DEPARTMENT OF HEALTH CARE ACCESS AND INFORMATION
OFFICE OF STATEWIDE HOSPITAL PLANNING AND DEVELOPMENT

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 steel fabricator that is approved for the project

Manufacturer's Technical Representative: N/A as it varies

Mailing Address: N/A as it varies

Telephone: N/A as it varies Email: N/A as it varies

Product Information

Product Name: DuraFuse

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.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Applicant Information

Applicant Company Name: DuraFuse Frames, LLC

Contact Person: Jared J Adams, PE

Mailing Address: 5801 West Wells Park Road, West Jordan, UT 84081

Telephone: 949-322-3313 Email: Jared.Adams@DuraFuseFrames.com

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: 10/9/2026

Title: Executive Vice President Company Name: DuraFuse Frames, LLC

Registered Design Professional Preparing Engineering Report

Company Name: DuraFuse Frames, LLC

Name: Derek Nastase, PE SE California License Number: S5995

Mailing Address: 5801 West Wells Park Road, West Jordan, UT 84081

Telephone: 801-727-4060 Email: DNastase@durafuseframes.com

OFFICE USE ONLY
APPLICATION #: PCS- 0004

Disciplines Involved

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

OFFICE USE ONLY – HCAI APPROVAL

Signature: _____ Date: 10/9/2026

Print Name: Kamalpreet Singh Kalsi, SE

Title: Sr. Structural Engineer - HCAI Seismic Compliance Unit

Approved Version Number V4.0

Version History

V1.0 September 18th, 2023 - Original Approval

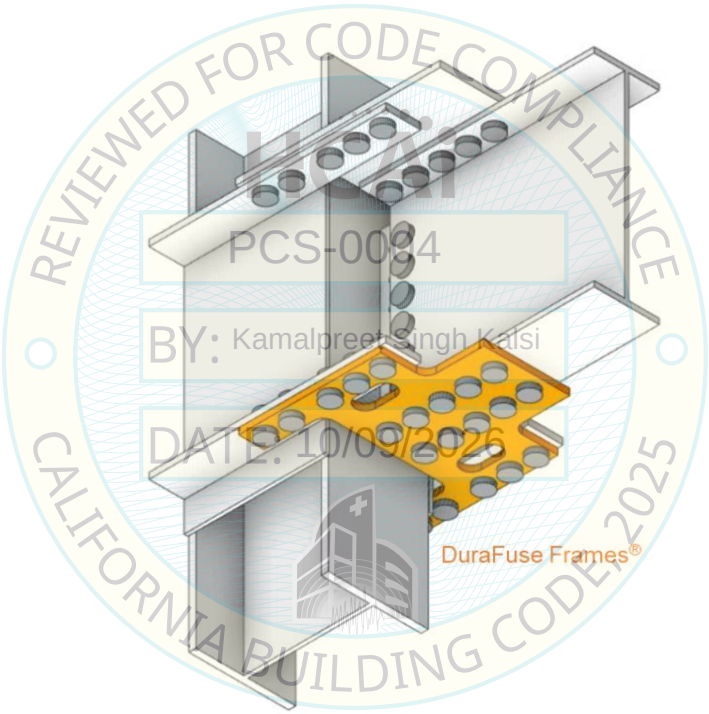
V2.0 January 15th, 2025 - Updated Member Size Approval

V3.0 November 14th, 2025 - Updated Member Size, plate material, and thickness (except for fuse plate)

V4.0 October 9th - 2022 and 2025 CBC versions with revised connection axial load design criteria

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® (DFF)
HCAI Plan Review Guide



Date: 10/9/2026

Version: 4.0



Table of Contents

1 Limits of Pre-Approval 1
2 Connection Description 2
2.1 Overview 2
2.2 Details and Nomenclature 3
2.3 Connection Behavior 5
2.4 Design Responsibility 8
3 Design Methodology 9
3.1 Overview 9
3.2 Drift Checks 9
3.3 Strength 9
3.3.1 Checks Done with ASCE 7 Load Combinations 9
3.3.2 Capacity Design Checks 9
3.4 Seismic Provisions 11
3.4.1 Strong-Column Weak-Beam 11
3.4.2 Lateral Bracing 12
3.4.3 Member Width-Thickness Requirements 13
3.5 Prequalification Limits 13
3.5.1 Beams 13
3.5.2 Columns 13
3.5.3 Plates 14
3.5.4 Bolts 14
3.6 Drags and Collectors 15
3.6.1 In-line 15
3.6.2 Orthogonal 17
3.7 Orthogonal Gravity Cantilevers 21
3.8 In-Plane Gravity Cantilevers 23
3.9 Sloped and Stepped Connections 23
3.10 Example Calculations Summary Sheet 25
4 Computer Modeling 27
4.1 Overview 27
4.2 Beam and Column Elements 27
4.3 Beam-to-Column Connections 27
4.4 Panel Zones 27
5 Fabrication and Erection 28
5.1 Overview 28
5.2 Fabrication and Erection 28

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

5.3 QA/QC 28
5.4 Protected Zones 28
5.5 Fuse Material Check 28
5.6 Welding 28
5.7 Bolting 29
5.8 Fabrication Tolerances 29
5.9 Specification Subsection and TIO 29
5.9.1 Specification Subsection 29
5.9.2 TIO 30
6 Checklist For Plan Reviewers and FAQ 31
7 Standard General Specifications and Drawings 32
8 References 41
9 Appendix A – Example Calculations 42

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

1 Limits of Pre-Approval

The DuraFuse Frames (DFF) connection in accordance with 2022 & 2025 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. A nonlinear analysis procedure is outside the scope of this PCS and shall be considered as an alternative system requiring review and approval by SEOR and HCAI.
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 and 2025 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. Rolled shape column depth shall be limited to W40x maximum for CBC 2022 and W44x for CBC 2025, and do not have a weight limit. The 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 30 inches.
f. The beam depth shall be limited to W40x sections, and the weight shall be limited to 431lbs/ft for CBC 2022
g. The beam depth shall be limited to W44x sections, and the weight shall be limited to 575 lbs/ft for CBC 2025.
h. Fuse Plates shall be A572 Gr. 50 and shall not be thicker than 2-1/4".
i. Beam web shall meet the minimum web slenderness limit of $\lambda_p = 3.76\sqrt{(E/Fy)}$ compact webs per AISC Table B4.1b.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCC 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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 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 welded to the column cover plates. The column has a shear tab with horizontal slotted holes, that is 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. When the top plates are welded to the beam flange, the corresponding, previously referenced, bolt holes are omitted. 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.

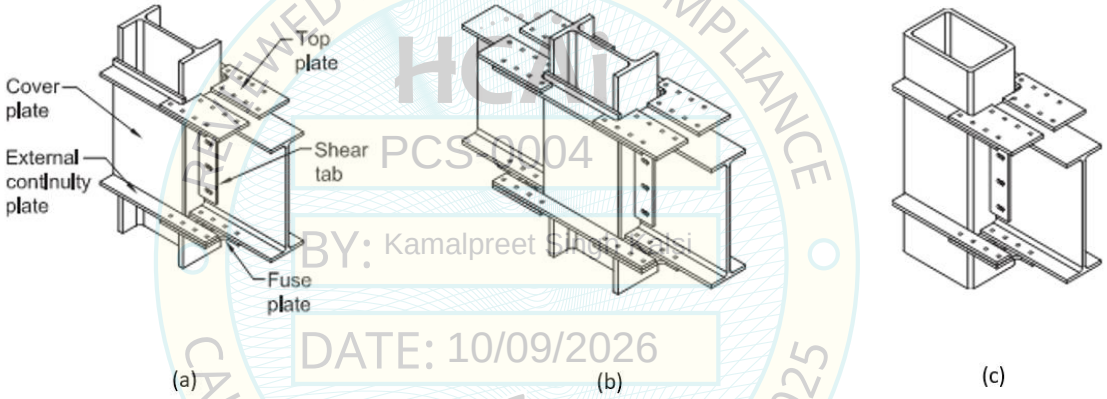


Fig. 1. DuraFuse Frames (DFF) connections (a) one-sided with wide-flange beam and column (b) two-sided with wide flange beams and column (c) HSS or box column condition.

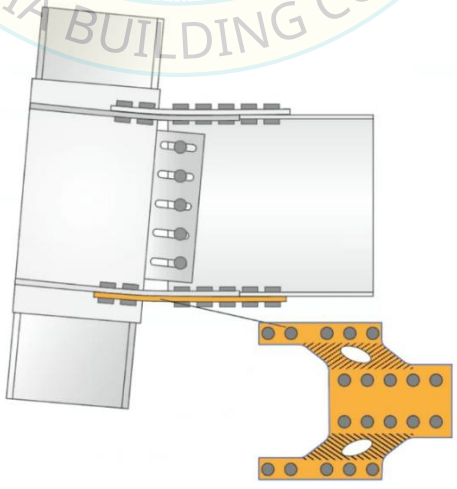


Fig. 2 Inelastic response of a DuraFuse connection with yielding confined to the fuse plate.

REVIEWED FOR CODE COMPLIANCE HCAI PCS-0004 BY: Kamalpreet Singh Gill DATE: 10/09/2026 CALIFORNIA BUILDING CODE 2025

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.

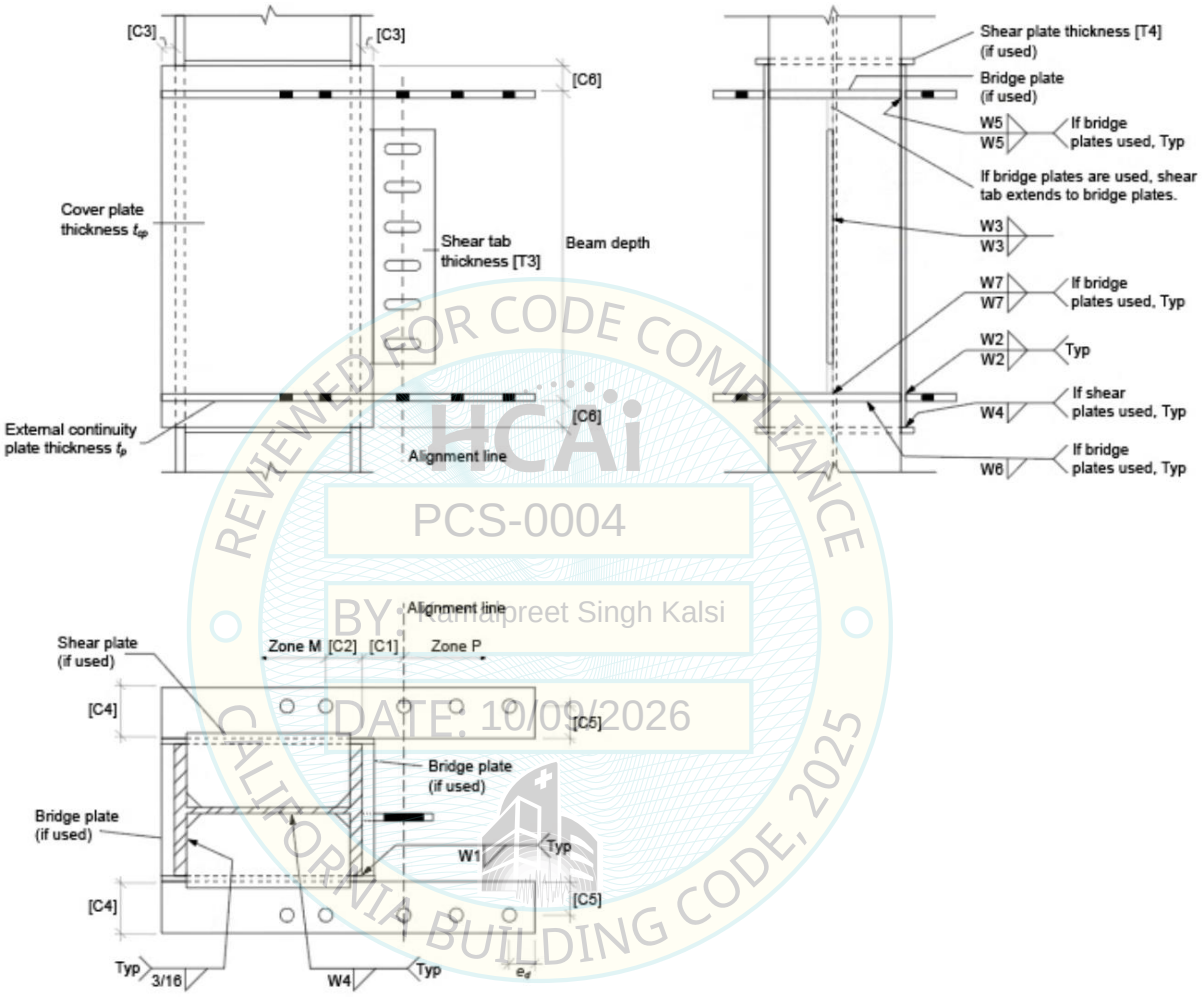


Fig. 3. Column assembly detail.

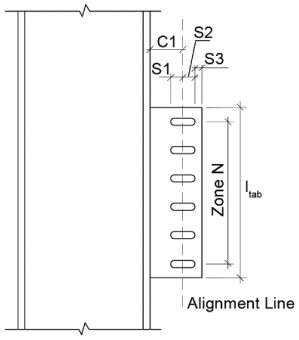


Fig. 4. Shear tab detail.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

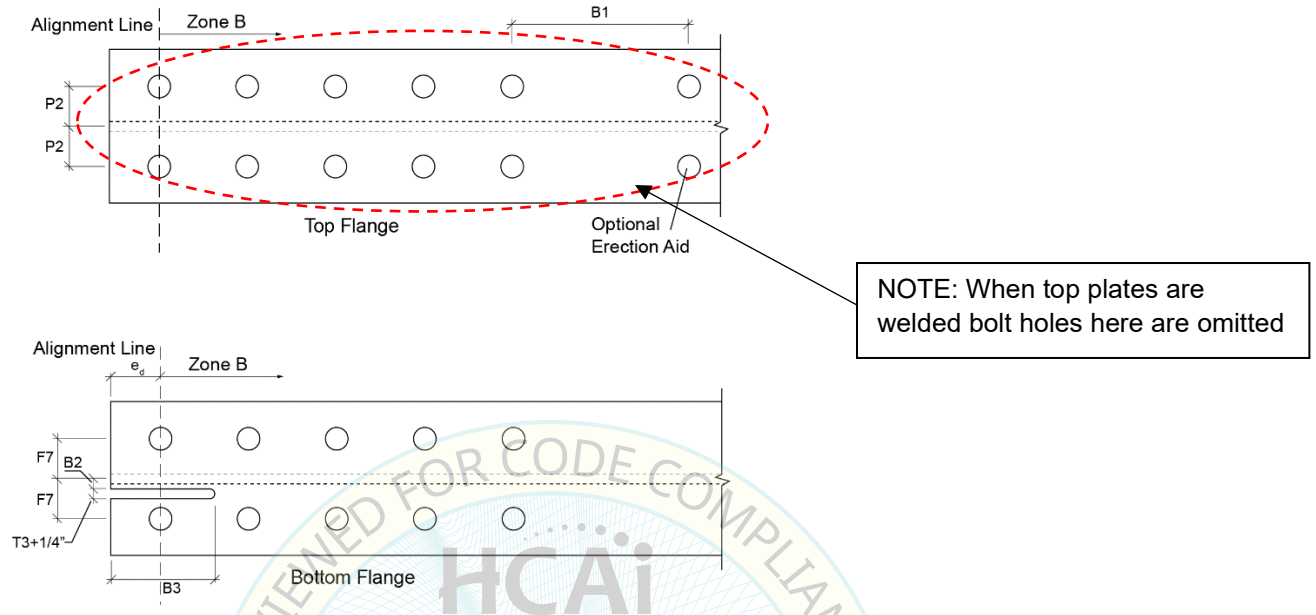


Fig. 5. Beam end detail.

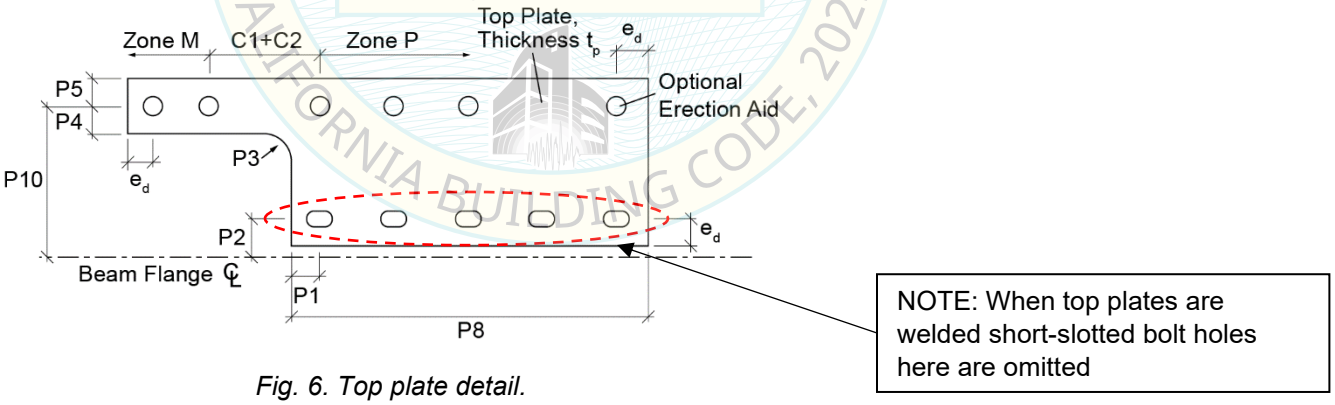


Fig. 6. Top plate detail.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

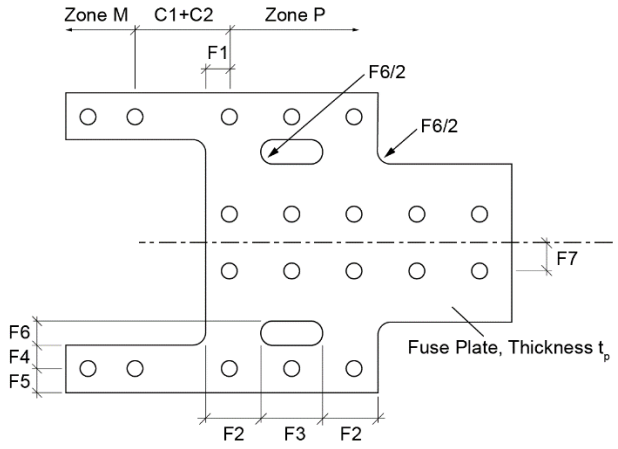


Fig. 7. Fuse plate detail.

