^2)	4.156
Gross shear area, t_p*[C9]*	A_{gv} (in^2)	3.391
Net tension area, $A_{gt} - ((t_p)*(d_{hole,std} + 1/16))$	A_{nt} (in^2)	3.117
Net shear area, A_{gv}	A_{nv} (in^2)	3.391
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up} A_{nv} + F_{up} A_{nt} < 0.6F_{yp} A_{gv} + F_{up} A_{nt}$	R_n (kips)	304.3
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	228.3
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.86 **OK**

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	245.4
Bolt spacing (see schedule)	*s* (in)	3.25
External continuity plate dimension	*C2 (in)*	0.375
Cover plate overhang dimension	*C3* (in)	3.5
External continuity plate width	*C4* (in)	4.75
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	7.125
External continuity plate thickness (see step 2)	t_p *(in)*	0.875

Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Gross tension area, t_p *[C4]*	A_{gt} (in^2)	4.156
Gross shear area, t_p *[C7]*	A_{gv} (in^2)	6.234
Net tension area, $A_{gt}-((t_p)(d_{hole}+1/16))$	A_{nt} (in^2)	3.117
Net shear area, A_{gv}	A_{nv} (in^2)	6.234
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	389.6
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	292.2
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.84 **OK**

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	7.0
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	9535
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	63.4
Beam depth	d_b (in)	24.3
Beam web thickness	t_{wb} (in)	0.515
Beam web area, $d_b t_{wb}$	A_w (in)	12.5145
Beam web width-to-thickness ratio, h / t_w	l_{web}	41.9
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	375.4
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	*D/C*	0.17 **OK**

*Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2*
C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	490.7
Beam flange thickness	t_{fb} (in)	0.875
Beam flange width	b_{fb} (in)	9.07
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.125
Number of holes along each bolt line (see step 5)	n_b	5
Length of connection on beam	*B4* (in)	14.375
Distance from beam centerline to bolt line	*P2* (in)	2.75
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	3.124
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	25.156
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	2.085
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	15.805
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	751.9
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	563.9
Demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.87 **OK**

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	63.4
Bolt diameter (see step 4)	d_{bolt} (in)	1
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	0.785
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	66.0
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	49.5
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	1
Provided number of shear tab bolts	n_n	4 **OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - 1in	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), T_{beam} - 1in	l_{tab} (in)	19 **OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.515
Shear tab thickness	*T3* (in)	0.5

Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	0.5
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\ t_{min}\ F_{up}\ n_n$	R_{nb} (kips)	312.0
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.125
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.250
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	5.500
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.4
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_n - 1)$	R_{nt1} (kips)	511.9
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.7
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	26.8
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	538.7
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	263.9
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	263.9
Factored shear capacity of all bolts, $0.75{*}R_n$	ϕR_n (kips)	197.9
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.32 **OK**

Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	64
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	32.5
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	130.2
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	63.4
Distance from alignment line to column	$C1$ (in)	6.875
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	499.8

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1
Slotted bolt hole width (standard slot width)	w_b (in)	1.125
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.25
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.375
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) t_{tab}$	A_{nv} (in^2)	7.13
Effective net area, $A_e = A_{nv}$	A_e (in^2)	7.13
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	208.41
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{nn} (kips)	347.34
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.15 **OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	63.37
Required axial strength of the shear tab (see above)	P_u (kips)	130.18
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	499.83
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in^3)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{nn} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/cacpacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.34 **OK**

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	22.55
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	63.4
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	2.8
Number of slotted bolt holes in shear tab	n_{vb}	4

Slotted bolt hole spacing	s_{slot} (in)	5.375
Distance from line of bolts to shear tab weld	$C1$ (in)	6.875
Size of weld in 1/16^{ths} of an inch	D_{W3}	3
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	499.8
Normal demand on weld $(3F_b s_{slot} n_{vb}{}^2 + 6M_u)/l_{w3}{}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	5.8
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	6.4
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W3}/16(1")(2\ sides)$	ϕr_n (kips)	8.4
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.77 **OK**

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.375
Beam depth	d_b (in)	24.3
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.7165
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.75 **OK**
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.75

Step 15. Check the top plate for shear yielding and shear rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	245.4

Yielding check

Top plate thickness (see step 2)	t_p (in)	0.875
Top plate dimension (see schedule)	$P6$ (in)	17
Gross shear area of top plate, $(P6)(t_p)$	A_g (in^2)	14.875
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	446.3
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	446.3
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.55 **OK**

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.3125
Number of short slotted bolts on top plate (see step 5)	n_b	5
Net shear area of top plate, $A_g-(t_p l_s n_b)$	A_n (in^2)	9.133
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	356.2
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	267.1
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.92 **OK**

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0
Total number of bolts (see step 5)	n_b	5
Number of bolts in the M region (see step 5)	n_m	2
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	98.1

Yielding check

Top plate dimension	$P4$ (in)	1.875
Top plate dimension	$P5$ (in)	1.75
Top plate thickness (see step 2)	t_p (in)	0.875
Gross area, $([P4]+[P5])*(t_p)$	A_g (in^2)	3.172
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	158.6
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	142.7
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.69 **OK**

Rupture check

Top plate dimension	$P4$ (in)	1.875

Top plate dimension	$P5$ (in)	1.75
Top plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25
Effective area of rupture, $([P4]+[P5]-d_O-1/16)(t_p)$	A_e (in^2)	2.023
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	131.5
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	98.6
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	1.00 **OK**

Step 17

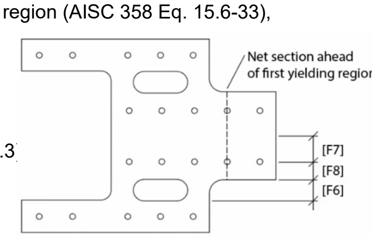
Entering and tightening check		
Beam flange fillet dimension	k_1	1.4
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.25
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	2.6875
Provided P2 dimension	$P2$ (in)	2.75 **OK**

Top plate yielding due to combined flexure and shear check		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0
Top plate thickness (see step 2)	t_p (in)	0.875
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	9.95
Inside length of the top plate	$P8$ (in)	17.00
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp}t_p)$	m (in)	10.386
Dimension, $([P8] - m) / 2$	e (in)	3.307
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	2.7
Provided P2 dimension	$P2$ (in)	2.75 **OK**

Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC *Specification* Chapter D)

Fuse plate yielding region proportions		
Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5
Provided width of yielding region	$F6$ (in)	2.375 **OK**
Top plate thickness (see step 2)	t_p (in)	0.875
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5
Provided width-thickness ratio of yielding region	$F6/t_p$	2.7 **OK**

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Number of bolts in the B region (see step 5)	n_b	5
Number of bolts in the M region (see step 5)	n_m	2
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	151.5
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.95
Fuse plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3	d_O (in)	1.25
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in^2)	9.975
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	498.8
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	448.9
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.34 **OK**



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)		
Fuse plate dimension	$F7$ (in)	2.75
Fuse plate dimension	$F8$ (in)	2.95
Fuse plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.250
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)t_p$	A_e (in^2)	7.678
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	P_n (kips)	598.9
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	449.2

Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	*D/C*	0.34 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Step 19. Determine the yielding region depth [F2]

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	189.4
Fuse plate thickness(see step 2)	t_p (in)	0.875
Width of fuse plate cut-out	*F6* (in)	2.375
Coefficient in the strain hardening expression, calibrated from experiments	*A*	1.52
Coefficient in the strain hardening expression, calibrated from experiments	*B*	0.16
Coefficient in the strain hardening expression, calibrated from experiments	*C*	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6]))/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	2.020
Provided yielding region depth	*F2* (in)	2.000 **OK**
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	105.0
Ratio	V_{fe} / V_y	1.8

Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, $0.5 <= [F6] / [F2] <= 1.25$	*F6 / F2*	1.19 **OK**

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	n_b	5
Number of bolts in the M region	n_m	2
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	151.5

Yielding check		
Fuse plate dimension	*F4* (in)	1.875
Fuse plate dimension	*F5* (in)	1.375
Fuse plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in^2)	5.6875
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	R_n (kips)	284.38
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	255.94
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.59 **OK**

Rupture check		
Fuse plate dimension	*F4* (in)	1.875
Fuse plate dimension	*F5* (in)	1.375
Fuse plate thickness (see step 2)	t_p (in)	0.875
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(t_p)$	A_e (in^2)	3.390625
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	R_n (kips)	264.5
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	198.4
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.76 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	56
External continuity plate dimension	*C5* (in)	2.75
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	26.8

Cover plate overhang	*C3* (in)	3.5	
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.625	
Bridge plate factored tensile capacity, $0.9^*F_{yp}\,t_p$ ([C3]-1")	ϕP_n (kips)	70.3	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	0.38	**OK**

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)$[C5]/(d_c+[C3])	R_u (kips)	26.8	
Weld 5 size	D_{W5} (1/16 in)	5	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16$ ([C3]-1")(2 sides)	ϕR_n (kips)	52.2	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.51	**OK**