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; Lin and Uang 2025) that have been published in peer-reviewed journals (Richards 2019; Richards 2021; Richards 2022; Richards and Oh 2019).

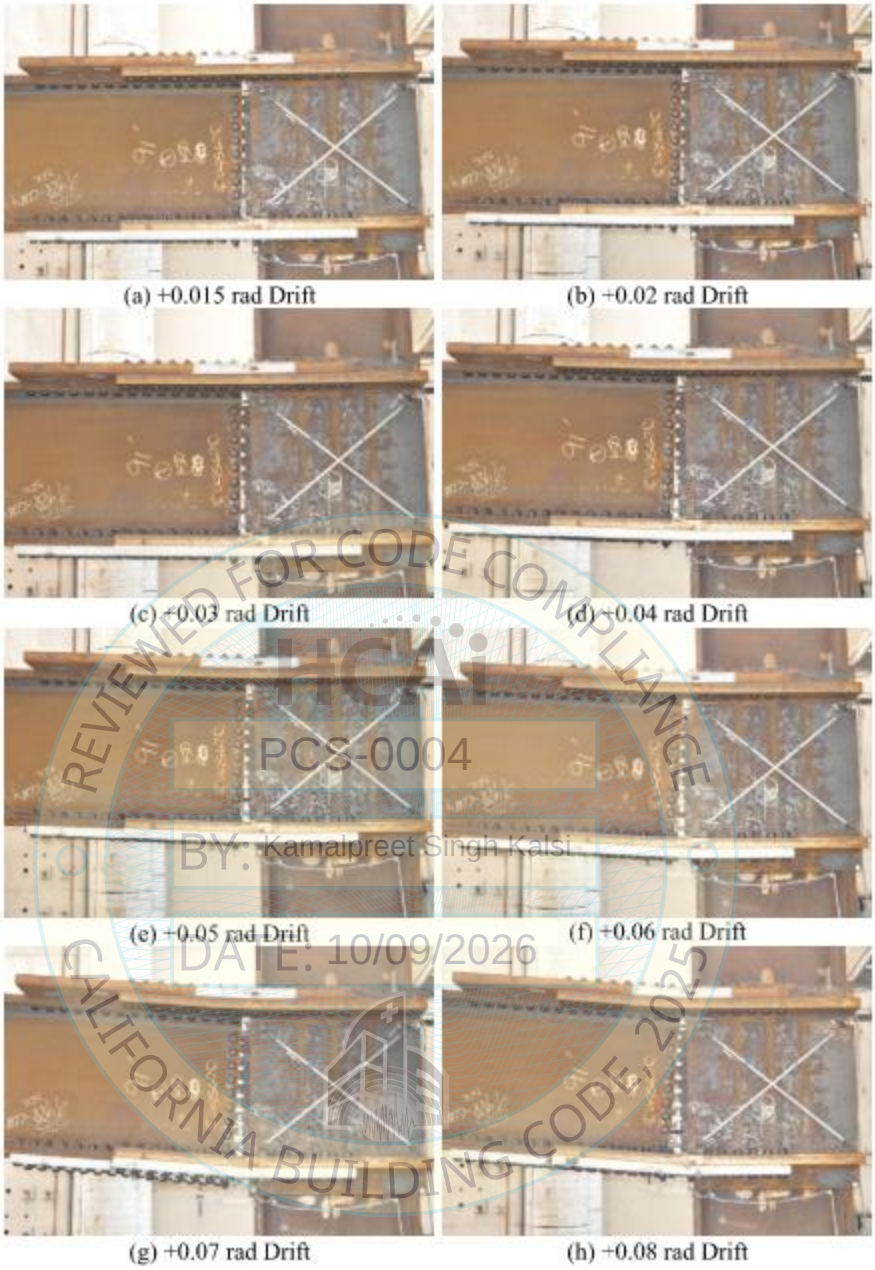
Figure 8 shows a connection test with typical behavior [Specimen E1.2 (Reynolds and Uang 2019a)]. The connection remained elastic throughout the 0.00375, 0.005, 0.0075 rad cycles. During the 0.01 rad cycles, slight flaking of the mill scale indicated localized yielding in the yield regions [Fig. 8(a)]. 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)]. Figure 8.1 shows a similar progression of connection behavior for the L4.3 test.

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).



REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2025
CALIFORNIA BUILDING CODE

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



REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Fig. 8.1. DFF connection behavior [Specimen L4.3 (Lin and Uang 2025)].

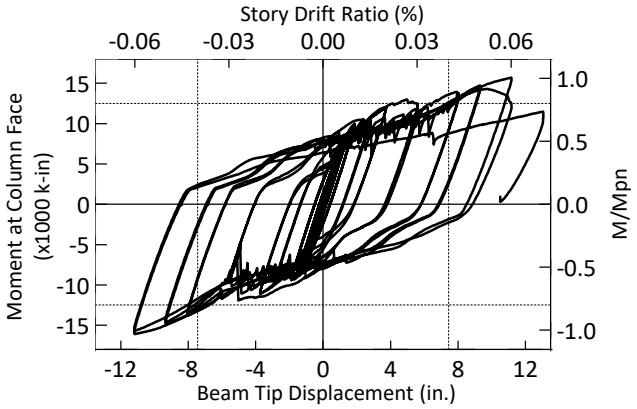


Fig. 9. DFF hysteretic behavior [Specimen E1.2 (Reynolds and Uang 2019a)].

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.”

REVIEWED FOR CODE COMPLIANCE HCAi PCS 0004 BY: DATE: 10/09/2026 CALIFORNIA BUILDING CODE, 2025

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 (CBC 2022 per AISC 341-16 and 358-22 and CBC 2025 per AISC 341-22 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 360-16 & 360-22 respectively and AISC 358-22). The design procedure (AISC 358-22 Section 15.6) addresses these requirements through 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) or modal response spectrum analysis (MRSA) procedures are used to determine elastic drifts, as permitted/required in ASCE 7. Elastic drifts are amplified (ASCE 7 12.8-16) 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) or modal response spectrum analysis (MRSA) procedures are 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 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 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 theapplicable code, 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.

REVIEWED FOR CODE COMPLIANCE HCAI PCS-0004 BY Kamalpreet Singh Kalsi DATE: 1/09/2026 CALIFORNIA BUILDING CODE 2025

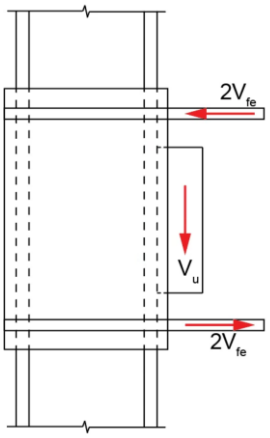


Fig. 10. The bottom flange force associated with M_{pr}.

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 conditions). 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 an 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.

REVIEWED FOR CODE COMPLIANCE HCAi PCS-0004 BY: [illegible] DATE: 10/09/2026 CALIFORNIA BUILDING CODE 2025

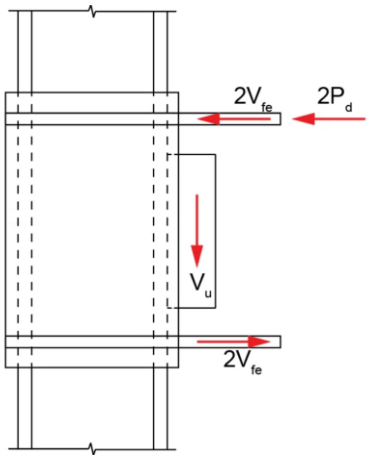


Fig. 11. Demands at top flange level when drag forces are present.

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 shall not be lower than 1.7. 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 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 for CBC 2022 in AISC 341-16 and AISC 360-16 and for CBC 2025 AISC 341-22 and 360-22. The unbraced lengths in the test specimens, L_b, far exceeded the bracing in AISC 341-16 or 341-22 for highly ductile beams [$0.95r_yE/(R_yF_y)$, and $0.86r_yE/(R_yF_y)$, respectively], and the plastic unbraced length, L_p [$1.76r_y\sqrt{E/F_y}$].

Table 1a. Unbraced lengths of test specimens for CBC 2022.

Series	Beam Size	Unbraced Length, L_b[a] (ft)	$0.095r_yE/(R_yF_y)$[b] (ft)	L_p (ft)	L_r (ft)
E	30×99	15.5	8.8	7.42	21.34
F	40×167	15.5	10	8.48	24.78
G	21×50	15.5	5.4	4.59	13.53
H	36×232	20.5	10.9	9.25	29.86
L	40×431	15.5	15.2	12.89	49.04

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

Table 1b. Unbraced lengths of test specimens for CBC 2025

Series	Beam Size	Unbraced Length, L_b[a] (ft)	$0.086r_yE/(R_yF_y)$[b] (ft)	L_p (ft)	L_f (ft)
E	30×99	15.5	7.9	7.42	21.34
F	40×167	15.5	9.1	8.48	24.78
G	21×50	15.5	4.9	4.59	13.53
H	36×232	20.5	9.9	9.25	29.86
L	40×431	15.5	13.8	12.89	49.04

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 and 360-22. DFF calculations include a maximum unbraced length of the beam based on capacity design.

REVIEWED FOR CODE COMPLIANCE CALIFORNIA BUILDING CODE 2025 HCAi P03-0004 BY: Kamalpreet Singh Kalsi DATE: 10/09/2026

3.4.2.2 Column Stability Bracing

Lateral bracing of the columns shall comply with AISC 341-16 and 341-22 respectively. 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 358-22.

(2) Beam depth shall be limited to a maximum of W40x for CBC 2022 and W44x for CBC 2025 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 431 lb/ft, for CBC 2022(2205.A.4.9) and for CBC 2025 shall be limited to a maximum of 575 lbs/ft.

The limitations on beam sizes are based on the extent of experimental testing that was considered by CPRP for AISC 358. 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 a W40×167 (Reynolds and Uang 2019a). The heaviest beam tested at the time of AISC 358-22 review was W36×232 (Reynolds and Uang 2019b). In August of 2021 the heaviest beam tested and reviewed by the AISC 358 CPRP committee is the W40x397 (Lee and Uang 2021) and in October of 2025 the heaviest beam tested is a W40x431 (Lin and Uang 2025).

The heaviest beams tested by 2019 were two K-series beams that were W40x397. Both beams met the HCAI acceptance criteria by completing at least two cycles at 0.04 rad drift and 0.03 rad inelastic rotation (Lee and Uang 2021). The tests indicated ultimate rotational capacity beyond 0.06 rad. The K-series tests were supported by others in the H & F-series tests that indicated ultimate rotation capacity beyond 0.06 rad (Reynolds and Uang 2019a and 2019b).

The heaviest and most substantial tests done in 2025 involved two L4 series beams, each with a W40x431 section. Both specimens successfully met HCAI acceptance criteria by completing at least two cycles at 0.04 radians of drift and 0.03 radians of inelastic rotation. Additionally, a monotonic test conducted on the same W40x431 beam section was performed and demonstrated an ultimate rotational capacity exceeding 0.08 radians (Lin and Uang, 2025).

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 W40x for CBC 2022 and W44x for CBC 2025 . 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 30 in .

REVIEWED FOR CODE COMPLIANCE HCAi PCS-0004 BY: Kamalpreet Singh Kalsi DATE: 10/30/2025 CALIFORNIA BUILDING CODE 2025

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 columns that have been tested are: W36×150 and W36×231 (Reynolds and Uang 2019a) and the W40 columns that have been tested are W40x372 (Lin and Uang 2025). The box columns that were tested were 40 in. deep, 30 in. wide, with a plate thickness of 2 in (Lee and Uang 2021). 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.

3.5.3 Plates

Plates shall satisfy the following limitations:

(1) All Fuse plates shall be fabricated from structural steel that complies with ASTM A572/A572M Grade 50 (Grade 345). Additionally, the 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. The thickness of all Fuse plates shall not exceed 2-1/4" and shall not be less than 0.5" .

(2) All other plates, including but not limited to the external continuity plates and top plates shall be fabricated from structural steel that complies with ASTM A572 Gr. 50, 55, 60, 65, and/or A709 Gr. 70. The thickness of the external continuity plates and top plates shall not exceed 3 in. .

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

(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 fuse plates (Reynolds and Uang 2019a; Reynolds and Uang 2019b; Lin and Uang 2025). In the absence of experimental data for other fuse 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

REVIEWED FOR CODE COMPLIANCE HCAi PCS-0004 BY: Kamalpreet Singh Kalsi DATE: 11/09/2025 CALIFORNIA BUILDING CODE 2025

(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, thermal cutting with grinding, or waterjet 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 or ASTM F3148 Grade 144 assemblies. Threads shall be excluded from the shear plane. Bolt diameter is limited to 1-1/4 in 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 General

Where diaphragm collector/drag forces or chord forces are transferred into or through a DuraFuse connection, the Structural Engineer of Record (SEOR) shall identify, quantify, and document the code-prescribed axial force demands required to establish a complete and continuous load path. A document summarizing the loading at each DuraFuse connection shall be provided in accordance with Section 3.6.1.4.

The DuraFuse connection design shall be based on the forces provided by the SEOR, DuraFuse shall use the provided loads to verify that the DuraFuse connection components are capable of transferring the specified demands.

This procedure is intended to clarify the required force identification and load path assumptions for diaphragm-to-frame interaction and does not replace the overall structural design responsibility of the SEOR.

The overall diaphragm design, collector design, chord design, and global structural load path remain the responsibility of the SEOR.

3.6.1.1 Axial/Collector Load Transfer Diagram

The SEOR shall evaluate and submit a collector load diagram and axial loads within the moment frame region at each DuraFuse connection and include this information in the calculation package. See Figure A below which shows this distributed collector concept. Please note that the load diagram in question is for steel members (not diaphragm). The extents of the collector region outside of the lateral force resisting system (LFRS) region is per the SEOR design.

A direct transfer without the traditional ASCE 7 collector definition (within the moment frame) would still result in axial loads, and the DuraFuse connection shall be designed for these axial demands similar to the code-required amplified collector loads. See Figure B below for reference.

REVIEWED FOR CODE COMPLIANCE
HCAi
DOC 2004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

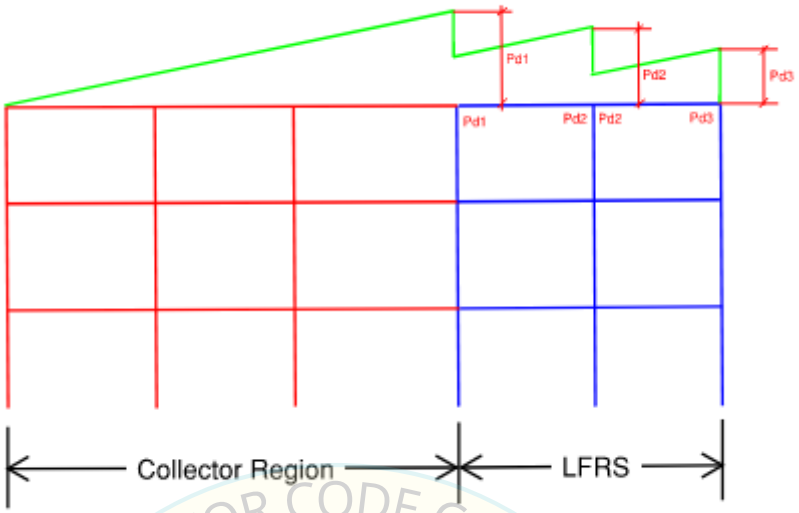


Figure A

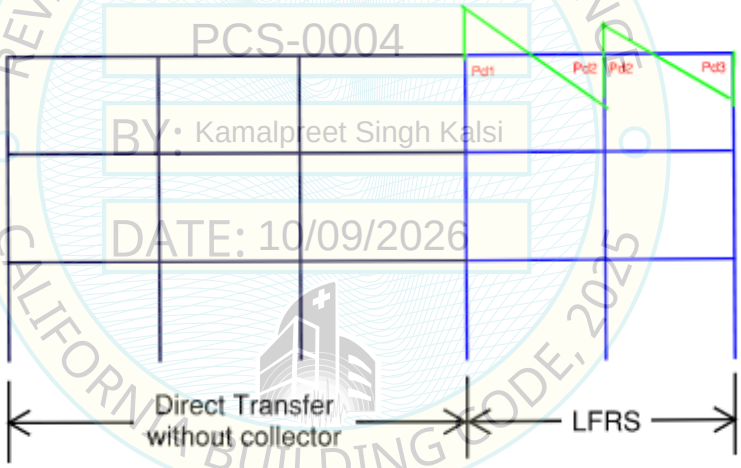


Figure B

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

3.6.1.2 Force Levels and Overstrength (Ω_0) Requirements

For all axial loads in the DuraFuse connections resulting from collectors or direct transfer, Ω_0-amplified forces shall be provided in accordance with CBC 2025 and ASCE 7-22 requirements.

3.6.1.3 Chord Forces

Where diaphragm chord forces are transferred through the moment frame connections, the SEOR shall provide the required axial loads due to chord force demands.

Because chord forces typically develop in the moment frame due to diaphragm action associated with loading perpendicular to the primary moment frames direction, these loads do not occur simultaneously with in-plane collector loads.

Chord forces shall be evaluated independently from moment frame action, collector forces, and in-line drag force demands, unless combined loading is specifically required and identified by the governing analysis provided by the SEOR.

3.6.1.4 Required Documentation

The project calculations shall clearly indicate locations where forces are transferred into and pass through the moment frame connections and shall include:

- Collector/drag/axial force magnitudes and direction
- Chord force magnitudes and direction
- Applicable overstrength factors
- Governing load combinations

The SEOR shall provide sufficient calculations, diagrams, or schedules to clearly communicate these demands to DuraFuse and for HCAI review.

3.6.2 Orthogonal

Orthogonal drag and collector forces reported to DuraFuse shall represent the required force transfer through the DuraFuse connection region and not merely global diaphragm collector totals. . The geometry of the external continuity plates are 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 plates and the weld sizes are checked based on the demands from the moment frame (V_{fe} + P_d) in combination with the orthogonal demands, $P_{d,o}$.

Detail 1/SDF-06 shall be employed in scenarios where the out-of-plane drag forces exceed the capacity of standard plate configurations. Specifically, when the required plate thickness to resist these loads becomes impractically large or fails to satisfy design checks, this alternative detail provides a viable solution.

Out-of-plane drag loads are routinely considered in design; however, Detail 1/SDF-06 is reserved for exceptional cases involving unusually high load demands. Its application is not typical and should be limited to instances where conventional detailing proves inadequate due to extreme loading conditions.

REVIEWED FOR CODE COMPLIANCE HCAi FCS-0004 REVIEWED BY: Name of Submitter DATE: 10/09/2026 FOR HCAI BUILDING CODE 2019

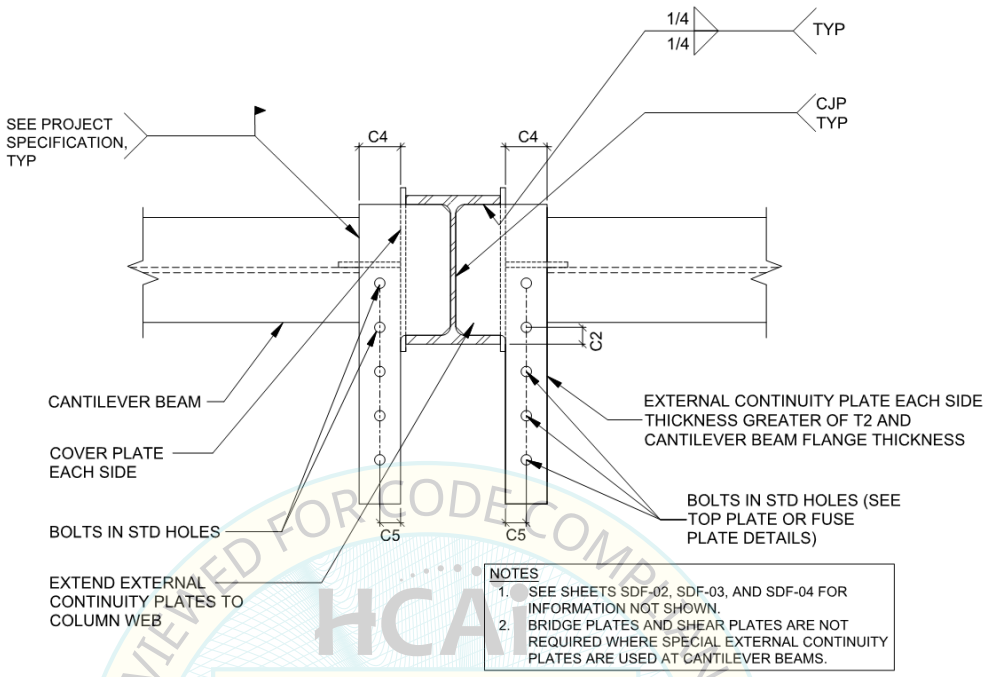


Fig. 12. Special external continuity plate for orthogonal drags or cantilevers.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

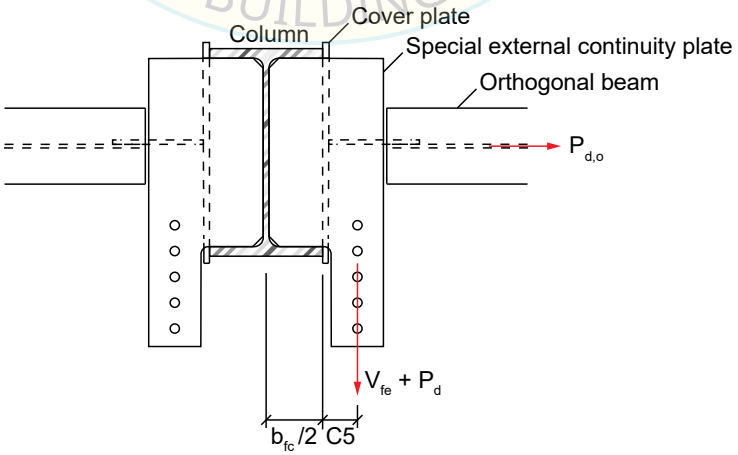


Fig. 13. Terms used in orthogonal load equations.

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.

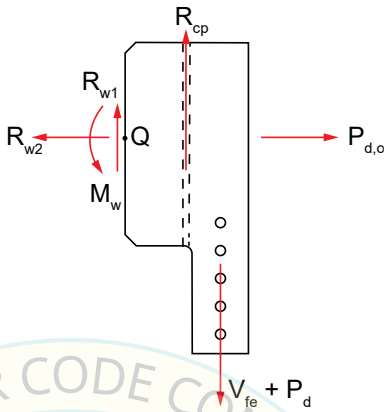


Fig. 14. Demands on special external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

$$\Sigma F_{x,dir} = 0\,, \quad R_{w2} = P_{d,o} \qquad \text{(Eq. 2)}$$

$$\Sigma F_{y,dir} = 0\,, \quad R_{cp} + R_{w1} = V_{fe} + P_d \qquad \text{(Eq. 3)}$$

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

$$\frac{R_{cp}}{t_p} = \frac{R_{w1}}{0.5t_{wc}} \qquad \text{(Eq. 5)}$$

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:

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

Then from Eq. 3:

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

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.

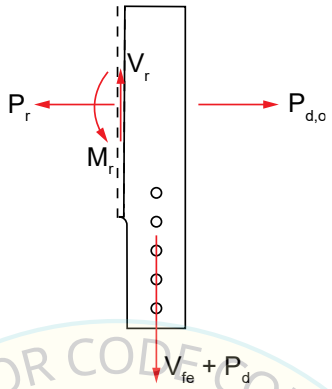


Fig. 15. Special external continuity plate with a cut at the cover plate section.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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

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

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

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

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).

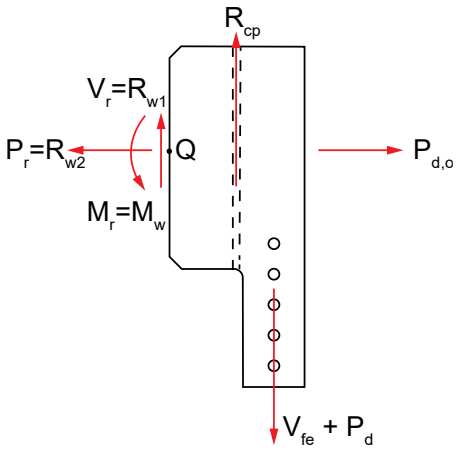


Fig. 16. Special external continuity plate with a cut at the cover plate section.

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

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

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

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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.

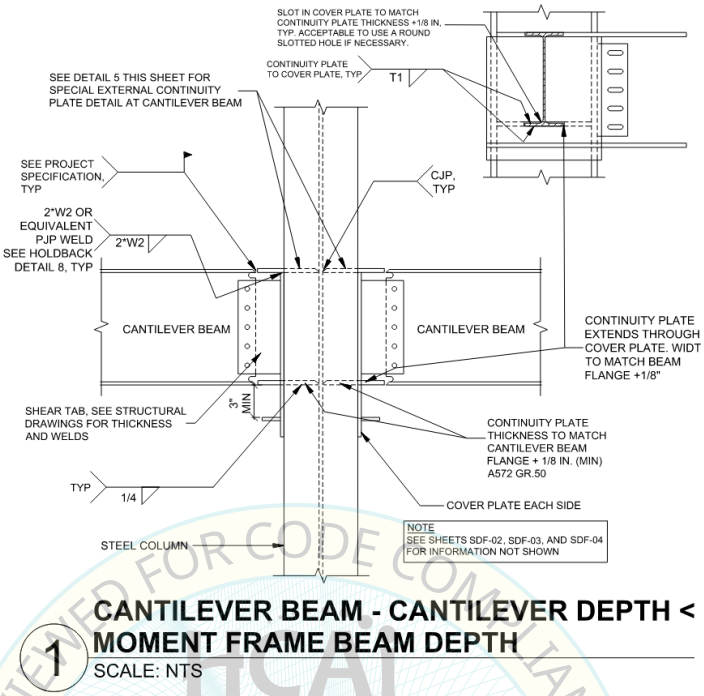


Fig. 17. Orthogonal gravity cantilever when cantilever depth is less than DFF beam depth.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

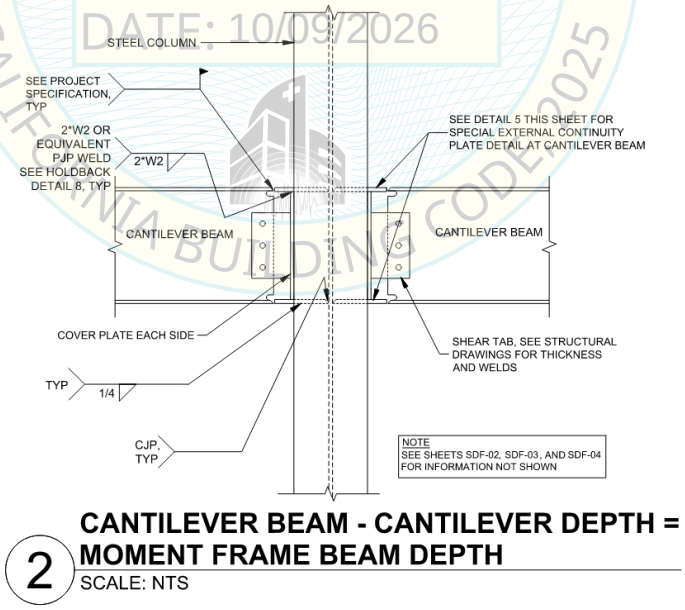


Fig. 18. Orthogonal gravity cantilever when cantilever depth equal to DFF beam depth.

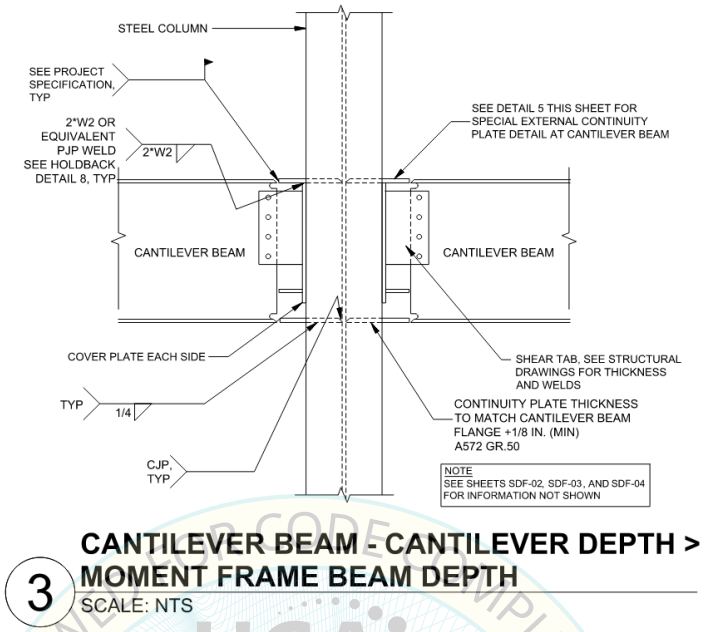


Fig. 19. Orthogonal gravity cantilever when cantilever depth is greater than DFF beam depth.

HCAi
PCS-0004
BY Ramalbeet Singh Kalsi
DATE: 10/09/2026

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.

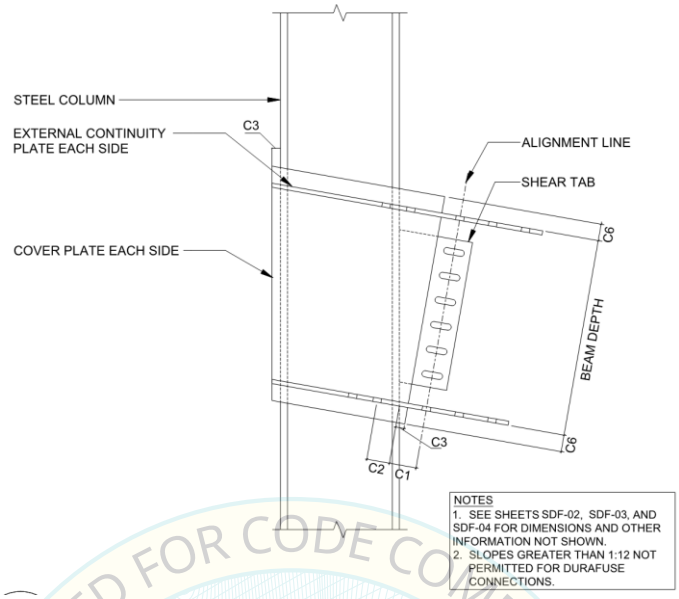


Fig. 20. Connection plates when the beam is sloped.

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).

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

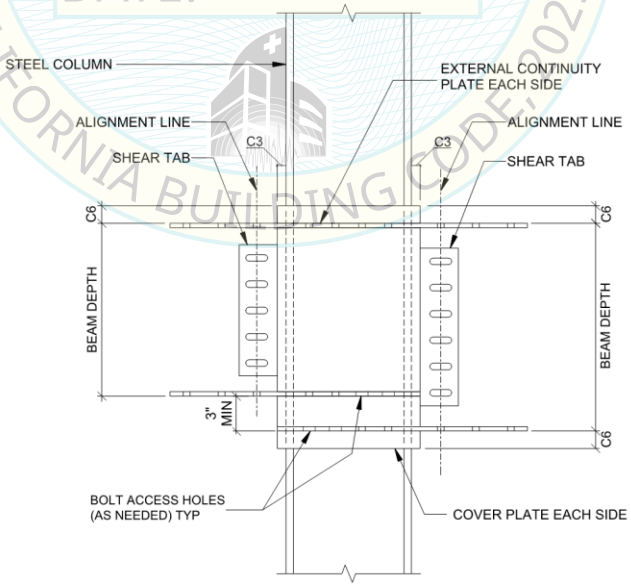


Fig. 21. Column detail for DFF beams of different depth.

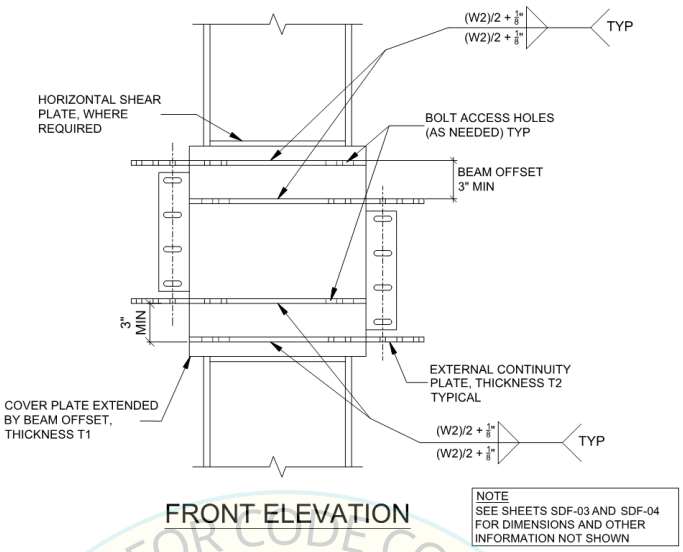


Fig. 22. Column detail for DFF beam that is dropped.

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).

REVIEWED FOR CODE COMPLIANCE HCAi PCS-0004 BY: Kamalpreet Singh Kalsi DATE: 10/20/2022 CALIFORNIA BUILDING CODES

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

HCAi PCS-0004 REVIEWED FOR CODE COMPLIANCE BY: Kamalpreet Singh Kalsi DATE: 10/09/2025 CALIFORNIA BUILDING CODE 2025

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. When justified by connection analysis a non-prismatic beam section can be modeled to include the additional stiffening conditions.

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).

REVIEWED FOR CODE COMPLIANCE
HCAI
PER 2025
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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 CBC 2022 & CBC 2025 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)

REVIEWED FOR CODE COMPLIANCE HCAI POS 0004 BY: Kamalpreet Singh Kalsi DATE: 10/09/2026 CALIFORNIA BUILDING CODE 2025

- 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 at a temperature lower than 0°F as indicated in Section A3.4 of AISC 341-16 and 341-22 as applicable, 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 (or 6.1.2 Table 6.2 as applicable) 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 or ASTM F3148 Grade 144. Bolt diameter is limited to a maximum of 1¼ inches .

Shims with a maximum overall thickness of ¼ inch 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. 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 DuraFuse Frames® 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.

HCAi PCS 0004 BY: Kamalpreet Singh Kalsi DATE: 10/19/2026 REVIEWED FOR CODE COMPLIANCE CALIFORNIA BUILDING CODE 2025

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-S5		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.		