Shear tab to bridge plate weld failure check

Bay width	*B* (in)	360.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.3	
Shear force from gravity	$V_{gravity}$ (kips)	7.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	9535	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	63.4	
Distance from alignment line to column	*C1* (in)	6.875	
Weld 7 demand, $V_u\,C1/d_b$	R_u (kips)	17.9	
Weld 7 size	D_{W7} (1/16 in)	3	
Cover plate overhang	*C3* (in)	3.5	
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(FEXX)(0.707)D_w/16$[C3](2 sides)	ϕR_n (kips)	29.2	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.61	**OK**

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	63.4	
Distance from alignment line to column	*C1* (in)	6.875	
Weld 6 demand, V_u[C1]/d_b	R_u (kips)	17.9	
Weld 6 size	D_{W6} (1/16 in)	4	
Cover plate overhang	*C3* (in)	3.5	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, MIN(b_{fc} - 2", 2[C3])	l_{eff} (in)	7	
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	58.5	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.3	**OK**

Shear plate shear failure check (when W1 > 3/4" without shear plates, shear plates are used)

Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Weld 4 size	D_{W4} (1/16 in)	6	
Weld 4 effective length, $d_c - 2\,t_{fc}$	l_{eff} (in)	19.78	
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16$ (leff)	R_u (kips)	165.2	
Shear plate thickness, t_{sp} >= 1.1*W4 or 3/8"	t_{sp} (in)	0.500	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}\,t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	296.7	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**

Connection stiffness check

The purpose of this check is to ensure that the connection has enough stiffness to be considered Fully Restrained (FR) when M_{pr} is less than M_p (see AISC 358-22 15.6 Step 2). Moment frame connections are considered FR for design purposes if the connection stiffness is large relative to the beam flexural stiffness (EI/L). A threshold of 18EI/L is used in Chapter 13 of AISC 358 based on experimental testing. This procedure follows the method in the commentary to AISC 358-22 15.6 Step 2.

Beam moment of inertia	I_{xb} (in^3)	2700
Column flange width	b_{fc} (in)	12.4
Beam depth	d_b (in)	24.3
Bay width	*B* (in)	360.0
Number of bolts in zone M	M	2
Number of bolts in zone P	P	3
Bolt spacing	*s* (in)	3.25
Cover plate thickness	t_{cp} (in)	1
Fuse plate thickness	t_p (in)	0.875
External continuity plate dimension	*C5* (in)	2.75

Fuse plate dimension	$F2$ (in)	2
Fuse plate dimension	$F3$ (in)	5.875
Fuse plate dimension	$F4$ (in)	1.9
Fuse plate dimension	$F6$ (in)	2.375
Fuse plate dimension	$F8$ (in)	2.95
Distance from beam centerline to bolt line	$P2$ (in)	2.75
Fuse region 1 stiffness, $(2[F2] + [F3])t_p G / [F4]$	K_1 (k/in)	51613.333
Fuse region 2 stiffness, $2[F2]t_p G / [F6]$	K_2 (k/in)	16505.3
Fuse region 3 stiffness, $2(0.37)Et_p [F2]^3/[F6]^3$	K_3 (k/in)	11213.4
Fuse region 4 stiffness, $(2[F2] + [F3])t_p G / [F8]$	K_4 (k/in)	32805.1
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	10018.8
Top plate stiffness, $(2P + M - 1)st_p G/(b_{fc}/2+ t_{cp} + [C5] - [P2])$	K_{top} (k/in)	30965.3
Connection stiffness, $(d_b + t_p)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	4797512
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	3915000
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.8 **OK**

Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

Derivation

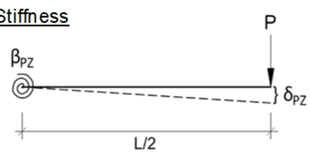
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$

$K_{PZ} = \beta_{PZ}/(L/2)^2$

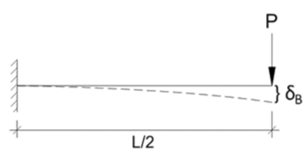


Beam Stiffness

$K_B = P/\delta_B$

$\delta_B = P(L/2)^3/3EI$

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.3
Beam flange thickness	t_{fb} (in)	0.875
Column depth	d_c (in)	21.7
Column flange width	b_{fc} (in)	12.4
Column web thickness	t_{wc} (in)	0.6
Column flange thickness	t_{fc} (in)	0.96
Cover plate overhang	$C3$ (in)	3.5
Cover plate thickness	t_{cp} (in)	1
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b(d_c+2[C3])t_{cp}$	V_p (in^3)	1711.2
Column ratio, $(d_c - t_{fc}) / B$	α	0.058
Beam ratio, $(d_b - t_{fb}) / H$	β	0.108
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	27558203
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	143551
Panel zone spring stiffness, $K_{ps}+K_{fs}$	β_{PZ} (k-in)	27701755
Beam moment of inertia	I_{xb} (in^3)	2700
Required panel zone spring stiffness , *$60EI_{xb} / B$ for one-sided, $120EI_{xb} / B$ for two-sided*	K_{req} (k-in)	26100000
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.9 **OK**

DuraFuse Frames® Calculations

Connection ID DF251

W21X122 Column and W24X76 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	E (ksi)	29000
Shear modulus (AISC 360-22)	G (ksi)	11200

Beams

Beam material grade	*Gr.*	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1

Columns

Column material grade	*Gr.*	ASTM A992
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	65
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.1
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.1

Plates

Plate material (AISC 358-22 15.3.3)	*Gr.*	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2

Bolts

Bolt grade (AISC 358-22 15.5.2(5))	*Gr.*	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks

Beam size		W24X76	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40	
Beam depth	d_b (in.)	23.9	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	232	
Beam weight	W_b (lb/ft)	76	**OK**
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6	
Beam web width-to-thickness ratio	λ_{web}	49	**OK**
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), 0.38(E/Fyb)^0.5	$\lambda_{p,flange}$	9.2	
Beam flange width-to-thickness ratio	λ_{flange}	6.61	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks

Column size		W21X122	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36	
Column depth	d_c (in.)	21.7	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	122	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	0.96	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	53.5	
Gross area of the column section	A_g (in^2)	35.9	
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.0	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	57.2	
Column web width-to-thickness ratio	λ_{web}	31.3	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	

Column flange width-to-thickness ratio	λ_{flange}	6.45	**OK**

Column-beam moment ratios (AISC 358 15.4(a))

Column plastic section modulus	Z_c (in^3)	307	
Column design axial load	P_u (kips)	53.5	
Column area	A_{gc} (in^2)	35.9	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	23.9	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma R_{yc} Z_c *(F_{yc} - P_u / A_{gc})*(H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	40494	
Beam plastic section modulus	Z_b (in^3)	200	
Bay width	B (in)	360.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	10000	
Clear distance between column faces, $B-d_c$	L_h (in)	338.3	
Live load factor in seismic load combination	$f1$	0.5	
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	7.0	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	66.1	
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	717	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	21435	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.89	**OK**

*Not applicable for top stories, see AISC 341 E3.4a(a1))

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

Beam moment demand from load combinations (from EOR)	M_u (k-in)	3422	
Beam plastic section modulus	Z_b (in^3)	200	
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	10000	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	10000	**OK**
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1	
Beam depth	d_b (in)	23.9	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	200.8	
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	6999.6	
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b + t_p))$	V_{fe2} (kips)	140.6	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	341.4	

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	341.4	
Beam area	A_{gb} (in^2)	22.4	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	56.0	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	397.4	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	35	
Plate thickness, t_p (see T2 on drawings)	t_p (in)	1	
Beam depth	d_b (in)	23.9	
Column depth	d_c (in)	21.7	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	441.3	

Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)

Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75	
Cover plate overhange dimension	$C3$ (in)	3.5	
Column depth	d_c (in)	21.7	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	28.7	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp} t_{cp} d_{cp}$	$R_{n,horiz}$ (kips)	645.8	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	645.8	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	*D/C*	0.62	**OK**

Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)

Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2{*}t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	29.90	

Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	672.8	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	672.8	
Shear yielding demand/capacity ratio, $R_{u,vert}$ / $\phi R_{n,vert}$	D/C	0.66	**OK**

Cover plate thickness check (AISC 341 E3.6e.2)

Beam depth	d_b (in)	23.9	
Beam flange thickness	t_{fb} (in)	0.68	
Column flange thickness	t_{fc} (in)	0.96	
Panel zone vertical dimension, d_b - $2t_{fb}$	d_z (in)	22.54	
Panel zone horizontal diemension, d_c - $2t_{fc}$	w_z (in)	19.78	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.470	
Thickness check, t_{cp} / t_{req}	t_{cp} / t_{req} > 1.0	1.59	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.125	
Beam plastic section modulus	Z_b (in^3)	200	
Beam flange thickness	t_{fb} (in)	0.68	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	0.850	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	39.5	
Beam plastic section modulus of the net section, Z_b - $Z_{bolt\ holes}$	$Z_{b,net}$	160.5	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	11000	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	11478	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.96	**OK**