REVIEWED FOR CODE COMPLIANCE
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Fig. 23. Example TIO.

Note: *Figure 23 is based on the 2022 CBC TIO form. TIO item numbers and code references may vary in the 2025 CBC TIO form; however, the required scope of testing, inspection, and observation for the DuraFuse Frame system is not intended to change.*

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 for CBC 2022 and W44x sections for CBC 2025.
    b. Verify that the maximum beam weight is limited to 431 lbs/ft for CBC 2022 and 575 lbs/ft for CBC 2025. Any deviation will be considered as an alternative system by HCAI.
    c. Verify that the maximum rolled column depth is limited to W40x for CBC 2022 and W44x sections for CBC 2025.
    d. Verify that the maximum built-up column width is 30 inches for CBC 2022 and 33 inches for CBC 2025 and that its depth does not exceed the depth permitted for rolled wide-flange shapes.
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).
    b. Verify that the beam flange width-to-thickness ratio is not exceeding λ_p of 0.38*sqrt(E/F_y) per AISC 360 Table B4.1b Case 11.
    c. Verify that the bottom Fuse Plate thickness is not greater than 2-1/4”.
3) Have the beams been braced appropriately to achieve M_p beam?
    a. Inelastic or Elastic lateral torsional buckling can occur for beams with unbraced length greater than Lp (see AISC Figure C-F1.2), 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.
    c. Verify if the models have non-prismatic beam sections, that they have a maximum moment of inertia not greater than the calculated weighted average and a length not greater than the design length of the respective connection ID. (For each connection ID, verify that the summary sheet indicates D/C ratios of 1.0 or less).
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. Verify that the Structural Engineer of Record (SEOR) has identified, quantified, and documented the code-prescribed axial force demands required to establish a complete and continuous load path.
    b. Verify that a document summarizing the loading at each DuraFuse connection is in accordance with Section 3.6.1.5.
    c. Verify that the DuraFuse connection designs are based on and consistent with the forces provided by the SEOR.
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.

NOTES FOR CBC 2022

COPYRIGHT, INTELLECTUAL PROPERTY, AND LICENSE FEE

1.1. © 2026 DURAFUSE FRAMES, LLC. THIS DRAWING IS PROPRIETARY TO DURAFUSE FRAMES, LLC AND IS NOT TO BE USED OR DUPLICATED WITHOUT THE EXPRESS WRITTEN CONSENT OF DURAFUSE FRAMES, LLC.

1.2. DURAFUSE FRAMES® IS A REGISTERED TRADEMARK OF DURAFUSE FRAMES, LLC.

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

PROJECT: HCAI PCS SUBMITTAL V4.0
LOCATION: XX, XX
DURAFUSE TRACKING NUMBER: XXXXXX-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, INCLUDING PENDING APPLICATIONS.

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 AND ACCEPT THE DURAFUSE FRAMES, LLC TERMS AND CONDITIONS BY FILLING OUT THE FORM ON THE WEBSITE: HTTP://DURAFUSEFRAMES.COM/REQUEST-LICENSE -FEE/. IN THE EVENT THE FEE IS NOT GENERATED IN A REASONABLE EXPECTED TIMEFRAME, PLEASE CALL OUR OFFICES AT 801-727-4060 OR EMAIL US AT QUOTE@DURAFUSEFRAMES.COM.

1.7.2. PROSPECTIVE STEEL FABRICATORS SHALL INCLUDE THE LICENSE FEE IN THEIR BID PRICE.

1.7.3. UPON THE 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

1.7. PRIOR TO COMMENCEMENT OF PRODUCING FABRICATION (SHOP) DRAWINGS, THE STEEL FABRICATOR SHALL FORMALLY REQUEST A PRE-CONSTRUCTION MEETING WITH DURAFUSE FRAMES. THIS COORDINATION MEETING CAN BE WEB-BASED OR IN-PERSON. THE PURPOSE WILL BE TO REVIEW PROJECT REQUIREMENTS, TO RESOLVE QUESTIONS, AND TO DISCUSS BEST PRACTICES FOR DETAILING, FABRICATION, AND ERECTION OF THE DURAFUSE FRAMES CONNECTIONS ON THE PROJECT.

DESIGN NOTES

2.1. DESIGN OF THE DURAFUSE FRAMES® SHALL BE IN ACCORDANCE WITH THE FOLLOWING:

- 2022 CALIFORNIA BUILDING CODE
- ALL APPLICABLE BUILDING AND JURISDICTIONAL CODES WITH
- HCAI (OSHPD) AMENDMENTS
- AISC SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS (ANSI/AISC 360-16)
- HCAI PREAPPROVED PREFABRICATED COMPONENTS AND SYSTEMS, PCS-0004 (MOST CURRENT VERSION POSTED)
- AISC SEISMIC PROVISIONS (ANSI/AISC 341-16) **AS PROVIDED FOR AND APPROVED BY HCAI PCS-0004 DURAFUSE FRAMES MOMENT CONNECTION:**
  - BEAM LATERAL STABILITY BRACING IS ONLY REQUIRED PER AISC 360-22. BEAM LATERAL STABILITY BRACING IS NOT REQUIRED PER AISC 341-16
  - MODERATE AND HIGHLY DUCTILE SECTIONS PER AISC 341-16 TABLE D1.1 ARE PERMITTED FOR THE MOMENT FRAME BEAMS
  - MOMENT FRAME BEAMS DO NOT HAVE SPAN-TO-DEPTH RATIO LIMITATIONS PER AISC 358-16
  - THE PROTECTED ZONES ARE THE 4 YIELDING REGIONS ON THE BOTTOM FLANGE FUSE PLATE ONLY (NEITHER THE BEAM, COLUMN, OR COVER PLATE IS PART OF THE PROTECTED ZONE) PER AISC 358-16
- PREQUALIFIED CONNECTIONS FOR SPECIAL AND INTERMEDIATE STEEL MOMENT FRAMES FOR SEISMIC
- APPLICATIONS (ANSI/AISC 358-16)
- 2020 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 DURAFUSE FRAMES ENGINEERING DESIGN RESPONSIBILITIES ARE LIMITED TO DESIGN OF THE BEAM-TO-COLUMN CONNECTIONS IN THE MOMENT FRAMES.

2.3 MEMBER SIZES, DIMENSIONS, AND ELEVATIONS ON DURAFUSE FRAMES SHEETS ARE SHOWN FOR COORDINATION ONLY. PROJECT DOCUMENTS SHALL GOVERN. DURAFUSE BEAM AND COLUMN SIZES SHALL BE LIMITED TO W40X SECTIONS, WITH THE BEAMS BEING LIMITED TO 431 LBS/FT AND NO WEIGHT LIMITS ON THE COLUMNS.

2.4 PRIOR TO SHOP FABRICATION, THE CONTRACTOR SHALL SUBMIT THE FOLLOWING TO DURAFUSE FRAMES, LLC (VIA THE ENGINEER OF RECORD) FOR REVIEW AND DISPOSITION:

2.4.1 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).

2.4.2 WELDING PROCEDURE SPECIFICATION (WPS) AND/OR PROCEDURE QUALIFICATION RECORD (PQR) AND CURRENT CERTIFICATE OF COMPLIANCE BY WIRE MANUFACTURER.

2.4.3 QUALITY CONTROL PROGRAM (REQUIRED IF FABRICATION SHOP IS NOT AISC CERTIFIED).

MATERIALS

3.1. STRUCTURAL SHAPES:

3.1.1. BEAMS SHALL BE ROLLED WIDE-FLANGE OR BUILT-UP I-SHAPED MEMBERS CONFORMING TO THE REQUIREMENTS OF SECTION 2.3 IN AISC 358-22. COLUMNS SHALL BE ANY OF THE SHAPES PERMITTED IN AISC 358 SECTION 2.3. 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 LONGITUDINAL 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 BE, AS A MINIMUM, ASTM 572 GRADE 50.

3.1.4. HSS COLUMNS SHALL CONFORM TO ASTM A500 OR A1085, AS SPECIFIED ON THE SCHEDULE.

3.2. DURAFUSE CONNECTION 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 BRIDGE PLATES SHALL BE FABRICATED FROM STRUCTURAL STEEL COMPLYING WITH ASTM A572 GR. 50, GR. 55, GR. 60, GR. 65, AND/OR A709 GR. 70 STEEL.

3.2.2. FUSE PLATES SHALL BE FABRICATED FROM ASTM A572 GR. 50 STEEL AND SHALL NOT BE THICKER THAN 2-1/4" 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 2 INCHES THICK AND THICKER SHALL HAVE A MINIMUM CHARPY V-NOTCH TOUGHNESS OF 20 FT-LB AT 70°F IN ACCORDANCE WITH A3.3 OF ANSI/AISC 341-22. 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 SHALL BE CONSIDERED AS AN ALTERNATIVE SYSTEM AND REQUIRES APPROVAL BY SEOR AND HCAI.

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, 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 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 AT 0°F IN ACCORDANCE WITH A3.4 OF ANSI/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 TABLE 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.

3.3.5. WHEN WELDING PLATE MATERIALS OF DIFFERENT GROUPS, EITHER OF THE FOLLOWING FILLER METALS MAY BE USED: (1) THAT WHICH MATCHES THE HIGHER STRENGTH BASE METAL, OR (2) THAT WHICH MATCHES THE LOWER STRENGTH BASE METAL AND PRODUCES A LOW-HYDROGEN DEPOSIT AS OUTLINED IN SECTION 3.3.3 OF THESE NOTES.

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 EXTERNAL CONTINUITY PLATES OR BEAMS AND BETWEEN THE FUSE PLATE AND EXTERNAL CONTINUITY PLATES OR BEAMS. SHIMS MAY BE FINGER SHIMS. DOUBLE OR STACKED SHIMS ARE NOT PERMITTED.

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 AND MAY BE PLACED POINTING UPWARDS, DOWNWARDS OR MIXED, AS LONG AS THE THREADS ARE NOT IN THE SHEAR PLANE.

4.3. BOLT HOLES SHALL BE MADE BY DRILLING, SUB-PUNCHING AND REAMING, LASER CUTTING, PLASMA CUTTING, THERMAL CUTTING WITH GRINDING, OR WATER-JET CUTTING. MISDRILLED OR IMPROPERLY LOCATED HOLES MAY BE REPAIRED BY WELD FILLING AND RE-DRILLING. REPAIRED AREAS SHALL BE RESTORED TO THE ORIGINAL BASE METAL THICKNESS AND FINISHED SMOOTH AND FLUSH WITH THE ADJACENT MATERIAL. ALL WELD REPAIRS AND NON DESTRUCTIVE TESTING SHALL BE PER AWS D1.8-2021 SECTION 6.18 AND MUST BE SUBMITTED TO HCAI FOR REVIEW AND APPROVAL PRIOR TO THE REPAIR BEING MADE.

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. 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.

4.13. WHEN COMBINING PLATE MATERIALS OF DIFFERENT GROUPS, IT SHALL BE NOTED THAT ASTM A572 GR. 60, 65 & A709 GR. 70 MATERIALS HAVE HIGHER PREQUALIFIED MINIMUM PREHEATING REQUIREMENTS, SEE AWS D1.1. TABLE 5.8.

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 OF THE TURNED ELEMENTS (TYPICALLY JUST THE 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 THE 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

5.1. 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.

5.2. WITHIN THE PROTECTED ZONE, DISCONTINUITIES CREATED BY FABRICATION OR ERECTION OPERATIONS, BEYOND THOSE REQUIRED PER THE CONTRACT DOCUMENTS FOR INITIAL FABRICATION OF THE FUSE PLATE, SUCH AS, BUT NOT LIMITED TO, 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 APPLIED WITH THE FOLLOWING CRITERIA:

7.2.1. IN THE BEAM REGION (I.E. TOP PLATE, FUSE PLATE, & SHEAR TAB) IT SHALL BE SIMILAR TO THAT OF THE BEAM SECTIONS.

7.2.2. FOR THE COVER PLATE COLUMN SECTIONS, IT SHALL BE DETERMINED JUST LIKE THAT OF A TUBE COLUMN SECTION USING THE THICKNESS OF THE DURAFUSE COVER PLATE.

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. THE CONTRACTOR SHALL PROVIDE SAFE ACCESS TO THE WORK FOR THE SPECIAL INSPECTOR AND OTHER AUTHORIZED INSPECTION PERSONNEL.

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. AS APPLICABLE, WHERE THE 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.

8.8. FOLLOWING COMPLETION OF BOLT INSTALLATION, MINOR VISIBLE GAPS BETWEEN CONNECTED PLIES AND LOCAL PLATE DEFORMATION MAY REMAIN DUE TO THE APPLICATION OF THE REQUIRED BOLT PRETENSION. THESE CONDITIONS ARE COMMON AND DO NOT, BY THEMSELVES, CONSTITUTE CAUSE FOR REJECTION, PROVIDED ALL BOLTS HAVE BEEN FULLY INSTALLED, PROPERLY PRETENSIONED, AND VERIFIED IN ACCORDANCE WITH PROJECT BOLT INSTALLATION AND INSPECTION REQUIREMENTS. FINAL ACCEPTANCE SHALL BE BASED ON COMPLIANCE WITH THE SPECIFIED BOLT INSTALLATION AND PRETENSIONING REQUIREMENTS, RATHER THAN THE COMPLETE ELIMINATION OF ALL VISIBLE GAPS.

DURAFUSE DRAWING VERSION: 1.9.3

PRIOR CONSULTANT WORK
HCAI PCS-0004 V4.0
BY: Kamalpreet Singh Kalsi
DATE: 10/2025
OAKLAND BUILDING

PROJECT:

HCAI PCS SUBMITTAL V4.0

XX, XX

NO.	DATE	DESCRIPTION

ADDITIONAL NOTES:
PCS-0004 V4.0

STAMP AND SIGNATURE:

REGISTERED PROFESSIONAL ENGINEER — SERGIO A. NAVARRETE — S 5995 — STRUCTURAL — STATE OF CALIFORNIA

10-9-26

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

SHEET TITLE:

GENERAL NOTES

DRAWN BY: AM	DATE: 9/18/2026
CHECKED BY: JA	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

SHEET NUMBER:

SDF-01 (2)

NOTES FOR CBC 2025

COPYRIGHT, INTELLECTUAL PROPERTY, AND LICENSE FEE

1.1. © 2026 DURAFUSE FRAMES, LLC. THIS DRAWING IS PROPRIETARY TO DURAFUSE FRAMES, LLC AND IS NOT TO BE USED OR DUPLICATED WITHOUT THE EXPRESS WRITTEN CONSENT OF DURAFUSE FRAMES, LLC.

1.2. DURAFUSE FRAMES® IS A REGISTERED TRADEMARK OF DURAFUSE FRAMES, LLC.

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

PROJECT: HCAI PCS SUBMITTAL V4.0
LOCATION: XX, XX
DURAFUSE TRACKING NUMBER: XXXXXX-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, INCLUDING PENDING APPLICATIONS.

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 AND ACCEPT THE DURAFUSE FRAMES, LLC TERMS AND CONDITIONS BY FILLING OUT THE FORM ON THE WEBSITE: HTTP://DURAFUSEFRAMES.COM/REQUEST-LICENSE -FEE/. IN THE EVENT THE FEE IS NOT GENERATED IN A REASONABLE EXPECTED TIMEFRAME, PLEASE CALL OUR OFFICES AT 801-727-4060 OR EMAIL US AT QUOTE@DURAFUSEFRAMES.COM.

1.7.2. PROSPECTIVE STEEL FABRICATORS SHALL INCLUDE THE LICENSE FEE IN THEIR BID PRICE.

1.7.3. UPON THE 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

1.7. PRIOR TO COMMENCEMENT OF PRODUCING FABRICATION (SHOP) DRAWINGS, THE STEEL FABRICATOR SHALL FORMALLY REQUEST A PRE-CONSTRUCTION MEETING WITH DURAFUSE FRAMES. THIS COORDINATION MEETING CAN BE WEB-BASED OR IN-PERSON. THE PURPOSE WILL BE TO REVIEW PROJECT REQUIREMENTS, TO RESOLVE QUESTIONS, AND TO DISCUSS BEST PRACTICES FOR DETAILING, FABRICATION, AND ERECTION OF THE DURAFUSE FRAMES CONNECTIONS ON THE PROJECT.

DESIGN NOTES

2.1. DESIGN OF THE DURAFUSE FRAMES® SHALL BE IN ACCORDANCE WITH THE FOLLOWING:

- 2025 CALIFORNIA BUILDING CODE
- ALL APPLICABLE BUILDING AND JURISDICTIONAL CODES WITH HCAI (OSHPD) AMENDMENTS
- AISC SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS (ANSI/AISC 360-22)
- HCAI PREAPPROVED PREFABRICATED COMPONENTS AND SYSTEMS, PCS-0004 (MOST CURRENT VERSION POSTED)
- AISC SEISMIC PROVISIONS (ANSI/AISC 341-22) **AS PROVIDED FOR AND APPROVED BY HCAI PCS-0004 DURAFUSE FRAMES MOMENT CONNECTION:**
  - BEAM LATERAL STABILITY BRACING IS ONLY REQUIRED PER AISC 360-22. BEAM LATERAL STABILITY BRACING IS NOT REQUIRED PER AISC 341-22
  - MODERATE AND HIGHLY DUCTILE SECTIONS PER AISC 341-22 TABLE D1.1 ARE PERMITTED FOR THE MOMENT FRAME BEAMS
  - MOMENT FRAME BEAMS DO NOT HAVE SPAN-TO-DEPTH RATIO LIMITATIONS PER AISC 358-22
  - THE PROTECTED ZONES ARE THE 4 YIELDING REGIONS ON THE BOTTOM FLANGE FUSE PLATE ONLY (NEITHER THE BEAM, COLUMN, OR COVER PLATE IS PART OF THE PROTECTED ZONE) PER AISC 358-22
- PREQUALIFIED CONNECTIONS FOR SPECIAL AND INTERMEDIATE STEEL MOMENT FRAMES FOR SEISMIC APPLICATIONS (ANSI/AISC 358-22)
- 2020 RCSC SPECIFICATION FOR STRUCTURAL JOINTS USING HIGH-STRENGTH BOLTS
- STRUCTURAL WELDING CODE-STEEL (AWS D1.1-2020)
- STRUCTURAL WELDING CODE SEISMIC SUPPLEMENT (AWS D1.8-2021)
- IAPMO UES ER 610, INCLUDING THE CALIFORNIA SUPPLEMENT

2.2. DURAFUSE FRAMES ENGINEERING DESIGN RESPONSIBILITIES ARE LIMITED TO DESIGN OF THE BEAM-TO-COLUMN CONNECTIONS IN THE MOMENT FRAMES.

2.3. MEMBER SIZES, DIMENSIONS, AND ELEVATIONS ON DURAFUSE FRAMES SHEETS ARE SHOWN FOR COORDINATION ONLY. PROJECT DOCUMENTS SHALL GOVERN. DURAFUSE BEAM AND COLUMN SIZES SHALL BE LIMITED TO W44X SECTIONS, WITH THE BEAMS BEING LIMITED TO 575 LBS/FT AND NO WEIGHT LIMITS ON THE COLUMNS.

2.4. PRIOR TO SHOP FABRICATION, THE CONTRACTOR SHALL SUBMIT THE FOLLOWING TO DURAFUSE FRAMES, LLC (VIA THE ENGINEER OF RECORD) FOR REVIEW AND DISPOSITION:

2.4.1. 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).

2.4.2. WELDING PROCEDURE SPECIFICATION (WPS) AND/OR PROCEDURE QUALIFICATION RECORD (PQR) AND CURRENT CERTIFICATE OF COMPLIANCE BY WIRE MANUFACTURER.

2.4.3. QUALITY CONTROL PROGRAM (REQUIRED IF FABRICATION SHOP IS NOT AISC CERTIFIED).

MATERIALS

3.1. STRUCTURAL SHAPES:

3.1.1. BEAMS SHALL BE ROLLED WIDE-FLANGE OR BUILT-UP I-SHAPED MEMBERS CONFORMING TO THE REQUIREMENTS OF SECTION 2.3 IN AISC 358-22. COLUMNS SHALL BE ANY OF THE SHAPES PERMITTED IN AISC 358 SECTION 2.3. 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 LONGITUDINAL 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 BE, AS A MINIMUM, ASTM 572 GRADE 50.

3.1.4. HSS COLUMNS SHALL CONFORM TO ASTM A500 OR A1085, AS SPECIFIED ON THE SCHEDULE.

3.2. DURAFUSE CONNECTION 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 BRIDGE PLATES SHALL BE FABRICATED FROM STRUCTURAL STEEL COMPLYING WITH ASTM A572 GR. 50, GR. 55, GR. 60, GR. 65, AND/OR A709 GR. 70 STEEL.

3.2.2. FUSE PLATES SHALL BE FABRICATED FROM ASTM A572 GR. 50 STEEL AND SHALL NOT BE THICKER THAN 2-1/4" 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 2 INCHES THICK AND THICKER SHALL HAVE A MINIMUM CHARPY V-NOTCH TOUGHNESS OF 20 FT-LB AT 70°F IN ACCORDANCE WITH A3.3 OF ANSI/AISC 341-22. 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 SHALL BE CONSIDERED AS AN ALTERNATIVE SYSTEM AND REQUIRES APPROVAL BY SEOR AND HCAI.

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, 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 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 AT 0°F IN ACCORDANCE WITH A3.4 OF ANSI/AISC 341-22, 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 TABLE 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.

3.3.5. WHEN WELDING PLATE MATERIALS OF DIFFERENT GROUPS, EITHER OF THE FOLLOWING FILLER METALS MAY BE USED: (1) THAT WHICH MATCHES THE HIGHER STRENGTH BASE METAL, OR (2) THAT WHICH MATCHES THE LOWER STRENGTH BASE METAL AND PRODUCES A LOW-HYDROGEN DEPOSIT AS OUTLINED IN SECTION 3.3.3 OF THESE NOTES.

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 EXTERNAL CONTINUITY PLATES OR BEAMS AND BETWEEN THE FUSE PLATE AND EXTERNAL CONTINUITY PLATES OR BEAMS. SHIMS MAY BE FINGER SHIMS. DOUBLE OR STACKED SHIMS ARE NOT PERMITTED.

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 2025 CBC CHAPTER 22A.

4.2. BOLTS SHALL BE ARRANGED SYMMETRICALLY ABOUT THE AXIS OF THE BEAM AND MAY BE PLACED POINTING UPWARDS, DOWNWARDS OR MIXED, AS LONG AS THE THREADS ARE NOT IN THE SHEAR PLANE.

4.3. BOLT HOLES SHALL BE MADE BY DRILLING, SUB-PUNCHING AND REAMING, LASER CUTTING, PLASMA CUTTING, THERMAL CUTTING WITH GRINDING, OR WATER-JET CUTTING. MISDRILLED OR IMPROPERLY LOCATED HOLES MAY BE REPAIRED BY WELD FILLING AND RE-DRILLING. REPAIRED AREAS SHALL BE RESTORED TO THE ORIGINAL BASE METAL THICKNESS AND FINISHED SMOOTH AND FLUSH WITH THE ADJACENT MATERIAL. ALL WELD REPAIRS AND NON DESTRUCTIVE TESTING SHALL BE PER AWS D1.8-2021 SECTION 6.18 - ANY CONDITIONS THAT REQUIRE REPAIRS SHALL BE DOCUMENTED AND SUBMITTED FOR REVIEW AND APPROVAL BY SEOR, HCAI AND DURAFUSE.

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. 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.

4.13. WHEN COMBINING PLATE MATERIALS OF DIFFERENT GROUPS, IT SHALL BE NOTED THAT ASTM A572 GR. 60, 65 & A709 GR. 70 MATERIALS HAVE HIGHER PREQUALIFIED MINIMUM PREHEATING REQUIREMENTS, SEE AWS D1.1. TABLE 5.8.

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 OF THE TURNED ELEMENTS (TYPICALLY JUST THE 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 THE 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 PER THE CONTRACT DOCUMENTS FOR INITIAL FABRICATION OF THE FUSE PLATE, SUCH AS, BUT NOT LIMITED TO, TACK WELDS, ERECTION AIDS, AIR-ARC GOUGING AND THERMAL CUTTING SHALL BE REPAIRED. REPAIR AND NON DESTRUCTIVE TESTING SHALL BE PER AWS D1.8-2021 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 APPLIED WITH THE FOLLOWING CRITERIA:

7.2.1. IN THE BEAM REGION (I.E. TOP PLATE, FUSE PLATE, & SHEAR TAB) IT SHALL BE SIMILAR TO THAT OF THE BEAM SECTIONS.

7.2.2. FOR THE COVER PLATE COLUMN SECTIONS, IT SHALL BE DETERMINED JUST LIKE THAT OF A TUBE COLUMN SECTION USING THE THICKNESS OF THE DURAFUSE COVER PLATE.

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 2025 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 2025 CBC CHAPTER 17A. WHERE REQUIRED BY CODE (SECTION 1704A.2.2), THE CONTRACTOR SHALL PROVIDE SAFE ACCESS TO THE WORK FOR THE SPECIAL INSPECTOR AND OTHER AUTHORIZED INSPECTION PERSONNEL.

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. AS APPLICABLE, WHERE THE 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.

8.8. FOLLOWING COMPLETION OF BOLT INSTALLATION, MINOR VISIBLE GAPS BETWEEN CONNECTED PLIES AND LOCAL PLATE DEFORMATION MAY REMAIN DUE TO THE APPLICATION OF THE REQUIRED BOLT PRETENSION. THESE CONDITIONS ARE COMMON AND DO NOT, BY THEMSELVES, CONSTITUTE CAUSE FOR REJECTION, PROVIDED ALL BOLTS HAVE BEEN FULLY INSTALLED, PROPERLY PRETENSIONED, AND VERIFIED IN ACCORDANCE WITH PROJECT BOLT INSTALLATION AND INSPECTION REQUIREMENTS. FINAL ACCEPTANCE SHALL BE BASED ON COMPLIANCE WITH THE SPECIFIED BOLT INSTALLATION AND PRETENSIONING REQUIREMENTS, RATHER THAN THE COMPLETE ELIMINATION OF ALL VISIBLE GAPS.

DURAFUSE DRAWING VERSION: 1.9.3

FOR CONSTRUCTION
HCAI PCS-0004 V4.0
DATE: 10/03/2025
BY: Amalpreet Singh Kalsi
CA BUILDING

PROJECT:

HCAI PCS SUBMITTAL V4.0

XX, XX

NO.	DATE	DESCRIPTION

ADDITIONAL NOTES:
PCS-0004 V4.0

STAMP AND SIGNATURE:

REGISTERED PROFESSIONAL ENGINEER — JOSHUA A. NATANZON — S 5995 — STRUCTURAL — STATE OF CALIFORNIA

10-9-26

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

SHEET TITLE:

GENERAL NOTES

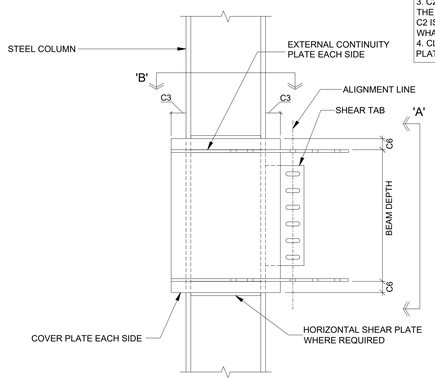
DRAWN BY: AM	DATE: 9/18/2026
CHECKED BY: JA	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

SHEET NUMBER:

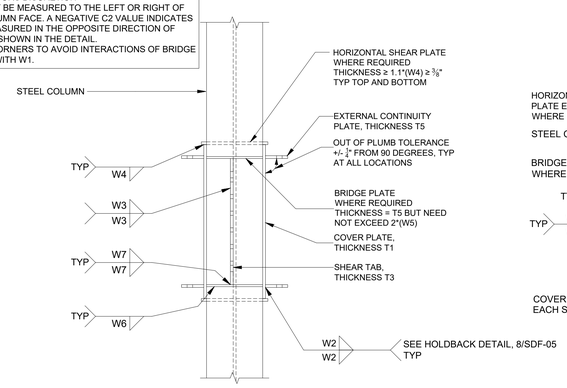
SDF-01

© 2026 DURAFUSE FRAMES, LLC

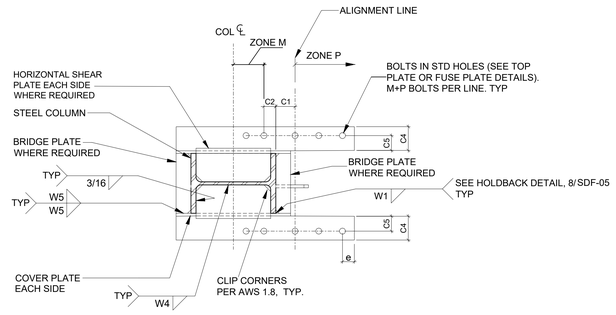
NOTES
1. FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-02 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04
2. WELDS W2, W3, W4, W5, W6, & W7 MAY BE SUBSTITUTED FOR AN EQUIVALENT PJP WELD AT FABRICATORS DISCRETION
3. C2 MAY BE MEASURED TO THE LEFT OR RIGHT OF THE COLUMN FACE. A NEGATIVE C2 VALUE INDICATES C2 IS MEASURED IN THE OPPOSITE DIRECTION OF WHAT IS SHOWN IN THE DETAIL.
4. CLIP CORNERS TO AVOID INTERACTIONS OF BRIDGE PLATES WITH W1.



FRONT ELEVATION



SECTION 'A'

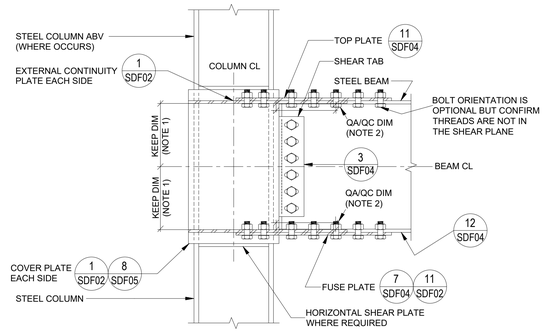


SECTION 'B'

NOTES:
1. KEEP DIMENSION: THE DISTANCE FROM THE THEORETICAL BEAM CENTERLINE SHOWN ON THE APPROVED SHOP DRAWINGS TO THE JUXTAPOSED FACES OF EACH EXTERNAL CONTINUITY PLATE SHALL BE EQUAL TO $\frac{1}{2}$ OF THE DETAILED DIMENSION OF THE BEAM DEPTH, WITH A TOLERANCE OF +1/8", MEASURED FROM THE BEAM CENTERLINE IN BOTH DIRECTIONS AT APPROXIMATELY THE CENTERLINE OF THE COLUMN.

2. QA/QC DIMENSION VERIFICATION: THE POST FABRICATION POSITION OF THE FREE CANTILEVERED ENDS OF THE EXTERNAL CONTINUITY PLATES SHALL BE MEASURED AT THE FARTHEST BOLT HOLE FROM THE COLUMN FACE. THE MEASURED POSITION SHALL NOT DEVIATE FROM THE CORRESPONDING PLATES LOCATION AT THE COLUMN CENTERLINE BY MORE THAN +/-3/8".

IF THE MEASURED DIMENSIONS EXCEED THIS TOLERANCE, THE FABRICATOR MAY CORRECT THE MEMBER BY HEAT STRAIGHTENING IN ACCORDANCE WITH INDUSTRY-ACCEPTED PRACTICES AND PROCEDURES.



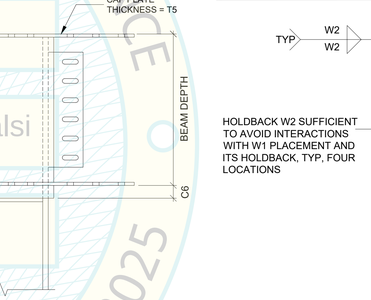
1 ONE SIDED CONNECTION - COLUMN ASSEMBLY
SCALE: NTS

ID	MEMBER SIZES		NO DRAG PARAMETERS		
	COLUMN	BEAM	M BOT	P BOT	B BOT

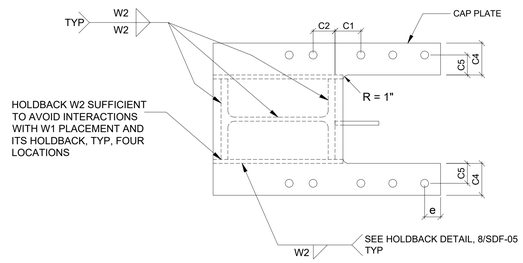
NOTES:
1. ALL BOLTS SHALL BE ASTM F3125 GR. A490 OR F2280 PRETENSIONED HIGH-STRENGTH BOLTS W/ THREADS EXCLUDED (X) FROM SHEAR PLANE.
2. THE "M", "P", AND "B" BOLT COUNTS LISTED IN THIS TABLE ONLY APPLY TO THE FUSE PLATES, BOTTOM EXTERNAL CONTINUITY PLATES, AND BOTTOM FLANGE OF THE BEAM. SEE ONE AND TWO SIDED CONNECTION SCHEDULES FOR BOLT COUNTS ON TOP PLATES, TOP EXTERNAL CONTINUITY PLATES, AND TOP FLANGE OF THE BEAM, DETAIL 9 ON THIS SHEET.

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.



FRONT ELEVATION



TOP VIEW

5 ONE SIDED BOTTOM BOLT CONNECTION SCHEDULE
SCALE: NTS

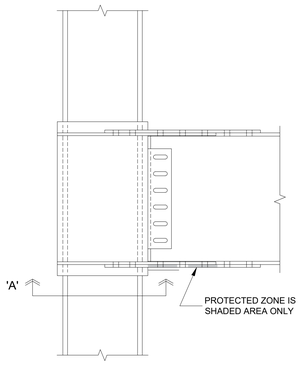
ID	MEMBER SIZES		PLATE THICKNESS					DIMENSIONS						WELDS							BOLTS PER CONNECTION (EACH SIDE)					
	COLUMN	BEAM	T1	T2	T3	T4	T5	C1	C2	C3	C4	C5	C6	W1	W2	W3	W4	W5	W6	W7	DIA	SPACING	M	P	B	N

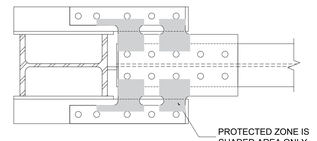
NOTES:
1. ALL BOLTS SHALL BE ASTM F3125 GR. A490 OR F2280 PRETENSIONED HIGH-STRENGTH BOLTS W/ THREADS EXCLUDED (X) FROM SHEAR PLANE.
2. ZONE "M" INCLUDES THE BOLTS THAT CONNECT THE TOP PLATE OR FUSE PLATE TO THE EXTERNAL CONTINUITY PLATE BEHIND THE ALIGNMENT LINE.
   ZONE "P" INCLUDES THE BOLTS THAT 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.
7. WHERE SHIMS ARE PROVIDED, INCREASE WELD SIZE AS NECESSARY TO MAINTAIN REQUIRED WELD CAPACITY.