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	200.8	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0	
Bolt diameter	d_{bolt} (in)	1	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	0.785	
Bolt shear strength, $\phi F_{nv}A_b$	ϕr_n (kips)	66.0	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	3.9	
Number of bolts per line (AISC 358 15.2(1))	n_b	6	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.375	
Bolt spacing (see schedule)	s (in)	3.25	
Plate thickness, t_p (see T2 in drawings)	t_p (in)	1	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	2.145	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	2.4	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	3	**OK**
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), n_b - n_p	n_m	3	

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	256.8	
Beam flange thickness	t_{fb} (in)	0.68	
Fuse and top plate thickness	$T2$ (in)	1	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	0.7	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}t_{min}F_{up}n_b$	R_{nb} (kips)	636.5	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25	
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, s - d_O	l_{c1} (in)	2.0	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}t_{min}F_{up}(n_b - 1)$	R_{nt1} (kips)	530.4	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, e_d - $d_O/2$	l_{c2} (in)	0.8	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}t_{min}F_{up}$	R_{nt2} (kips)	39.8	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	570.2	
Nominal shear strength of all bolts per line, $F_{nv}A_{bolt}n_b$	R_{nv} (kips)	395.8	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	395.8	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	296.9	
Beam flange bolt shear demand/capacity ratio, $R_u/\phi R_n$	D/C	0.87	**OK**

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1451
Bolt group demand from wind per line, $M_u/(2(d_b+t_p))$	R_u *(kip)*	29.1
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	64
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	6
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	130.2
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	110.6
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.26 **OK**

Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	23.9
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.375
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$*, 1.0 otherwise*	α	0
Cover plate overhang dimension	*C3* (in)	3.5
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	2.809
Distance from alignment line to column	*C1* (in)	6.375 **OK**

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	341.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per weld, $P_{d,total}/2$	P_d (kips)	56
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	35
Column depth	d_c (in)	21.7
Column flange thickness	t_{fc} (in)	0.96
Beam depth	d_b (in)	23.9
Plate thickness (see T2 on drawings)	t_p (in)	1
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2{*}t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	29.9
Gage dimension from the cover plate to the line of bolts on the external continuity plates	*C5* (in)	2.75
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} *(in)*	0.4375
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.03
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.75
Weld 1 size	D_{W1} (1/16 in)	5
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	5
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	19.78
Horizontal distance from center of rotation to geometric center	e_x (in)	0.87
Vertical distance from center of rotation to geometric center	e_y (in)	0.54

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d){*}L$	ΣRL (k-in)	14764

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R{*}L_R$ (k-in)
W1RT	17.82	7.31	55.94	130.2
W4TL	18.54	4.45	34.73	82.5
W1LB	18.02	7.09	49.41	127.8
W4BR	16.68	4.38	31.92	73.0
W1RC	10.01	4.62	4.76	46.2
W4TC	15.28	3.41	4.01	52.1
W1LC	11.75	4.74	4.05	55.7
W4BC	14.20	3.38	4.32	48.0
		7.31	55.94	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	7.3

Orientation of Resultant Force	θ_r (degrees)	55.9	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.58	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	7.8	
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c*l_{We1})$	r_{un} (kip/in)	0.0	
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	7.8	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.29	
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75*0.6*70*(1+0.5*\sin^{1.5}(\theta_r))*(p_i(1.9-0.9p_i))^{0.3}*0.707*D_{W1}/16)$	ϕr_n (kips)	9.44	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.83	**OK**

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	341.4	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0000	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56	
Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang dimension	*C3* (in)	3.5	
External continuity plate dimension	*C5* (in)	2.75	
Weld 2 size	D_{W2} (1/16 in)	7	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.44	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + t_{cp} + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	4.77	
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe}+P_d) / (d_c+2[C3])$	r_{uv2} (kip/in)	13.8	
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe} + P_d)[C5] / (l_{we2,eff}(d_c + 2[C3] - 2l_{hb2} - l_{we2,eff}))$	r_{un2} (kip/in)	9.9	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	17.0	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2\ sides)$	ϕr_n (kip/in)	19.5	
External continuity plate-to-cover plate weld tearing check, $r_u / \phi r_n$	D/C	0.87	**OK**

Failure of external continiuty plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	341.4	
External continiuty plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	1092.7	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	397.4	
External continuity plate thickness (see step 2)	t_p (in)	1.000	
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c+2[C3])T2$	ϕR_n (kips)	861.0	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c+2*[C3])/2)^2$	ϕM_n (kips)	9266.5	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	D/C	0.58	**OK**

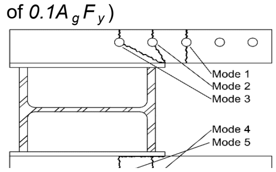
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

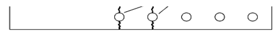
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	341.4	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0	
Column fillet dimension	*k* (in)	1.5	
Column fillet dimension	*k1* (in)	1.5	
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3]-2*l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	93.2	
Column flange width	b_{fc} (in)	12.4	
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	43.6	
Weld 2 size	D_{W2} (1/16 in)	7	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	1210.9	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	566.4	
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl} + R_{nwt})$ or $(0.85R_{nwl} + 1.5R_{nwt})$	R_n (kips)	1878.9	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1409.2	
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	D/C	0.28	**OK**

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC *Specification* Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	3.0
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	128.4
External continuity plate thickness (see step 2)	t_p (in)	1
External continuity plate width	*C4* (in)	4.5





Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Net area of external continuity plate cross section, $t_p([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	3.313	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	215.3	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	161.5	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.80	**OK**

Mode 2: Rupture through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	171.2	
External continuity plate dimension	*C2 (in)*	-1.625	
Cover plate overhang dimension	*C3* (in)	3.5	
External continuity plate dimension	*C5* (in)	2.75	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	1.875	
External continuity plate thickness (see step 2)	t_p (in)	1	
External continuity plate width	*C4* (in)	4.5	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	*Bonus* (in)	0.320	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	3.632	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	236.1	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	177.1	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.97	**OK**

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	214.0	
Bolt spacing (see schedule)	*s* (in)	3.25	
External continuity plate dimension	*C2 (in)*	-1.625	
Cover plate overhang dimension	*C3* (in)	3.5	
External continuity plate dimension	*C5* (in)	2.75	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5.125	
External continuity plate thickness (see step 2)	t_p (in)	1	
External continuity plate width	*C4* (in)	4.5	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	*Bonus* (in)	2.388	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Area of external continuity plate cross section with bonus, $(t_p)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	5.700	
Nominal tensile rupture capacity, A_nF_{up}	P_n (kips)	370.5	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	277.9	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.77	**OK**

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	171.2	
External continuity plate dimension	*C2 (in)*	-1.625	
Cover plate overhang dimension	*C3* (in)	3.5	
External continuity plate width	*C4* (in)	4.5	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	1.875	
External continuity plate thickness (see step 2)	t_p (in)	1	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Gross tension area, $t_p[C4$	A_{gt} (in^2)	4.500	
Gross shear area, $t_p[C9]$	A_{gv} (in^2)	1.875	
Net tension area, $A_{gt} - ((t_p)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	3.313	
Net shear area, A_{gv}	A_{nv} (in^2)	1.875	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	271.6	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	203.7	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.84	**OK**

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	214.0	
Bolt spacing (see schedule)	*s* (in)	3.25	
External continuity plate dimension	*C2 (in)*	-1.625	
Cover plate overhang dimension	*C3* (in)	3.5	
External continuity plate width	*C4* (in)	4.5	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5.125	
External continuity plate thickness (see step 2)	t_p *(in)*	1	

Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125
Gross tension area, t_p *[C4]*	A_{gt} (in^2)	4.500
Gross shear area, t_p *[C7]*	A_{gv} (in^2)	5.125
Net tension area, $A_{gt}-((t_p)(d_{hole}+1/16))$	A_{nt} (in^2)	3.313
Net shear area, A_{gv}	A_{nv} (in^2)	5.125
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	369.1
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	276.8
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.77 **OK**

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Bay width	B (in)	360.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	7.0
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	10000
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	66.1
Beam depth	d_b (in)	23.9
Beam web thickness	t_{wb} (in)	0.44
Beam web area, $d_b t_{wb}$	A_w (in)	10.516
Beam web width-to-thickness ratio, h / t_w	l_{web}	49
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	315.5
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	*D/C*	0.21 **OK**

*Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2*
C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	56.0
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	513.6
Beam flange thickness	t_{fb} (in)	0.68
Beam flange width	b_{fb} (in)	8.99
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.125
Number of holes along each bolt line (see step 5)	n_b	6
Length of connection on beam	*B4* (in)	17.625
Distance from beam centerline to bolt line	*P2* (in)	2.75
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	2.373
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	23.970
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	1.566
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	15.088
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	690.2
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	517.6
Demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.99 **OK**

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	66.1
Bolt diameter (see step 4)	d_{bolt} (in)	1
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	0.785
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	66.0
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	49.5
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	1
Provided number of shear tab bolts	n_n	4 **OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - 1in	$l_{tab,req}$ (in)	19
Length of shear tab (see drawings), T_{beam} - 1in	l_{tab} (in)	19 **OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.44
Shear tab thickness	*T3* (in)	0.5

Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	0.4
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\ t_{min}\ F_{up}\ n_n$	R_{nb} (kips)	274.6
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.125
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.250
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	5.500
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	4.4
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{up}\ (n_n\ -1)$	R_{nt1} (kips)	450.5
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d\ - d_{hole,std}\ / 2$	l_{c2} (in)	0.7
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{up}$	R_{nt2} (kips)	23.6
Total tearout strength, $R_{nt1}\ + R_{nt2}$	R_{nt} (kips)	474.0
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	263.9
Controlling strength, $min(R_{nb}, R_{nt,}\ R_{nv})$	R_n (kips)	263.9
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	197.9
Shear tab bolt shear demand/capacity ratio, $R_u\ /\ \phi R_n$	D/C	0.33 **OK**

Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	64
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	32.5
Provided number of shear tab bolts (see step 12)	n_n	4
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	130.2
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	66.1
Distance from alignment line to column	$C1$ (in)	6.375
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	421.5

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1
Slotted bolt hole width (standard slot width)	w_b (in)	1.125
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.5
Shear tab length (see step 12)	l_{tab} (in)	19
Beam T dimension limit	T_{beam} (in)	20
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.25
Slotted bolt hole spacing, $(T_{beam}\ - 1"\ - 2e_d)\ /\ (n_n\ - 1)$	s_{slot} (in)	5.375
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50
Shear tab net area, $A_{gv} - n_n(w_b + 1/16)t_{tab}$	A_{nv} (in^2)	7.13
Effective net area, $A_e\ = A_{nv}$	A_e (in^2)	7.13
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up}A_{nv}$	ϕR_{nv} (kips)	208.41
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up}A_e$	ϕR_{nn} (kips)	347.34
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{nn})^2 + (V_u/\phi\ R_{nv})^4$	D/C	0.15 **OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	66.12
Required axial strength of the shear tab (see above)	P_u (kips)	130.18
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	421.51
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in^2)	9.50
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in^3)	45.13
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp}A_{gv}$	ϕR_{nv} (kips)	285.00
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp}A_{gv}$	ϕR_{nn} (kips)	427.50
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	2030.63
Shear tab yielding demand/cacpacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nn})^2\ + (V_u/\phi R_{nv})^4$	D/C	0.30 **OK**

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b\ - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	21.9
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	66.1
Weld 3 shear demand, $V_u\ /\ l_{tab}$	r_{uv} (k/in)	3.0
Number of slotted bolt holes in shear tab	n_{vb}	4

Slotted bolt hole spacing	s_{slot} (in)	5.375
Distance from line of bolts to shear tab weld	$C1$ (in)	6.375
Size of weld in 1/16^{ths} of an inch	D_{W3}	3
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	421.5
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	5.9
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	6.7
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W3}/16(1")(2\ sides)$	ϕr_n (kips)	8.4
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.80 **OK**

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.375
Beam depth	d_b (in)	23.9
Length of the shear tab (see step 12)	l_{tab} (in)	19
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.7045
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.75 **OK**
Provided outside shear tab slot dimension, *[S2] = [S1]*	$S2$ (in)	1.75

Step 15. Check the top plate for shear yielding and shear rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	256.8

Yielding check

Top plate thickness (see step 2)	t_p (in)	1
Top plate dimension (see schedule)	$P6$ (in)	19.5
Gross shear area of top plate, $(P6)(t_p)$	A_g (in^2)	19.500
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	585.0
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	585.0
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.44 **OK**

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	1
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.3125
Number of short slotted bolts on top plate (see step 5)	n_b	6
Net shear area of top plate, $A_g-(t_p l_s n_b)$	A_n (in^2)	11.625
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{up}$	R_n (kips)	453.4
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	340.0
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.76 **OK**

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0
Total number of bolts (see step 5)	n_b	6
Number of bolts in the M region (see step 5)	n_m	3
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	128.4

Yielding check

Top plate dimension	$P4$ (in)	1.5
Top plate dimension	$P5$ (in)	2.5
Top plate thickness (see step 2)	t_p (in)	1
Gross area, $([P4]+[P5])*(t_p)$	A_g (in^2)	4.000
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	200.0
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	180.0
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.71 **OK**

Rupture check

Top plate dimension	$P4$ (in)	1.5

Top plate dimension	$P5$ (in)	2.5	
Top plate thickness (see step 2)	t_p (in)	1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25	
Effective area of rupture, $([P4]+[P5]-d_O-1/16)(t_p)$	A_e (in^2)	2.688	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	174.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	131.0	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.98	**OK**

Step 17

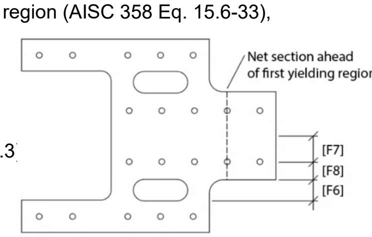
Entering and tightening check			
Beam flange fillet dimension	k_1	1.4	
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.25	
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	2.6875	
Provided P2 dimension	$P2$ (in)	2.75	**OK**

Top plate yielding due to combined flexure and shear check			
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	56.0	
Top plate thickness (see step 2)	t_p (in)	1	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	9.7	
Inside length of the top plate	$P8$ (in)	19.50	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp}t_p)$	m (in)	9.511	
Dimension, $([P8] - m) / 2$	e (in)	4.994	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-3.0	
Provided P2 dimension	$P2$ (in)	2.75	**OK**

Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC *Specification* Chapter D)

Fuse plate yielding region proportions			
Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4	
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5	
Provided width of yielding region	$F6$ (in)	2.125	**OK**
Top plate thickness (see step 2)	t_p (in)	1	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/t_p$	2.1	**OK**

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8	
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0	
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0	
Number of bolts in the B region (see step 5)	n_b	6	
Number of bolts in the M region (see step 5)	n_m	3	
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	200.8	
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.75	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	3.325	
Fuse plate thickness (see step 2)	t_p (in)	1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25	
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in^2)	12.150	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	607.5	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	546.8	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.37	**OK**



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Fuse plate dimension	$F7$ (in)	2.75	
Fuse plate dimension	$F8$ (in)	3.325	
Fuse plate thickness (see step 2)	t_p (in)	1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.250	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)t_p$	A_e (in^2)	9.525	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e{}^a$	P_n (kips)	743.0	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	557.2	

Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.36 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Step 19. Determine the yielding region depth [F2]

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	200.8
Fuse plate thickness(see step 2)	t_p (in)	1
Width of fuse plate cut-out	$F6$ (in)	2.125
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6]))/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	1.790
Provided yielding region depth	$F2$ (in)	1.750 **OK**
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	105.0
Ratio	V_{fe} / V_y	1.9

Fuse plate yielding region geometry		
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, 0.5 $<= [F6] / [F2] <= 1.25$	$F6 / F2$	1.21 **OK**

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	n_b	6
Number of bolts in the M region	n_m	3
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	200.8

Yielding check		
Fuse plate dimension	$F4$ (in)	1.5
Fuse plate dimension	$F5$ (in)	1.625
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in^2)	6.25
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	312.50
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	281.25
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.71 **OK**

Rupture check		
Fuse plate dimension	$F4$ (in)	1.5
Fuse plate dimension	$F5$ (in)	1.625
Fuse plate thickness (see step 2)	t_p (in)	1
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(t_p)$	A_e (in^2)	3.625
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e$ [a]	R_n (kips)	282.8
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	212.1
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.95 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

Bridge plate tension failure check		
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	200.8
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	112.0
Drag force per line of bolts, $P_d / 2$	P_d (kips)	56
External continuity plate dimension	$C5$ (in)	2.75
Column depth	d_c (in)	21.7
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	28.0

Cover plate overhang	$C3$ (in)	3.5	
Bridge plate thickness (see T2 on drawings, not greater than 2W5)	t_p (in)	0.625	
Bridge plate factored tensile capacity, $0.9*F_{yp}\,t_p\,([C3]\text{-}1")$	ϕP_n (kips)	70.3	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	D/C	0.40	**OK**

Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	28.0	
Weld 5 size	D_{W5} (1/16 in)	5	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16\,([C3]\text{-}1")(2\ sides)$	ϕR_n (kips)	52.2	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.54	**OK**

Shear tab to bridge plate weld failure check

Bay width	B (in)	360.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	23.9	
Shear force from gravity	$V_{gravity}$ (kips)	7.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	10000	
Total vertical force at alignment line, $2M_{pr}/(B\text{-}d_c) + V_{gravity}$	V_u (kips)	66.1	
Distance from alignment line to column	$C1$ (in)	6.375	
Weld 7 demand, $V_u\,C1/d_b$	R_u (kips)	17.6	
Weld 7 size	D_{W7} (1/16 in)	3	
Cover plate overhang	$C3$ (in)	3.5	
Weld 7 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(FEXX)(0.707)D_w/16[C3](2\ sides)$	ϕR_n (kips)	29.2	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.60	**OK**