9 ONE SIDED CONNECTION SCHEDULE
SCALE: NTS

7 TOP-OF-COLUMN CAP PLATE DETAIL FOR ONE 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 BY SUBMITTING THE PROPER PROCEDURE THAT MAY INCLUDE THE FOLLOWING BUT IS NOT LIMITED TO: GRINDING TO SOUND METAL. AREAS REQUIRING MATERIAL REMOVAL IN EXCESS OF THE BASE METAL THICKNESS TOLERANCE SHALL BE RESTORED TO THE ORIGINAL PLATE THICKNESS USING AN APPROVED WELD REPAIR PROCEDURE. ALL REPAIRED SURFACES SHALL BE GROUND SMOOTH AND BLENDED INTO THE ADJACENT BASE METAL TO PRODUCE A WORKMANLIKE FINISH WITH A SURFACE ROUGHNESS NOT EXCEEDING 1000 MICRO-INCHES (SECTION 6.2 IN THE NOTES). ANY CONDITIONS THAT REQUIRE REPAIRS SHALL BE DOCUMENTED AND SUBMITTED FOR REVIEW AND APPROVAL BY THE SEOR, HCAI AND DURAFUSE.
   3.2. WELDS, BOLTS, SCREWS OR SHOT-IN ATTACHMENTS FOR PARTITIONS, DUCT WORK, PIPING OR OTHER CONSTRUCTION SHALL NOT BE PLACED WITHIN THE PROTECTED ZONE.
   3.3. THE SURFACES OF BOLT HOLES AND FUSE PLATE PENETRATIONS LOCATED ADJACENT TO PROTECTED ZONES SHALL EXHIBIT A FINAL SURFACE ROUGHNESS NOT EXCEEDING 1000 MICRO-INCHES. ALL EDGES SHALL BE SMOOTH, FREE OF NOTCHES, GOUGES, AND OTHER DETRIMENTAL SURFACE DISCONTINUITIES.





SECTION 'A'

11 ONE SIDED PROTECTED ZONE
SCALE: NTS

DURAFUSE DRAWING VERSION: 1.9.3

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

PROJECT:

HCAI PCS SUBMITTAL V4.0

XX, XX

NO.	DATE	DESCRIPTION

ADDITIONAL NOTES:
PCS-0004 V4.0

STAMP AND SIGNATURE:

REGISTERED PROFESSIONAL ENGINEER
DEREK A. NASTASE
S 5995
STRUCTURAL
STATE OF CALIFORNIA
10-9-26

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

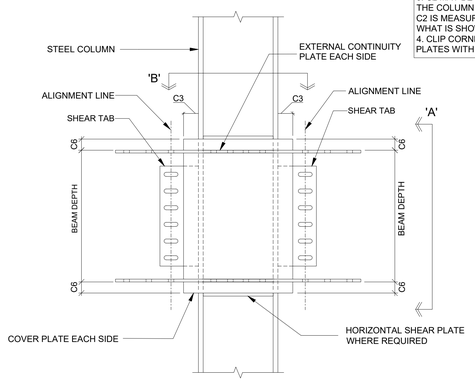
SHEET TITLE:
ONE-SIDED CONNECTION DETAILS

DRAWN BY:	DATE:
AM	9/18/2026
CHECKED BY:	REV #:
JA	
SCALE:	PROJECT NUMBER:
NONE	XXXXXX-00

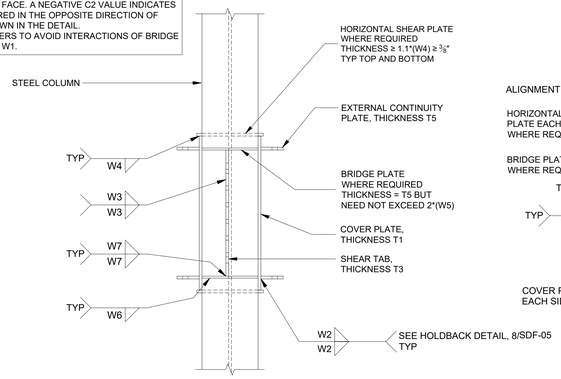
SHEET NUMBER:
SDF-02

© 2026 DURAFUSE FRAMES, LLC

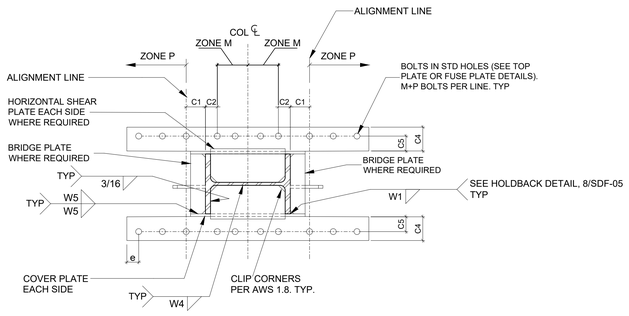
NOTES
1. FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-03 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04
2. WELDS W2, W3, W4, W5, W6, & W7 MAY BE SUBSTITUTED FOR AN EQUIVALENT PJP WELD AT FABRICATORS DISCRETION
3. C2 MAY BE MEASURED TO THE LEFT OR RIGHT OF THE COLUMN FACE. A NEGATIVE C2 VALUE INDICATES C2 IS MEASURED IN THE OPPOSITE DIRECTION OF WHAT IS SHOWN IN THE DETAIL.
4. CLIP CORNERS TO AVOID INTERACTIONS OF BRIDGE PLATES WITH W1.



FRONT ELEVATION



SECTION 'A'



SECTION 'B'

1 TWO SIDED CONNECTION - COLUMN ASSEMBLY
SCALE: NTS

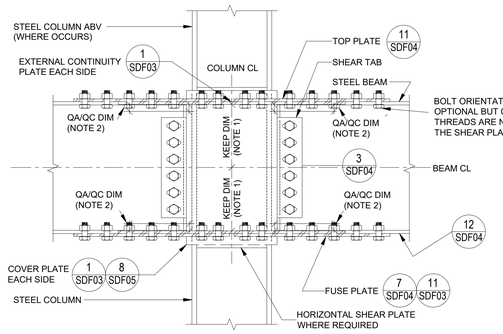
ID	MEMBER SIZES		NO DRAG PARAMETERS		
	COLUMN	BEAM	M BOT	P BOT	B BOT

NOTES:
1. ALL BOLTS SHALL BE ASTM F3125 GR. A490 OR F2280 PRETENSIONED HIGH-STRENGTH BOLTS W/ THREADS EXCLUDED (X) FROM SHEAR PLANE.
2. THE "M", "P", AND "B" BOLT COUNTS LISTED IN THIS TABLE ONLY APPLY TO THE FUSE PLATES, BOTTOM EXTERNAL CONTINUITY PLATES, AND BOTTOM FLANGE OF THE BEAM. SEE ONE AND TWO SIDED CONNECTION SCHEDULES FOR BOLT COUNTS ON TOP PLATES, TOP EXTERNAL CONTINUITY PLATES, AND TOP FLANGE OF THE BEAM, DETAIL 9 ON THIS SHEET.

NOTES:
1. KEEP DIMENSION: THE DISTANCE FROM THE THEORETICAL BEAM CENTERLINE SHOWN ON THE APPROVED SHOP DRAWINGS TO THE JUXTAPOSED FACES OF EACH EXTERNAL CONTINUITY PLATE SHALL BE EQUAL TO ⅛" OF THE DETAILED DIMENSION OF THE BEAM DEPTH, WITH A TOLERANCE OF +1/8", MEASURED FROM THE BEAM CENTERLINE IN BOTH DIRECTIONS AT APPROXIMATELY THE CENTERLINE OF THE COLUMN.

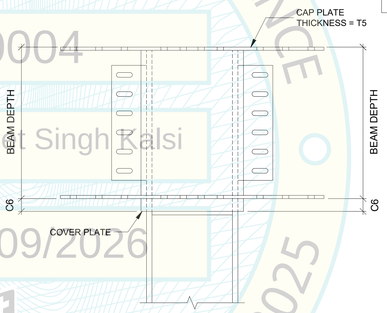
2. QA/QC DIMENSION VERIFICATION: THE POST FABRICATION POSITION OF THE FREE CANTILEVERED ENDS OF THE EXTERNAL CONTINUITY PLATES SHALL BE MEASURED AT THE FARTHEST BOLT HOLE FROM THE COLUMN FACE. THE MEASURED POSITION SHALL NOT DEVIATE FROM THE CORRESPONDING PLATES LOCATION AT THE COLUMN CENTERLINE BY MORE THAN +/-3/8".

IF THE MEASURED DIMENSIONS EXCEED THIS TOLERANCE, THE FABRICATOR MAY CORRECT THE MEMBER BY HEAT STRAIGHTENING IN ACCORDANCE WITH INDUSTRY-ACCEPTED PRACTICES AND PROCEDURES.

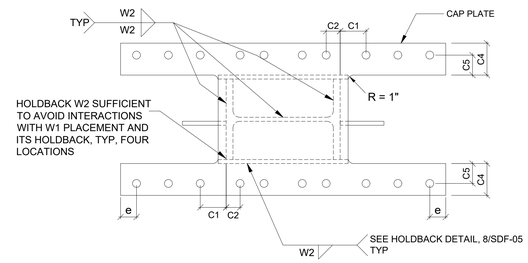


4 TWO SIDED CONNECTION
SCALE: NTS

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

5 TWO SIDED BOTTOM BOLT CONNECTION SCHEDULE
SCALE: NTS

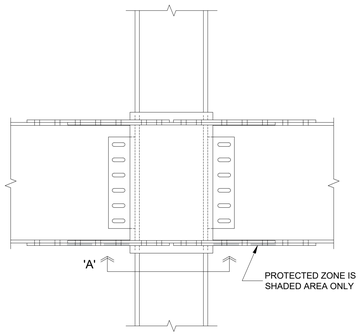
ID	MEMBER SIZES		PLATE THICKNESS					DIMENSIONS						WELDS							BOLTS PER CONNECTION (EACH SIDE)					
	COLUMN	BEAM	T1	T2	T3	T4	T5	C1	C2	C3	C4	C5	C6	W1	W2	W3	W4	W5	W6	W7	DIA	SPACING	M	P	B	N

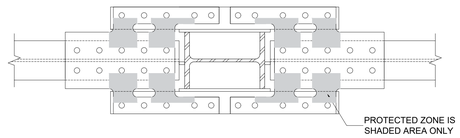
NOTES:
1. ALL BOLTS SHALL BE ASTM F3125 GR. A490 OR F2280 PRETENSIONED HIGH-STRENGTH BOLTS W/ THREADS EXCLUDED (X) FROM SHEAR PLANE.
2. ZONE "M" INCLUDES THE BOLTS THAT CONNECT THE TOP PLATE OR FUSE PLATE TO THE EXTERNAL CONTINUITY PLATE BEHIND THE ALIGNMENT LINE.
   ZONE "P" INCLUDES THE BOLTS THAT 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.
7. WHERE SHIMS ARE PROVIDED, INCREASE WELD SIZE AS NECESSARY TO MAINTAIN REQUIRED WELD CAPACITY.

9 TWO SIDED CONNECTION SCHEDULE
SCALE: NTS

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 BY SUBMITTING THE PROPER PROCEDURE THAT MAY INCLUDE THE FOLLOWING BUT IS NOT LIMITED TO: GRINDING TO SOUND METAL. AREAS REQUIRING MATERIAL REMOVAL IN EXCESS OF THE BASE METAL THICKNESS TOLERANCE SHALL BE RESTORED TO THE ORIGINAL PLATE THICKNESS USING AN APPROVED WELD REPAIR PROCEDURE. ALL REPAIRED SURFACES SHALL BE GROUND SMOOTH AND BLENDED INTO THE ADJACENT BASE METAL TO PRODUCE A WORKMANLIKE FINISH WITH A SURFACE ROUGHNESS NOT EXCEEDING 1000 MICRO-INCHES (SECTION 6.2 IN THE NOTES). ANY CONDITIONS THAT REQUIRE REPAIRS SHALL BE DOCUMENTED AND SUBMITTED FOR REVIEW AND APPROVAL BY THE SEOR, HCAI AND DURAFUSE.
   3.2. WELDS, BOLTS, SCREWS OR SHOT-IN ATTACHMENTS FOR PARTITIONS, DUCT WORK, PIPING OR OTHER CONSTRUCTION SHALL NOT BE PLACED WITHIN THE PROTECTED ZONE.
   3.3. THE SURFACES OF BOLT HOLES AND FUSE PLATE PENETRATIONS LOCATED ADJACENT TO PROTECTED ZONES SHALL EXHIBIT A FINAL SURFACE ROUGHNESS NOT EXCEEDING 1000 MICRO-INCHES. ALL EDGES SHALL BE SMOOTH, FREE OF NOTCHES, GOUGES, AND OTHER DETRIMENTAL SURFACE DISCONTINUITIES.





SECTION 'A'

11 TWO SIDED PROTECTED ZONE
SCALE: NTS

DURAFUSE DRAWING VERSION: 1.9.3

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

PROJECT:

HCAI PCS SUBMITTAL V4.0

XX, XX

NO.	DATE	DESCRIPTION

ADDITIONAL NOTES:
PCS-0004 V4.0

STAMP AND SIGNATURE:

REGISTERED PROFESSIONAL ENGINEER DEREK A. NASTASE S 5995 STRUCTURAL STATE OF CALIFORNIA
10-9-26

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

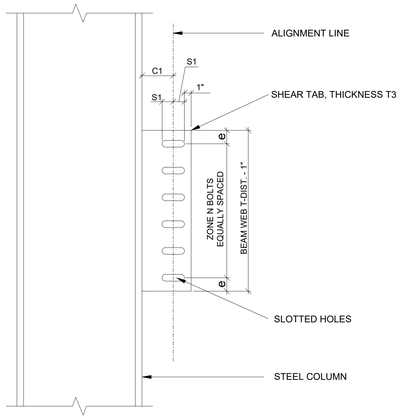
SHEET TITLE:
TWO-SIDED CONNECTION DETAILS

DRAWN BY: AM	DATE: 9/18/2026
CHECKED BY: JA	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

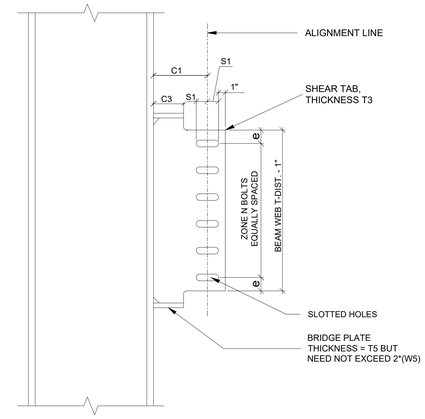
SHEET NUMBER:
SDF-03

© 2026 DURAFUSE FRAMES, LLC

NOTES
1. 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

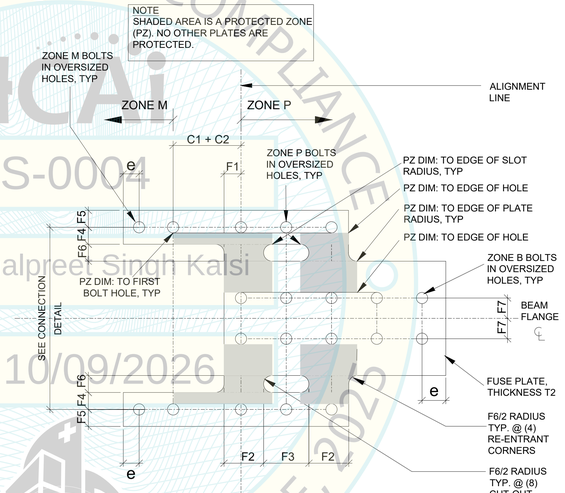


NO BRIDGE PLATE REQUIRED

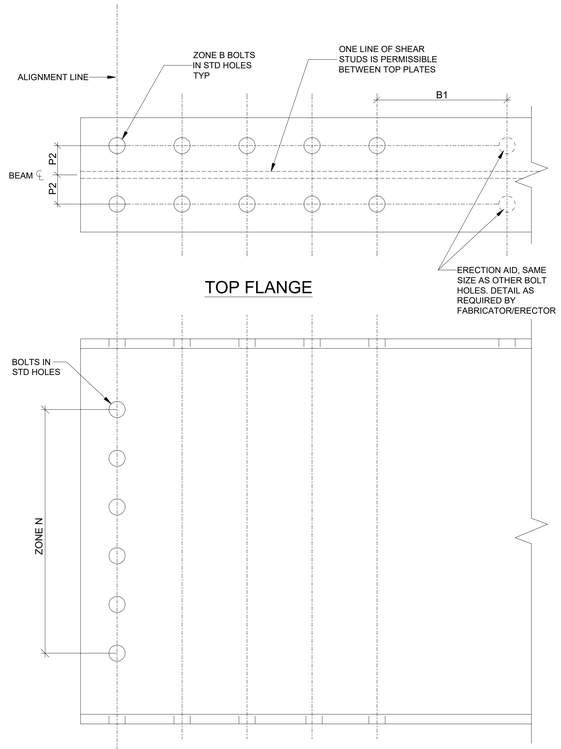


WHEN BRIDGE PLATE REQUIRED

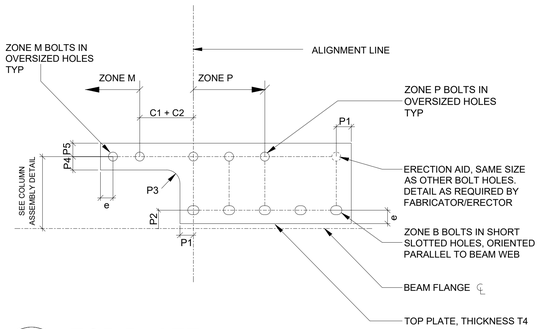
3 SHEAR TAB
SCALE: NTS



7 FUSE PLATE
SCALE: NTS



NOTE:
THE PURPOSE OF THE FLANGE SLOT IS TO ALLOW THE BEAM TO DROP-IN PAST THE SHEAR TAB DURING ERECTION.