Bridge plate to column flange weld failure check

Total vertical force at alignment line, $2M_{pr}/(B\text{-}d_c) + V_{gravity}$	V_u (kips)	66.1	
Distance from alignment line to column	$C1$ (in)	6.375	
Weld 6 demand, $V_u\,[C1]/d_b$	R_u (kips)	17.6	
Weld 6 size	D_{W6} (1/16 in)	4	
Cover plate overhang	$C3$ (in)	3.5	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	7	
Weld 6 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w6}/16(l_{eff})$	ϕR_n (kips)	58.5	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	0.3	**OK**

Shear plate shear failure check (when W1 > 3/4" without shear plates, shear plates are used)

Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Weld 4 size	D_{W4} (1/16 in)	5	
Weld 4 effective length, $d_c - 2\,t_{fc}$	l_{eff} (in)	19.78	
Shear plate demand based on shear plate design weld capacity $(0.75)(0.6)(F_{EXX})(0.707)D_{W4}/16\ (leff)$	R_u (kips)	137.7	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.375	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{yp}\,t_{sp}\,(d_c\text{-}2t_{fc})$	ϕR_n (kips)	222.5	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6	**OK**

Rigid panel zone check

The check is presented since panel zones were assumed rigid in the analysis model. Guidance for panel zone modeling comes from AISC 7-41 9.4.2.2.1.3 which states, "where the expected shear strength of the panel zone exceeds the flexural shear strengths of the beams at a beam-to-column connection, and the stiffness of the panel zone (converted to a rotational spring) is at least 10 times larger than the flexural stiffness of the beam, direct modeling of the panel zone shall not be required. In such cases, rigid offsets from the center of the column shall be permitted to represent the effective span of the beam."

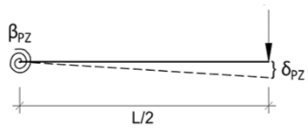
The derivation below shows that the panel zone spring stiffness needs to exceed 60EI/L in order for the stiffness of the panel zone (converted to a rotational spring) to be at least 10 times larger than the flexural stiffness of the beam.

Derivation

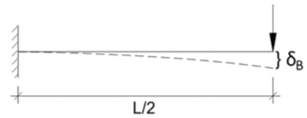
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness   P   Beam Stiffness   P

$K_{PZ} = P/\delta_{PZ}$
$\delta_{PZ} = P(L/2)^2/\beta_{PZ}$
$K_{PZ} = \beta_{PZ}/(L/2)^2$



$K_B = P/\delta_B$
$\delta_B = P(L/2)^3/3EI$
$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$
$\beta_{PZ}/(L/2)^2 \geq 10{*}3EI/(L/2)^3$
---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	23.9	
Beam flange thickness	t_{fb} (in)	0.68	
Column depth	d_c (in)	21.7	
Column flange width	b_{fc} (in)	12.4	
Column web thickness	t_{wc} (in)	0.6	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang	*C3* (in)	3.5	
Cover plate thickness	t_{cp} (in)	0.75	
Bay width	*B* (in)	360.0	
Story height	*H* (in)	216.0	
Panel zone factor, $d_b d_c t_{wc} + 2d_b(d_c+2[C3])t_{cp}$	V_p (in^3)	1340.1	
Column ratio, $(d_c - t_{fc})/B$	α	0.058	
Beam ratio, $(d_b - t_{fb})/H$	β	0.108	
Web and cover plate stiffness, $GV_p/(1-\alpha-\beta)^2$	K_{ps} (k-in)	21532235	
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2)/(1-\alpha-\beta)^2$	K_{fs} (k-in)	143225	
Panel zone spring stiffness, $K_{ps}+K_{fs}$	β_{PZ} (k-in)	21675460	
Beam moment of inertia	I_{xb} (in^3)	2100	
Required panel zone spring stiffness , *$60EI_{xb}/B$ for one-sided, $120EI_{xb}/B$ for two-sided*	K_{req} (k-in)	20299999	
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.9	**OK**

HCAI Calculations

DuraFuse Frames® Calculations

Connection ID DHF101

HSS10X10X3/4 Column and W14X38 Beam

Material properties

Modulus of elasticity (AISC 360-22 Table B4.1b)	E (ksi)	29000
Shear modulus (AISC 360-22)	G (ksi)	11200

Beams		
Beam material grade	*Gr.*	ASTM A992
Beam specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yb} (ksi)	50
Beam specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{ub} (ksi)	65
Beam ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yb}	1.1
Beam ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tb}	1.1

Columns		
Column material grade	*Gr.*	ASTM A500 Gr.C
Column specified minimum yield stress (2016 AISC Manual Table 2-4)	F_{yc} (ksi)	50
Column specified minimum tensile stress (2016 AISC Manual Table 2-4)	F_{uc} (ksi)	62
Column ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yc}	1.3
Column ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tc}	1.2

Plates		
Plate material (AISC 358-22 15.3.3)	*Gr.*	A572 Gr. 50
Plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yp} (ksi)	50
Plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{up} (ksi)	65
Plate ratio of expected yield stress to the specified minimum yield stress (AISC 341 Table A3.1)	R_{yp}	1.1
Plate ratio of expected tensile stress to the specified minimum tensile stress (AISC 341 Table A3.1)	R_{tp}	1.2

Bolts		
Bolt grade (AISC 358-22 15.5.2(5))	*Gr.*	F2280X
Bolt nominal tensile strength (AISC 360 Table J3.2)	F_{nt} (ksi)	113
Bolt nominal shear strength (AISC 360, Table J3.2)	F_{nv} (ksi)	84

Step 1. Check beam and column limitations. Confirm column-beam moment ratio is satisfied

Beam limitation checks			
Beam size		W14X38	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	40	
Beam depth	d_b (in.)	14.1	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	232	
Beam weight	W_b (lb/ft)	38	**OK**
Beam web width-to-thickness ratio limit (AISC 358 15.3.1(4), AISC 360-22 Table B4.1b), $3.76(E/F_{yb})^{0.5}$	$\lambda_{p,web}$	90.6	
Beam web width-to-thickness ratio	λ_{web}	39.6	**OK**
Beam flange width-to-thickness ratio limit (HCAI design guide 1.d), 0.38(E/Fyb)^0.5	$\lambda_{p,flange}$	9.2	
Beam flange width-to-thickness ratio	λ_{flange}	6.57	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		HSS10X10X3/4	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	36	
Column depth	d_c (in.)	10	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	89.5	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	0.698	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	123.5	
Gross area of the column section	A_g (in^2)	24.7	
Threshold variable for checking controlling with-to-thickness ratio	C_a (kips)	0.1	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1)	$\lambda_{hd,web}$	52.8	
Column web width-to-thickness ratio	λ_{web}	11.3	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), For wide flanges $0.32(E / (R_{yc}F_{yc}))^{0.5}$, for HSS $0.65(E / (R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	13.73	

Column flange width-to-thickness ratio	λ_{flange}	11.3 **OK**

Column-beam moment ratios (AISC 358 15.4(a))		
Column plastic section modulus	Z_c (in^3)	84.7
Column design axial load	P_u (kips)	123.5
Column area	A_{gc} (in^2)	24.7
Story Height	H (in)	180.0
Column depth	d_c (in)	10
Beam depth	d_b (in)	14.1
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma R_{yc} Z_c*(F_{yc}-P_u/A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	9842
Beam plastic section modulus	Z_b (in^3)	61.5
Bay width	B (in)	240.0
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	3075
Clear distance between column faces, B-d_c	L_h (in)	230.0
Live load factor in seismic load combination	$f1$	0.5
Beam shear caused by gravity loads in seismic load combination (AISC 358 15.4a), $1.2D + f1L + 0.2S$	V_g (kips)	11.0
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	37.7
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	189
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	3264
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	3.02 **OK**

*Not applicable for top stories, see AISC 341 E3.4a(a1))

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

Beam moment demand from load combinations (from EOR)	M_u (k-in)	0
Beam plastic section modulus	Z_b (in^3)	61.5
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	3075
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	3075 **OK**
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.625
Beam depth	d_b (in)	14.1
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + t_p))$	V_{fe} (kips)	104.4
In-plane cantilever beam moment (from EOR) (only present when in-plane gravity cantilever uses DFF)	$M_{cantilever}$ (k-in)	0.0
Maximum probable force per bolt line from second beam (only present for 2-sided DFF), V_{fe} or $M_{cantilever} / (2(d_b+t_p))$	V_{fe2} (kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	104.4

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	104.4
Beam area	A_{gb} (in^2)	11.2
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	42.8
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	147.2
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	10
Plate thickness, t_p (see T2 on drawings)	t_p (in)	0.625
Beam depth	d_b (in)	14.1
Column depth	d_c (in)	10
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + t_p) / d_c + P_d(d_b + t_p) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	190.2

Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)		
Cover plate thickness (see T1 in drawings)	t_{cp} (in)	0.75
Cover plate overhange dimension	$C3$ (in)	2
Column depth	d_c (in)	10
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	14
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}d_{cp}$	$R_{n,horiz}$ (kips)	315.0
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	315.0
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	*D/C*	0.47 **OK**

Cover plate shear yielding limit state, vertical shear (AISC 360 Chapter J)		
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	20.10

Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yp}t_{cp}h_{cp}$	$R_{n,vert}$ (kips)	452.3	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	452.3	
Shear yielding demand/capacity ratio, $R_{u,vert}$ / $\phi R_{n,vert}$	D/C	0.42	**OK**

Cover plate thickness check (AISC 341 E3.6e.2)

Beam depth	d_b (in)	14.1	
Beam flange thickness	t_{fb} (in)	0.515	
Column flange thickness	t_{fc} (in)	0.698	
Panel zone vertical dimension, d_b - $2t_{fb}$	d_z (in)	13.07	
Panel zone horizontal diemension, d_c - $2t_{fc}$	w_z (in)	8.604	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.241	
Thickness check, t_{cp} / t_{req}	t_{cp} / t_{req} > 1.0	3.11	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	0.75	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	0.8125	
Beam plastic section modulus	Z_b (in^3)	61.5	
Beam flange thickness	t_{fb} (in)	0.515	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	0.483	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	13.1	
Beam plastic section modulus of the net section, Z_b - $Z_{bolt\ holes}$	$Z_{b,net}$	48.4	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	3383	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	3459	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.98	**OK**

Step 5. Determine the number of bolts for each bolt line on the fuse plate and top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	104.4	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.8	
Bolt diameter	d_{bolt} (in)	0.75	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	0.442	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	37.1	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	4.0	
Number of bolts per line (AISC 358 15.2(1))	n_b	6	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1.0625	
Bolt spacing (see schedule)	s (in)	2.75	
Plate thickness, t_p (see T2 in drawings)	t_p (in)	0.625	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{up}R_{tp}t_p)]$	$F2_a$ (in)	1.785	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e_d]/s + 1$	$n_{p,min}$	2.6	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	3	**OK**
Number of bolts behind the alignment line on the outside lines (zone M) (AISC 358 Eq. 15.6-8), n_b - n_p	n_m	3	

Flange bolt limit states check

Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	147.2	
Beam flange thickness	t_{fb} (in)	0.515	
Fuse and top plate thickness	$T2$ (in)	0.625	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	0.5	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{up} n_b$	R_{nb} (kips)	361.5	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	0.9375	
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s) l_{c1} is based on an oversized hole, s - d_O	l_{c1} (in)	1.8	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{up} (n_b -1)$	R_{nt1} (kips)	364.0	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material l_{c2} is based on an oversized hole, e_d - d_O / 2	l_{c2} (in)	0.6	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{up}$	R_{nt2} (kips)	23.9	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	387.9	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	222.7	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	222.7	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	167.0	
Beam flange bolt shear demand/capacity ratio, R_u / ϕR_n	D/C	0.88	**OK**

Bolt slip for wind load check

Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	600
Bolt group demand from wind per line, $M_u/(2(d_b+t_p))$	R_u (kip)	20.4
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	35
Mean slip coefficient	μ	0.3
Ratio of installed bolt pretension to specified bolt pretension	D_u	1.13
Filler factor	h_f	1
Number of slip planes	n_s	1
Number of bolts per line	n_b	6
Resistance factor for bolt slip	ϕ	0.85
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_b$	R_n (kips)	71.2
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	60.5
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.34 **OK**

Step 6. Locate the alignment line to accommodate beam rotation

Beam design rotation (AISC 358 Eq. 15.6-9)	γ_{max} (rad)	0.06
Beam depth	d_b (in)	14.1
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e_d (in)	1.0625
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$, *1.0 otherwise*	α	0
Cover plate overhang dimension	*C3* (in)	2
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e_d + \alpha[C3]$	$C1_{req}$ (in)	1.9085
Distance from alignment line to column	*C1* (in)	5.125 **OK**

Step 7. Size weld 1

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per weld, $P_{d,total}/2$	P_d (kips)	42.75
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	10
Column depth	d_c (in)	10
Column flange thickness	t_{fc} (in)	0.698
Beam depth	d_b (in)	14.1
Plate thickness (see T2 on drawings)	t_p (in)	0.625
Cover plate thickness (see T1 on drawings)	t_{cp} (in)	0.75
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - t_p$ or just $d_b - 2*t_p$ for cap plate and ortho. cant./drag conditi	h_{cp} (in)	20.1
Gage dimension from the cover plate to the line of bolts on the external continuity plates	*C5* (in)	2.375
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.5000
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	19.10
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $t_p + t_{cp} + [C6]$	l_{We1} (in)	4.38
Weld 1 size	D_{W1} (1/16 in)	6
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	0
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	8.604
Horizontal distance from center of rotation to geometric center	e_x (in)	0.87
Vertical distance from center of rotation to geometric center	e_y (in)	0.54

The Instantaneous Center of Rotation method was used to design Weld 1 and Weld 4. The method is used to sum the moment capacity of weld segments about the Center of Rotation of the weld group. If there are bridge plates or shear plates on the connection then the orthogonal normal demand on Weld 1 (r_{un}) is zero.

Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	3798

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	10.73	6.09	67.36	65.3
W4TL	11.50	0.00	25.97	0.0
W1LB	10.59	5.86	56.35	62.1
W4BR	9.83	0.00	19.58	0.0
W1RC	4.19	3.48	10.04	14.6
W4TC	10.38	0.00	5.28	0.0
W1LC	5.92	3.66	7.10	21.6
W4BC	9.31	0.00	5.90	0.0
		6.09	67.36	

Weld strength check according to AISC 360 Chapter J		
Force on Critical Weld Segment	R_{crit} (kips)	6.1

Orientation of Resultant Force	θ_r (degrees)	67.4	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.38	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}(d_b+t_p)+P_d(d_b+t_p)/2)/\Sigma RL)$	r_{uv} (kip/in)	7.8	
Orthogonal normal demand on weld (when bridge plates are not present), $\Sigma(V_{fe}+P_d)[C5]/(d_c*l_{We1})$	r_{un} (kip/in)	8.0	
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	11.1	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.24	
Factored weld capacity (AISC Manual Eq. 8-3), $(0.75*0.6*70*(1+0.5*\sin^{1.5}(\theta_r))*(p_i(1.9-0.9p_i))^{0.3}*0.707*D_{W1}/16)$	ϕr_n (kips)	11.95	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	*D/C*	0.93	**OK**

Step 8. Size the external continuity plate-to-cover plate (W2)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	104.4	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5000	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.75	
Column depth	d_c (in)	10	
Column flange thickness	t_{fc} (in)	0.698	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate dimension	*C5* (in)	2.375	
Weld 2 size	D_{W2} (1/16 in)	6	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.38	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc}+t_{cp}+[C3]-l_{hb2}$	$l_{we2,eff}$ (in)	3.07	
Longitudinal shear demand on weld 2 (AISC 358 Eq. 15.6-15), $(\Sigma V_{fe}+P_d)/(d_c+2[C3])$	r_{uv2} (kip/in)	10.5	
Normal demand on weld 2 (AISC 358 Eq. 15.6-16), $(\Sigma V_{fe}+P_d)[C5]/(l_{we2,eff}(d_c+2[C3]-2l_{hb2}-l_{we2,eff}))$	r_{un2} (kip/in)	11.2	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	15.3	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W2}/16(1")(2\ sides)$	ϕr_n (kip/in)	16.7	
External continuity plate-to-cover plate weld tearing check, $r_u / \phi r_n$	*D/C*	0.92	**OK**

Failure of external continiuty plate base material check

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	104.4	
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	349.5	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	147.2	
External continuity plate thickness (see step 2)	t_p (in)	0.625	
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yp}(d_c+2[C3])T2$	ϕR_n (kips)	262.5	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yp}t_p((d_c+2*[C3])/2)^2$	ϕM_n (kips)	1378.1	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.81	**OK**

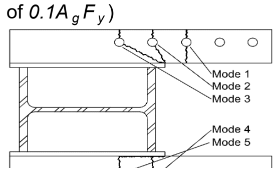
Weld check for alternative cap plate detail (7/SDF-02 or 7/SDF-03)

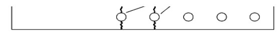
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	104.4	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.8	
Column fillet dimension	*k* (in)	1.0	
Column fillet dimension	*k1* (in)	0.7	
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	42.3	
Column flange width	b_{fc} (in)	10	
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	37.216	
Weld 2 size	D_{W2} (1/16 in)	6	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	471.2	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	414.4	
Nominal strength of weld group (AISC 360, Eq. J2-6), greater of $(R_{nwl}+R_{nwt})$ or $(0.85R_{nwl}+1.5R_{nwt})$	R_n (kips)	1022.2	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	766.6	
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	*D/C*	0.19	**OK**

Step 9. Check external continuity plate for tensile rupture in the net section limit states (AISC *Specification* Chapter D)

Mode 1: Rupture through the bolt hole on the alignment line

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	104.4	
Total drag force transmitted through the connection (from EOR or min of $0.1A_gF_y$)	$P_{d,total}$ (kips)	85.5	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.8	
Number of bolts ahead of rupture, $n_b - n_m$	n_{rup}	3.0	
Mode 1 bolt rupture demand, $(V_{fe}+P_d)n_{rup}/n_b$	P_u (kips)	73.6	
External continuity plate thickness (see step 2)	t_p (in)	0.625	
External continuity plate width	*C4* (in)	4.125	





Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Net area of external continuity plate cross section, $t_p([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	2.031	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	132.0	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	99.0	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.74	**OK**

Mode 2: Rupture through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	98.1	
External continuity plate dimension	*C2 (in)*	-1.5	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate dimension	*C5* (in)	2.375	
External continuity plate dimension, *[C2]* + *[C3]*	*C9* (in)	0.5	
External continuity plate thickness (see step 2)	t_p (in)	0.625	
External continuity plate width	*C4* (in)	4.125	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(t_p)/4[C5]$	*Bonus* (in)	0.016	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Area of external continuity plate cross section with bonus, $t_p([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	2.048	
Nominal tensile rupture capacity, $F_{up}A_n$	P_n (kips)	133.1	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	99.8	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.98	**OK**

Mode 3: Rupture through second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	122.6	
Bolt spacing (see schedule)	*s* (in)	2.75	
External continuity plate dimension	*C2 (in)*	-1.5	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate dimension	*C5* (in)	2.375	
External continuity plate dimension, *[C2]* + *[C3]* + *s*	*C7* (in)	3.25	
External continuity plate thickness (see step 2)	t_p (in)	0.625	
External continuity plate width	*C4* (in)	4.125	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(t_p)/(4[C5])$	*Bonus* (in)	0.695	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Area of external continuity plate cross section with bonus, $(t_p)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	2.726	
Nominal tensile rupture capacity, A_nF_{up}	P_n (kips)	177.2	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	132.9	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.92	**OK**

Mode 4: Block shear through the first bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 1$	n_{rup}	4	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	98.1	
External continuity plate dimension	*C2 (in)*	-1.5	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate width	*C4* (in)	4.125	
External continuity plate dimension, *[C2]* + *[C3]*	*C9* (in)	0.5	
External continuity plate thickness (see step 2)	t_p (in)	0.625	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Gross tension area, $t_p[C4$	A_{gt} (in^2)	2.578	
Gross shear area, $t_p[C9]$	A_{gv} (in^2)	0.313	
Net tension area, $A_{gt} - ((t_p)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	2.031	
Net shear area, A_{gv}	A_{nv} (in^2)	0.313	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	141.4	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	106.1	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.93	**OK**

Mode 5: Block shear through the second bolt hole in the direction of the column

Number of bolts ahead of rupture, $n_b - n_m + 2$	n_{rup}	5	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	122.6	
Bolt spacing (see schedule)	*s* (in)	2.75	
External continuity plate dimension	*C2 (in)*	-1.5	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate width	*C4* (in)	4.125	
External continuity plate dimension, *[C2]* + *[C3]* + *s*	*C7* (in)	3.25	
External continuity plate thickness (see step 2)	t_p *(in)*	0.625	

Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125
Gross tension area, t_p *[C4]*	A_{gt} (in^2)	2.578
Gross shear area, t_p *[C7]*	A_{gv} (in^2)	2.031
Net tension area, $A_{gt}-((t_p)(d_{hole}+1/16))$	A_{nt} (in^2)	2.031
Net shear area, A_{gv}	A_{nv} (in^2)	2.031
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{up}A_{nv} + F_{up}A_{nt} < 0.6F_{yp}A_{gv} + F_{up}A_{nt}$	R_n (kips)	193.0
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	144.7
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.85 **OK**

Step 10. Determine the required shear strength for the beam and shear tab, and check beam web capacity (AISC 360 Chapter G)

Bay width	B (in)	240.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	11.0
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	3075
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	37.7
Beam depth	d_b (in)	14.1
Beam web thickness	t_{wb} (in)	0.31
Beam web area, $d_b t_{wb}$	A_w (in)	4.371
Beam web width-to-thickness ratio, h / t_w	l_{web}	39.6
Beam web shear buckling coefficient (AISC 360 G2.1)	k_v	5.34
Beam web shear strength coefficient (AISC 360 Eq. G2-2, 2-3, or 2-4, as applicable)	C_v	1
Beam factored shear strength (AISC 360 Eq. G2-1), $(1.0)(0.6)F_{yb}A_w C_v$	ϕV_n (kips)	131.1
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	*D/C*	0.29 **OK**

*Note: $V_{gravity}$ is typically provided by EOR. When $V_{gravity}$ is not provided, it is conservatively assumed to equal 3.6 kips/ft * Bay Width/2*
C_{v1} is calculated according to AISC Section G2.1

Step 11. Check the beam flange for block shear (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.8
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	294.3
Beam flange thickness	t_{fb} (in)	0.515
Beam flange width	b_{fb} (in)	6.77
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	0.8125
Number of holes along each bolt line (see step 5)	n_b	6
Length of connection on beam	*B4* (in)	14.8125
Distance from beam centerline to bolt line	*P2* (in)	2.375
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	1.040
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	15.257
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	0.590
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	10.300
Nominal rupture capacity, (AISC 360 Eq. J4-5), $0.6F_{ub}A_{nv} + F_{ub}A_{nt} < 0.6F_{yb}A_{gv} + F_{ub}A_{nt}$	R_n (kips)	440.0
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	330.0
Demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.89 **OK**

Step 12. Determine the required number of shear tab bolts for bolt shear

The required strength of shear tab bolt group (see step 10)	R_u (kips)	37.7
Bolt diameter (see step 4)	d_{bolt} (in)	0.75
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	0.442
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	37.1
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	27.8
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	1
Provided number of shear tab bolts	n_n	3 **OK**

Shear tab geometry requirements		
Beam T dimension limit	T_{beam} (in)	11.625
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - 1in	$l_{tab,req}$ (in)	10.625
Length of shear tab (see drawings), T_{beam} - 1in	l_{tab} (in)	10.625 **OK**

Web bolt limit states check		
Beam web thickness	t_{wb} (in)	0.31
Shear tab thickness	*T3* (in)	0.375

Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	0.3
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}\, t_{min}\, F_{up}\, n_n$	R_{nb} (kips)	108.8
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	0.8125
Edge distance on shear tab (AISC 360 Table J3.4)	e_d (in)	1.000
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	4.313
Clear distance, parallel to applied force, between the edge of the hole and the edge of adjacent hole(s), $s - d_{hole,std}$	l_{c1} (in)	3.5
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\, t_{min}\, F_{up}\, (n_n - 1)$	R_{nt1} (kips)	169.3
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e_d - d_{hole,std} / 2$	l_{c2} (in)	0.6
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\, t_{min}\, F_{up}$	R_{nt2} (kips)	14.4
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	183.6
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	111.3
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	108.8
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	81.6
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.46 **OK**

Step 13. Design the shear tab connection for the required strength (AISC 360)

Minimum fastener tension (AISC 360 Table J3.1)	T_b	35
Mean slip coefficient for Class A or B surfaces (AISC 360 J3.8)	μ	0.3
A multiplier that reflects the ratio of the mean installed bolt pretension to specified minimum bolt pretension (AISC Specification Chapter J3.8)	D_u	1.13
Factor for fillers (AISC 360 Chapter J3.8)	h_f	1
Number of slip planes required to permit the connection to slip	n_s	1
Maximum slip force from a single bolt (AISC 360 Eq. J3-4 with 1.5 multiplier, AISC 358 15.6), $1.5T_b \mu D_u h_f n_s$	F_b (kips)	17.8
Provided number of shear tab bolts (see step 12)	n_n	3
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	53.4
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	37.7
Distance from alignment line to column	$C1$ (in)	5.125
Shear tab required flexural strength (AISC 358 Eq. 15.6-24), $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	193.4

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	0.75
Slotted bolt hole width (standard slot width)	w_b (in)	0.8125
Shear tab provided thickness (see T3 on drawings)	t_{tab} (in)	0.375
Shear tab length (see step 12)	l_{tab} (in)	10.625
Beam T dimension limit	T_{beam} (in)	11.625
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e_d (in)	1
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	4.1875
Gross area of shear tab, $t_{tab} l_{tab}$	A_{gv} (in^2)	3.98
Shear tab net area, $A_{gv} - n_n(w_b + 1/16) t_{tab}$	A_{nv} (in^2)	3.00
Effective net area, $A_e = A_{nv}$	A_e (in^2)	3.00
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{up} A_{nv}$	ϕR_{nv} (kips)	87.75
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{up} A_e$	ϕR_{nn} (kips)	146.25
Shear tab net section fracture demand/capacity ratio (AISC 360 Eq. 9-1), $(P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.17 **OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	37.74
Required axial strength of the shear tab (see above)	P_u (kips)	53.39
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	193.41
Shear tab gross area, $t_{tab} l_{tab}$	A_{gv} (in^2)	3.98
Shear tab plastic section modulus, $(l_{tab}/2)^2 t_{tab}$	Z_{tab} (in^3)	10.58
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yp} A_{gv}$	ϕR_{nv} (kips)	119.53
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yp} A_{gv}$	ϕR_{nn} (kips)	179.30
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_y Z_{tab}$	ϕM_n (kips)	476.26
Shear tab yielding demand/cacpacity ratio, (AISC 360 Eq. 9-1), $(M_u/\phi M_n) + (P_u/\phi R_{nn})^2 + (V_u/\phi R_{nv})^4$	D/C	0.50 **OK**