11 TOP PLATE
SCALE: NTS

12 BEAM END
SCALE: NTS

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	B1	B2	B3

NOTES:
1. SEE CONNECTION SCHEDULE FOR C1, C2, T1, T2, T3, T4, T5, 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

DURAFUSE DRAWING VERSION: 1.9.3

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

PROJECT:

HCAI PCS SUBMITTAL V4.0

XX, XX

NO.	DESCRIPTION	DATE

ADDITIONAL NOTES:
PCS-0004 V4.0

STAMP AND SIGNATURE:

REGISTERED PROFESSIONAL ENGINEER DEREK A. NASTASE S 5995 STRUCTURAL STATE OF CALIFORNIA
10-9-26

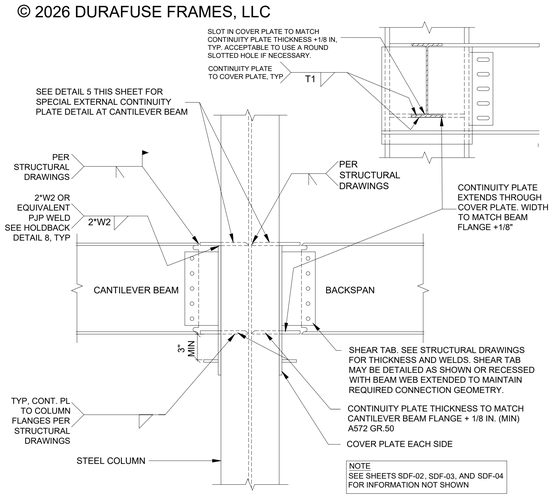
DURAFUSE FRAMES
THE RESILIENT SEISMIC SOLUTION
5801 West Wells Park Rd.
West Jordan, Utah 84081
(801) 727-4060

SHEET TITLE:
BEAM AND PLATE DETAILS

DRAWN BY: AM	DATE: 9/18/2026
CHECKED BY: JA	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

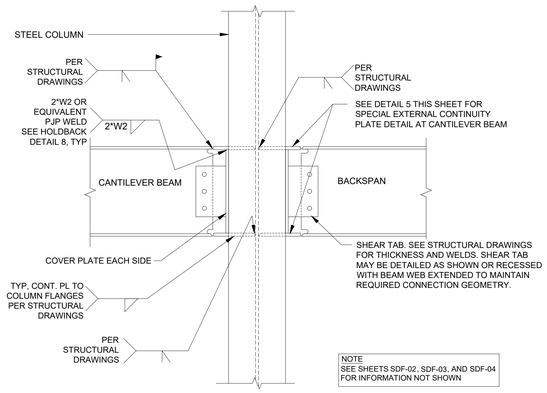
SHEET NUMBER:
SDF-04

© 2026 DURAFUSE FRAMES, LLC



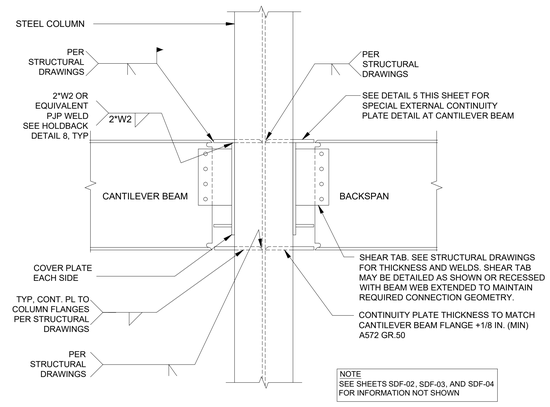
1 CANTILEVER BEAM - CANTILEVER DEPTH < MOMENT FRAME BEAM DEPTH

SCALE: NTS



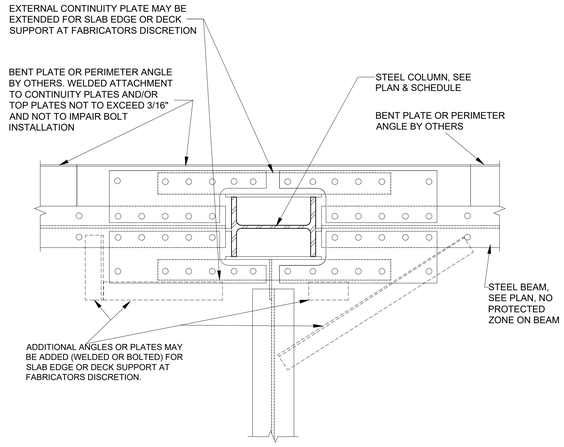
2 CANTILEVER BEAM - CANTILEVER DEPTH = MOMENT FRAME BEAM DEPTH

SCALE: NTS



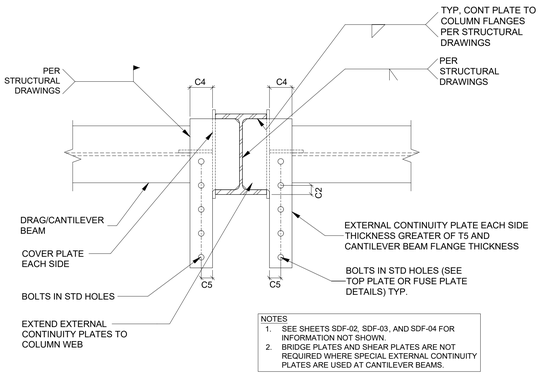
3 CANTILEVER BEAM - CANTILEVER DEPTH > MOMENT FRAME BEAM DEPTH

SCALE: NTS



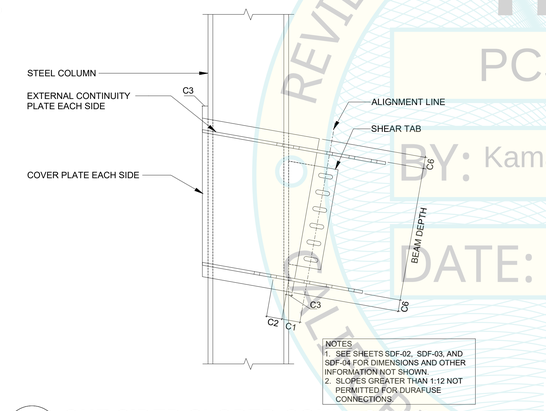
4 SLAB EDGE AND DECK SUPPORT

SCALE: NTS



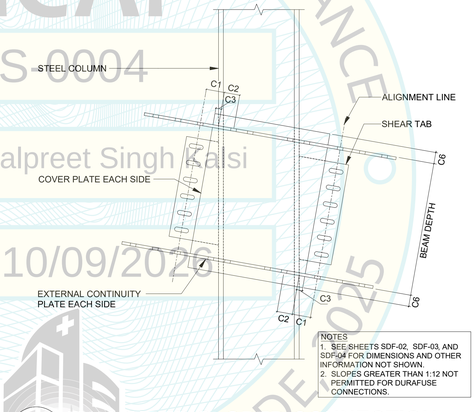
5 SPECIAL EXTERNAL CONTINUITY PLATE AT DRAG/CANTILEVER BEAM

SCALE: NTS



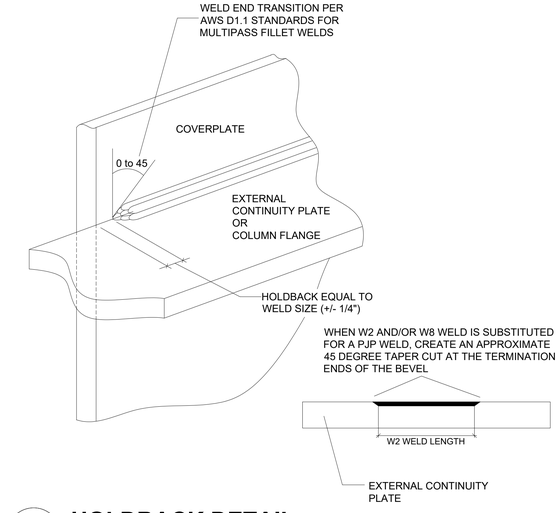
6 ONE SIDED SLOPED CONNECTION

SCALE: NTS



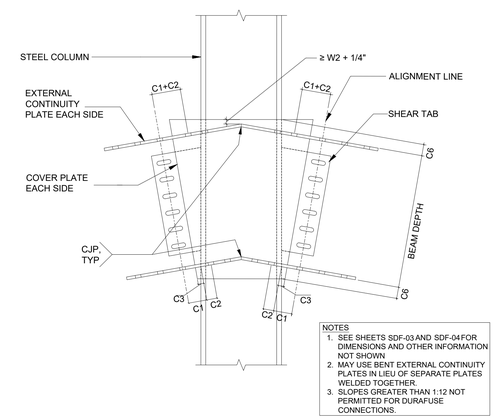
7 TWO SIDED SLOPED CONNECTION

SCALE: NTS



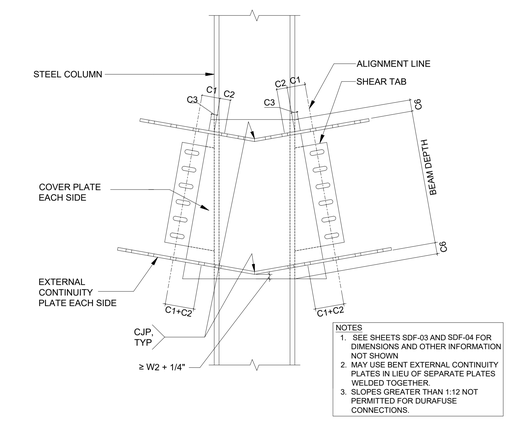
8 HOLDBACK DETAIL

SCALE: NTS



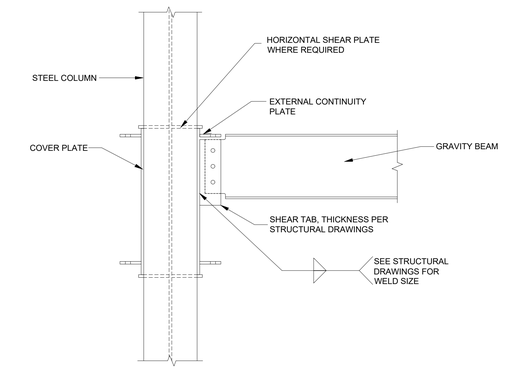
9 TWO SIDED DOUBLE SLOPED CONNECTION

SCALE: NTS



10 TWO SIDED DOUBLE SLOPED CONNECTION

SCALE: NTS



12 EXTENDED WEB OPTION FOR GRAVITY BEAM TO DURAFUSE CONNECTION

SCALE: NTS

DURAFUSE DRAWING VERSION: 1.9.3

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

PROJECT:

HCAI PCS SUBMITTAL V4.0

XX, XX

NO.	DATE	DESCRIPTION

ADDITIONAL NOTES:

PCS-0004 V4.0

STAMP AND SIGNATURE:

REGISTERED PROFESSIONAL ENGINEER DEREK A. NASTASE S 5995 STRUCTURAL STATE OF CALIFORNIA

10-9-26

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

SHEET TITLE:

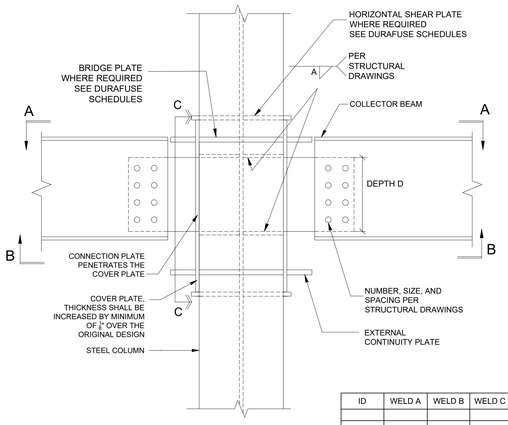
MISCELLANEOUS DETAILS

DRAWN BY: AM	DATE: 9/18/2026
CHECKED BY: JA	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

SHEET NUMBER:

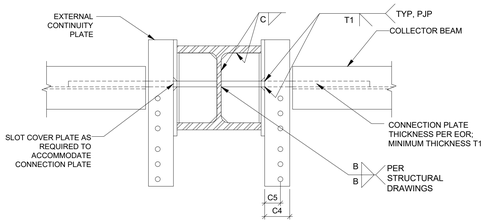
SDF-05

© 2026 DURAFUSE FRAMES, LLC

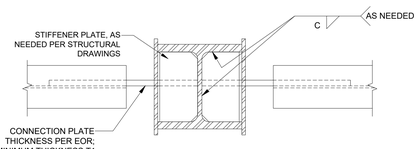


SIDE ELEVATION

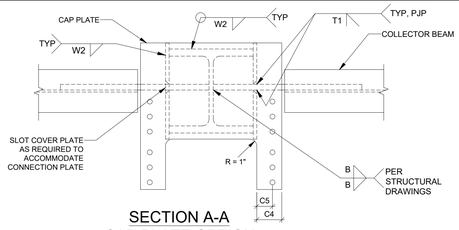
ID	WELD A	WELD B	WELD C	DEPTH D	# BOLTS
*EOR TO POPULATE TABLE					



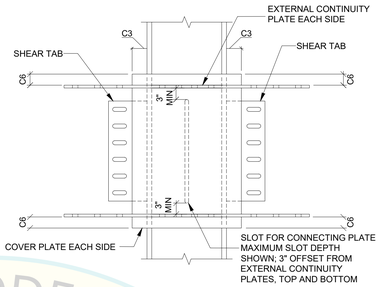
SECTION A-A



SECTION B-B



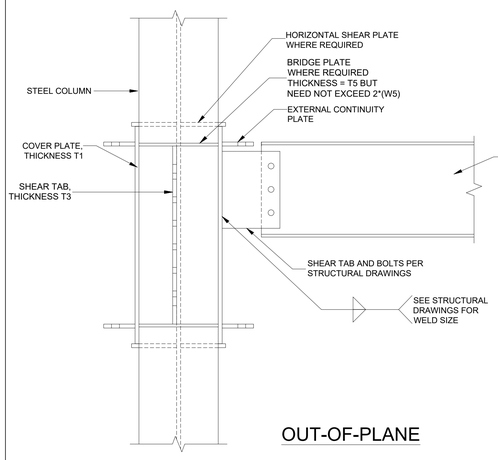
SECTION A-A
CAP PLATE OPTION



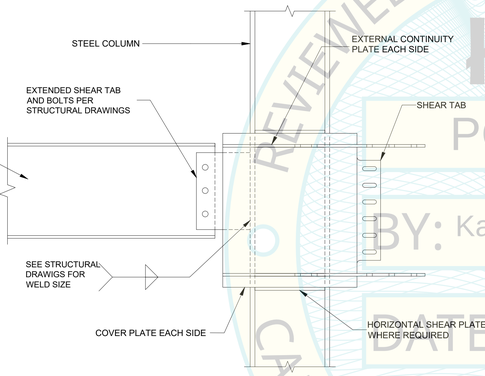
SECTION C-C

1 PERPENDICULAR COLLECTOR CONNECTION

SCALE: NTS



OUT-OF-PLANE

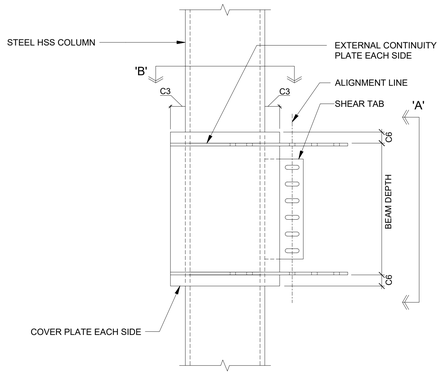


IN-PLANE WHEN BRIDGE PLATE REQUIRED

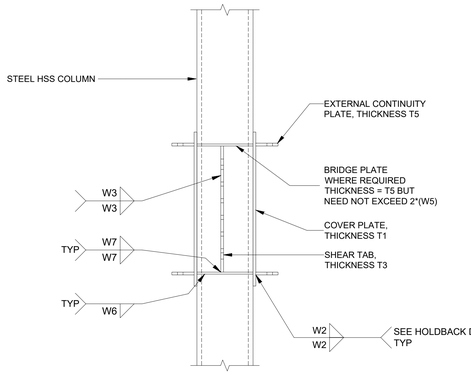
5 EXTENDED SHEAR TAB OPTION FOR GRAVITY BEAM AT DURAFUSE CONNECTION

SCALE: NTS

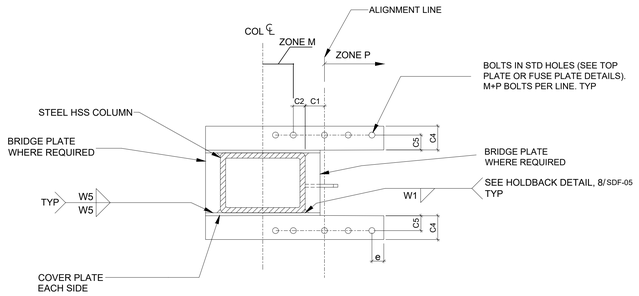
NOTE
FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-02 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04



FRONT ELEVATION



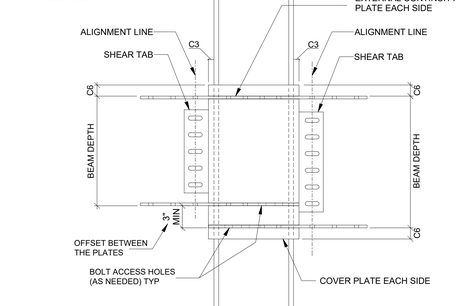
SECTION 'A'



SECTION 'B'

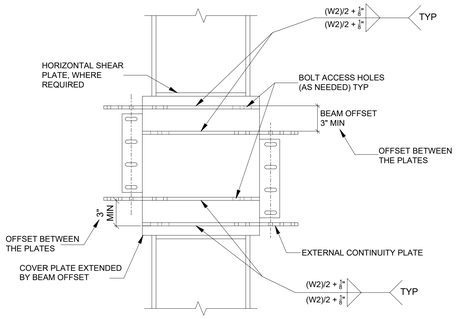
9 ONE SIDED CONNECTION - HSS COLUMN ASSEMBLY