Shear tab weld failure check (when bridge plates are used, see misc. calculations for additional information)

Weld 3 length, equal to l_{tab} except when bridge plates present $d_b - 2t_p$ (see drawings detail 3/SDF-04)	l_{w3} (in)	10.625
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	37.7
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	3.6
Number of slotted bolt holes in shear tab	n_{vb}	3

Slotted bolt hole spacing	s_{slot} (in)	4.1875
Distance from line of bolts to shear tab weld	$C1$ (in)	5.125
Size of weld in 1/16^{ths} of an inch	D_{W3}	7
Required flexural strength of the shear tab, $V_u[C1]$ or $V_u([C1]+d_b/24)$ for roof sloped condition	M_u (k-in)	193.4
Normal demand on weld $(3F_b s_{slot} n_{vb}^2 + 6M_u)/l_{w3}^2 - 2F_b n_{vb}/l_{w3}$ except when bridge plates present, $F_b n_{vb}/l_{w3}$	r_{un} (k/in)	18.1
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	18.4
Weld 3 factored capacity (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)D_{W3}/16(1")(2\ sides)$	ϕr_n (kips)	19.5
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.94 **OK**

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	0.75
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e_d (in)	1.0625
Beam depth	d_b (in)	14.1
Length of the shear tab (see step 12)	l_{tab} (in)	10.625
Maximum beam rotation considered for design (see step 6)	γ_{max} (rad)	0.06
Required shear tab slot dimension (AISC 358 Eq. 15.6-25), $\gamma_{max}(d_b/2 + l_{tab}/2 - e_d) + d_{bolt}/2$	$S1_{req}$ (in)	1.053
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.125 **OK**
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	1.125

Step 15. Check the top plate for shear yielding and shear rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	42.8
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	147.2

Yielding check

Top plate thickness (see step 2)	t_p (in)	0.625
Top plate dimension (see schedule)	$P6$ (in)	17.25
Gross shear area of top plate, $(P6)(t_p)$	A_g (in^2)	10.781
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yp}A_g$	R_n (kips)	323.4
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	323.4
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.46 **OK**

Rupture check

Bolt diameter (see step 4)	d_{bolt} (in)	0.75
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1
Number of short slotted bolts on top plate (see step 5)	n_b	6
Net shear area of top plate, $A_g-(t_p l_s n_b)$	A_n (in^2)	7.031
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{up}$	R_n (kips)	274.2
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	205.7
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.72 **OK**

Step 16. Check the top plate for the limit states of tensile yield and tensile rupture in the net section of the narrow portion (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	42.8
Total number of bolts (see step 5)	n_b	6
Number of bolts in the M region (see step 5)	n_m	3
Required tensile strength of top plates in the narrow portion (AISC 358 Eq. 15.6-28), $(n_m / n_b)(V_{fe} + P_d)$	P_u (kips)	73.6

Yielding check

Top plate dimension	$P4$ (in)	1.375
Top plate dimension	$P5$ (in)	2.125
Top plate thickness (see step 2)	t_p (in)	0.625
Gross area, $([P4]+[P5])*(t_p)$	A_g (in^2)	2.188
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp}A_g$	P_n (kips)	109.4
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	98.4
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	D/C	0.75 **OK**

Rupture check

Top plate dimension	$P4$ (in)	1.375

Top plate dimension	$P5$ (in)	2.125	
Top plate thickness (see step 2)	t_p (in)	0.625	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	0.9375	
Effective area of rupture, $([P4]+[P5]-d_O-1/16)(t_p)$	A_e (in^2)	1.563	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}A_e$	P_n (kips)	101.6	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	76.2	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	D/C	0.97	**OK**

Step 17

Entering and tightening check

Beam flange fillet dimension	k_1	0.8	
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	0.9375	
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	1.75	
Provided P2 dimension	$P2$ (in)	2.375	**OK**

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4	
Total drag force transmitted through the connection (from EOR or min of $0.1A_{gb}F_y$)	$P_{d,total}$ (kips)	85.5	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	42.8	
Top plate thickness (see step 2)	t_p (in)	0.625	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	$P10$ (in)	8.125	
Inside length of the top plate	$P8$ (in)	17.25	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yp}t_p)$	m (in)	8.721	
Dimension, $([P8] - m) / 2$	e (in)	4.265	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yp})e(m+e)t_p / (V_{fe} + P_d)$	$P2_{req}$ (in)	-2.5	
Provided P2 dimension	$P2$ (in)	2.375	**OK**

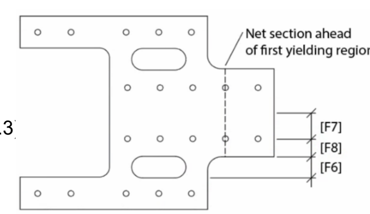
Step 18. Check the fuse plate yielding region proportions and the net section ahead of the first yielding region (AISC *Specification* Chapter D)

Fuse plate yielding region proportions

Maximum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{max}$ (in)	4	
Minimum width of yielding region of fuse plate (AISC 358 15.3(3))	$F6_{min}$ (in)	1.5	
Provided width of yielding region	$F6$ (in)	1.625	**OK**
Top plate thickness (see step 2)	t_p (in)	0.625	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/t_p)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/t_p$	2.6	**OK**

Yielding in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4	
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0	
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0	
Number of bolts in the B region (see step 5)	n_b	6	
Number of bolts in the M region (see step 5)	n_m	3	
Required strength of the fuse plate ahead of the first yielding region (AISC 358 Eq. 15.6-33), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fp}$ (kips)	104.4	
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.375	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.75	
Fuse plate thickness (see step 2)	t_p (in)	0.625	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3	d_O (in)	0.9375	
Gross area for yielding, $2([F7]+[F8])t_p$	A_g (in^2)	6.406	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp}A_g$	P_n (kips)	320.3	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	288.3	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.36	**OK**



Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)

Fuse plate dimension	$F7$ (in)	2.375	
Fuse plate dimension	$F8$ (in)	2.75	
Fuse plate thickness (see step 2)	t_p (in)	0.625	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	0.938	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)t_p$	A_e (in^2)	5.156	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up}R_{tp}A_e$ [a]	P_n (kips)	402.2	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	301.6	

Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.35 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

Step 19. Determine the yielding region depth [F2]

Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	104.4
Fuse plate thickness(see step 2)	t_p (in)	0.625
Width of fuse plate cut-out	$F6$ (in)	1.625
Coefficient in the strain hardening expression, calibrated from experiments	A	1.52
Coefficient in the strain hardening expression, calibrated from experiments	B	0.16
Coefficient in the strain hardening expression, calibrated from experiments	C	0.09
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.95V_{fe}+2t_p(0.6F_{up}R_{tp})B[F6]))/(2t_p(0.6F_{up}R_{tp})[A-C[F6]/t_p])$	$F2_{max}$ (in)	1.521
Provided yielding region depth	$F2$ (in)	1.500 **OK**
Fuse yield force (one side), $2(0.6F_{yp})[F2]t_p$	V_y (kips)	56.3
Ratio	V_{fe} / V_y	1.9

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.25
Minimum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{min}$	0.5
Check width-depth ratios of the yielding regions in the fuse plates, 0.5 $\le [F6] / [F2] \le 1.25$	$F6 / F2$	1.08 **OK**

Step 20. Check the fuse plate for tensile yield and tensile rupture in the net section of the narrow extension (AISC *Specification* Chapter D)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4
Total drag force through fuse plate (all drag force transmitted through top plate)	$P_{d,fuse}$ *(kips)*	0.0
Drag force per line of bolts, $P_{d,fuse} / 2$	P_d (kips)	0.0
Total number of bolts (one side)	n_b	6
Number of bolts in the M region	n_m	3
Required tensile strength of the fuse plates in the narrow extension (AISC 358 Eq. 15.6-35), $2(n_m / n_b)(V_{fe} + P_d)$	$R_{u,fpn}$ (kips)	104.4

Yielding check

Fuse plate dimension	$F4$ (in)	1.375
Fuse plate dimension	$F5$ (in)	1.125
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	0.9375
Gross area of yielding, $2([F4]+[F5])(t_p)$	A_g (in^2)	3.125
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yp} A_g$	R_n (kips)	156.25
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	140.63
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.74 **OK**

Rupture check

Fuse plate dimension	$F4$ (in)	1.375
Fuse plate dimension	$F5$ (in)	1.125
Fuse plate thickness (see step 2)	t_p (in)	0.625
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	0.9375
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(t_p)$	A_e (in^2)	1.875
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{up} R_{tp} A_e$ [a]	R_n (kips)	146.3
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	109.7
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	D/C	0.95 **OK**

[a]The expected tensile strength ($F_{up}R_{tp}$) of the plate may be used when calculating the capacity for rupture of the net section of the fuse plate

MISCELLANEOUS CALCULATIONS