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DURAFUSE DRAWING VERSION: 1.9.3

PROJECT:

HCAI PCS SUBMITTAL V4.0

XX, XX

NO.	DATE	DESCRIPTION

ADDITIONAL NOTES:

PCS-0004 V4.0

STAMP AND SIGNATURE:

REGISTERED PROFESSIONAL ENGINEER DEREK A. NASTASE S 5995 STRUCTURAL STATE OF CALIFORNIA

10-9-26

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

SHEET TITLE:

MISCELLANEOUS DETAILS

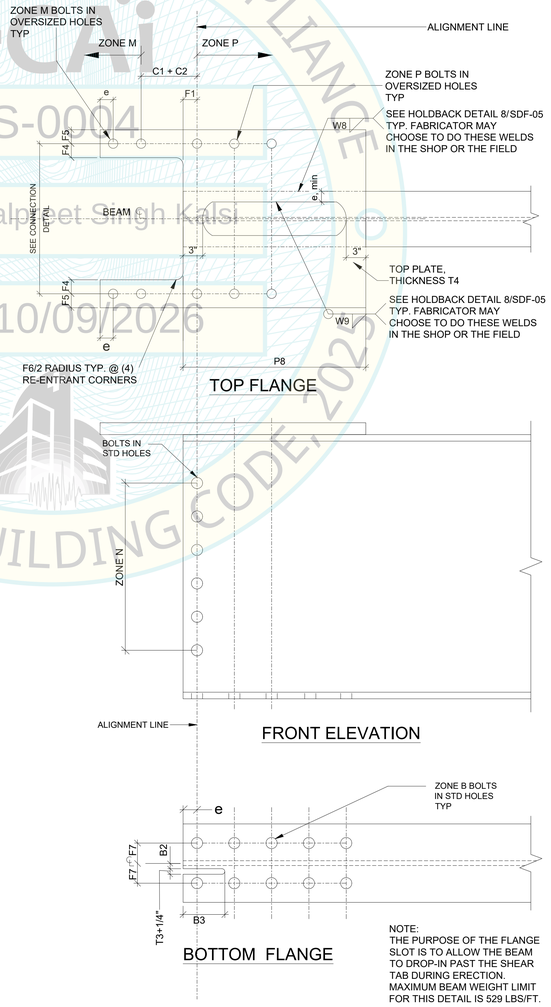
DRAWN BY:	DATE:
AM	9/18/2026
CHECKED BY: JA	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

SHEET NUMBER:

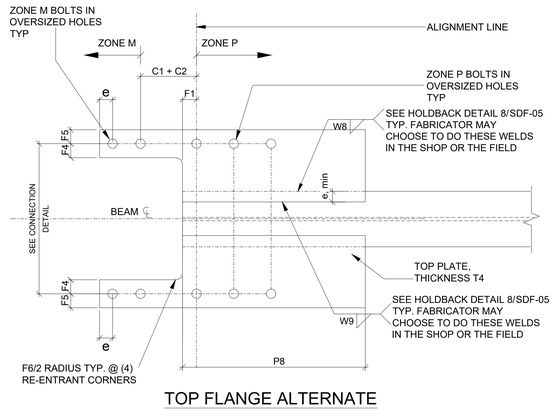
SDF-06

© 2026 DURAFUSE FRAMES, LLC

NOTES
1. 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-07 AND 9/SDF-04
2. WELDS W8 & W9 MAY BE SUBSTITUTED FOR AN EQUIVALENT PJP WELD AT FABRICATORS DISCRETION



TOP FLANGE



TOP FLANGE ALTERNATE

FRONT ELEVATION

BOTTOM FLANGE

NOTE:
THE PURPOSE OF THE FLANGE SLOT IS TO ALLOW THE BEAM TO DROP-IN PAST THE SHEAR TAB DURING ERECTION. MAXIMUM BEAM WEIGHT LIMIT FOR THIS DETAIL IS 529 LBS/FT.

ID	MEMBER SIZES		TOP PLATE PARAMETERS		
	COLUMN	BEAM	P8	W8	W9

NOTES:
1. SEE CONNECTION SCHEDULES ON PREVIOUS SHEETS FOR DIMENSIONS NOT PROVIDED IN THIS TABLE.
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.
4. WHERE SHIMS ARE PROVIDED, INCREASE WELD SIZE AS NECESSARY TO MAINTAIN REQUIRED WELD CAPACITY.

9 **WELDED TOP PLATE AND BEAM END SCHEDULE**
SCALE: NTS

11 **WELDED TOP PLATE & BEAM END**
SCALE: NTS

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

PROJECT:

HCAI PCS SUBMITTAL V4.0

XX, XX

NO.	DATE	DESCRIPTION

ADDITIONAL NOTES:
PCS-0004 V4.0

STAMP AND SIGNATURE:

REGISTERED PROFESSIONAL ENGINEER
JEDREK A. NASTASE
S 5995
STRUCTURAL
STATE OF CALIFORNIA
10-9-26

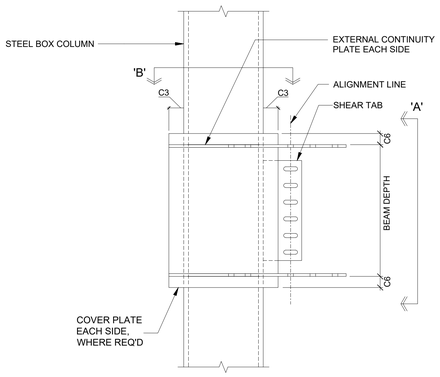
DURAFUSE FRAMES
THE RESILIENT SEISMIC SOLUTION
5801 West Wells Park Rd.
West Jordan, Utah 84081
(801) 727-4060

SHEET TITLE:
ALTERNATE BEAM AND PLATE DETAILS

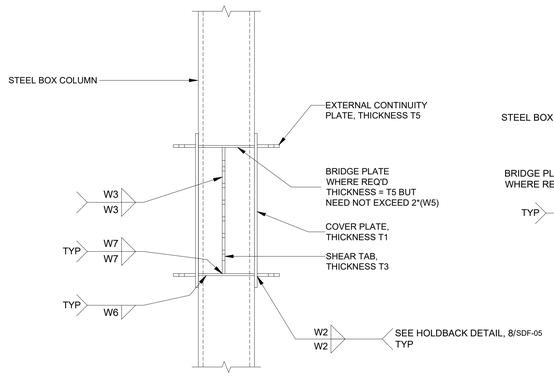
DRAWN BY:	DATE:
AM	9/18/2026
CHECKED BY:	REV #:
JA	
SCALE:	PROJECT NUMBER:
NONE	XXXXXX-00

SHEET NUMBER:
SDF-07

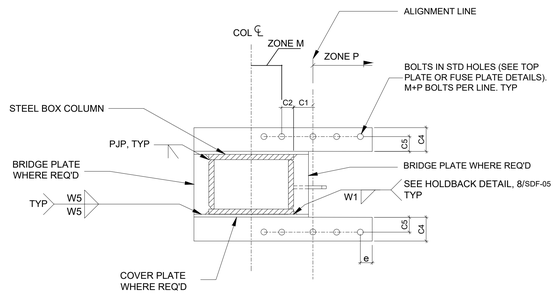
NOTE
FOR DIMENSIONS INDICATED, SEE CONNECTION SCHEDULE DETAIL 9/SDF-02 OR PLATE AND BEAM END SCHEDULE DETAIL 9/SDF-04



FRONT ELEVATION



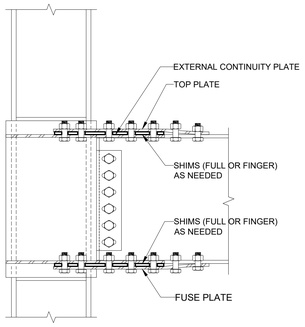
SECTION 'A'

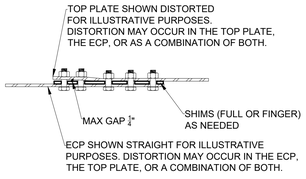


SECTION 'B'

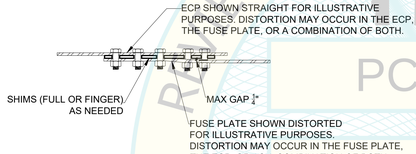
1 ONE SIDED CONNECTION - BOX COLUMN ASSEMBLY

SCALE: NTS





TOP CONDITION



BOTTOM CONDITION

NOTES:
1. CONNECTED PLATES (ECPS, TOP PLATES, AND FUSE PLATES) MAY EXHIBIT MINOR OUT-OF-PLANE DEFORMATION ASSOCIATED WITH FIT-UP, BOLT INSTALLATION, OR WELDING OPERATIONS. SUCH DEFORMATION SHALL NOT BE CAUSE FOR REJECTION PROVIDED THE PLATE MATERIAL EXHIBITS NO EVIDENCE OF CRACKING, TEARING, DELAMINATION, LAMELLAR TEARING, OR OTHER FABRICATION-INDUCED DEFECTS.
2. PLATES EXHIBITING OUT-OF-PLANE DISTORTION GREATER THAN 1 IN. PER FT SHALL BE SUBJECT TO ADDITIONAL EVALUATION USING ULTRASONIC TESTING (UT) OR OTHER APPROVED NONDESTRUCTIVE EXAMINATION (NDE) METHODS AS DIRECTED BY THE SEOR, HCAI, AND DURAFUSE.
3. ANY INDICATIONS OF CRACKING OR OTHER MATERIAL DISCONTINUITIES SHALL BE REPORTED IMMEDIATELY FOR HCAI, SEOR, AND DURAFUSE REVIEW AND DISPOSITION.
4. SHIMS (FULL OR FINGER) MAY BE PROVIDED AS NECESSARY TO ACCOMMODATE FIT-UP TOLERANCES. THE TOTAL SHIM THICKNESS AT ANY INDIVIDUAL INTERFACE SHALL NOT EXCEED 1/4 IN. SHIMS MAY BE INSTALLED BETWEEN THE TOP OR BOTTOM FACES OF THE BEAM FLANGES AND MAY BE BETWEEN THE TOP PLATES AND/OR BETWEEN THE TOP EXTERNAL CONTINUITY PLATE (ECP) AND THE TOP BEAM FLANGE PLATE OR, BETWEEN THE BOTTOM FACE OF THE BEAM FLANGE AND THE FUSE PLATES AND/OR BETWEEN THE BOTTOM EXTERNAL CONTINUITY PLATES AND THE FUSE PLATES RESPECTIVELY AND AS REQUIRED.
5. PRIOR TO BOLT PRETENSIONING, GAPS BETWEEN FAYING SURFACES MAY BE GREATER THAN THOSE PRESENT IN THE FINAL ASSEMBLED CONDITION. THE INSTALLING CONTRACTOR SHALL VERIFY THAT THE CONNECTED ELEMENTS CAN BE SATISFACTORILY DRAWN TOGETHER BY BOLT PRETENSIONING.
6. FOLLOWING COMPLETION OF BOLT PRETENSIONING, LOCALIZED TRAPEZOIDAL GAPS BETWEEN FAYING SURFACES SHALL BE PERMITTED PROVIDED THE MAXIMUM GAP, MEASURED AT ANY LOCATION, DOES NOT EXCEED 1/4 IN.

4 DURAFUSE FIELD CONDITION FIT-UP EXAMPLE DETAIL

(SHOWING POTENTIAL SHIM PLACEMENT, BOLT TOLERANCE AND PLATE DISTORTION)

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

PROJECT:

HCAI PCS SUBMITTAL V4.0

XX, XX

NO.	DESCRIPTION	DATE

ADDITIONAL NOTES:
PCS-0004 V4.0

STAMP AND SIGNATURE:

REGISTERED PROFESSIONAL ENGINEER — JOSEPH A. NASTASI — S 5995 — STRUCTURAL — STATE OF CALIFORNIA
10-9-26

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

SHEET TITLE:

MISCELLANEOUS DETAILS

DRAWN BY: AM	DATE: 9/18/2026
CHECKED BY: JA	REV #:
SCALE: NONE	PROJECT NUMBER: XXXXXX-00

SHEET NUMBER:

SDF-08

8 References

ASCE (2017). "ASCE/SEI 41-17, Seismic Evaluation and Retrofit of Existing Buildings." ASCE, Reston, Virgina.

Charney, F., and Marshall, J. (2006). "A comparison of the Krawinkler and Scissors models for including beam-column joint deformations in the analysis of moment-resisting steel frames." *Engineering Journal*, 43, 31-48.

Reynolds, M., and Uang, C.-M. (2019a). "Cyclic Testing of DuraFuse (DF) Moment Frame Connections for SMF and IMF Applications: Series E, F, and G Specimens." University of California San Diego, La Jolla, CA.

Reynolds, M., and Uang, C.-M. (2019b). "Cyclic Testing of DuraFuse (DF) Moment Frame Connections for SMF and IMF Applications: Series H Specimens." University of California San Diego, La Jolla, CA.

Lee H, and Uang, C.-M. (2021). "Cyclic Testing of DuraFuse (DF) Moment Frame Connections for SMF and IMF Applications: Series K Specimens." University of California San Diego, La Jolla, CA.

Lin, L. and Uang, C.-M. (2025). "Full-Scale Testing of DuraFuse for SMF Applications: Series L4 Specimens." University of California San Diego, La Jolla, CA.

Richards, P. (2019). "A repairable connection for earthquake-resisting moment frames." *Steel Construction*, 12(3), 191-197.

Richards, P. W. (2021). "Cyclic Hardening Factor for Replaceable Shear Fuse Connections." *Journal of Construction Steel Research*, 185, 106838.

Richards, P. W. (2022). "Cyclic behavior of DuraFuse Frames moment connections." *Engineering Journal*, 2022(2).

Richards, P. W., and Oh, S. S. (2019). "Cyclic Behavior of Replaceable Shear Fuse Connections for Steel Moment Frames." *Journal of Structural Engineering*, 145(12), 04019156.

Uang, C., and Fan, C. (2001). "Cyclic Stability Criteria for Steel Moment Connections with Reduced Beam Section." *Journal of Structural Engineering*, 127(9), 1021-1027.

HCAI PCS-0004 DATE: 10/09/2026 CALIFORNIA BUILDING CODE, 2025

9 Appendix A – Example Calculations

CALCULATION PACKAGE

HCAI PCS SUBMITTAL V4.0

Software Version 1.9.13



DURAFUSE FRAMES

THE RESILIENT SEISMIC SOLUTION

7/30/2026

XXXXXX-00

www.durafuseframes.com

TABLE OF CONTENTS

HCAI CALCULATIONS

1 GENERAL BUILDING INFORMATION

Calculation Package Description	1.1
Plan View & Frame Elevations	1.2

2 DURAFUSE FRAMES DESIGN INFORMATION

Demands from EOR	2.1
Dimension Diagram	2.2
Project Design Summary	2.3
Limit State Summary	2.4
Beam Local Buckling & Lateral Bracing Report	2.5

3 DURAFUSE FRAMES DETAILED CONNECTION DESIGN

DF101 Detailed Connection Design	3.1
DF102 Detailed Connection Design	3.2
DF103 Detailed Connection Design	3.3
DF201 Detailed Connection Design	3.4

4 ADDITIONAL EXAMPLE CONNECTIONS

DF150 Detailed Connection Design (in-plane drag load)	4.1
DF151 Detailed Connection Design (orthogonal drag load)	4.2
DF152 Detailed Connection Design (orthogonal cantilever beam)	4.3
DF153 Detailed Connection Design (Heavy Beam AISC 358-27 F2 Eq.)	4.4
DHF101 Detailed Connection Design (HSS column)	4.5
DF250 Detailed Connection Design (roof condition; RPZ and stiffness checks: Mpr<Mp)	4.6
DF251 Detailed Connection Design (in-plane cantilever beam)	4.7

5 NEW CONNECTIONS FOR PCS V3.0

DF101 Detailed Connection Design (W36X529 Beam)	5.1
DF102 Detailed Connection Design (W44X335 Beam)	5.2
DF103 Detailed Connection Design (BUW44X17X2.875 Beam)	5.3
DF201 Detailed Connection Design (BUW44X17X2.875 Beam)	5.4

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kuldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

DURAFUSE FRAMES CONNECTION CALCULATION PACKAGE

HCAI PCS SUBMITTAL V4.0

Software Version 1.9.13

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

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

DuraFuse Design Concept Explained

The DuraFuse connection is a unique capacity based design where the controlling component of the connection is always the fuse plate. The fuse plate is an inelastic mechanism where its ultimate strength is controlled by shear yielding of the plate. The ultimate strength of this mechanism becomes the demand for all other connection components, which are required to remain elastic (i.e., top plate, external continuity plates, shear tab, welds, etc.).

The fuse plate is designed so that its full strain-hardened strength develops the plastic moment capacity of the beam at the column face. This expected connection capacity is referred to as Mpr. The quantity Vfe represents the ultimate shear force in one side of the fuse plate required to develop Mpr. This force, Vfe, is used as the demand for designing the other connection components as well as for checking the remaining limit states of the fuse plate.

The ultimate strength of the fuse regions is based on developing Vfe on each side of the fuse plate. Due to practical fabrication limitations, the calculated fuse depth is rounded to the nearest 1/8" smaller than the theoretical required depth. This ensures that the fuse plate mechanism does not exceed the intended strength (Vfe) and therefore remains the yielding mechanism. In the design output, this check is not expressed as a DCR because it is not a traditional DCR; it is instead a verification step that the connection satisfies the requirements established during the

1 GENERAL BUILDING INFORMATION

Calculation Package Description	1.1
Plan View & Frame Elevations	1.2

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

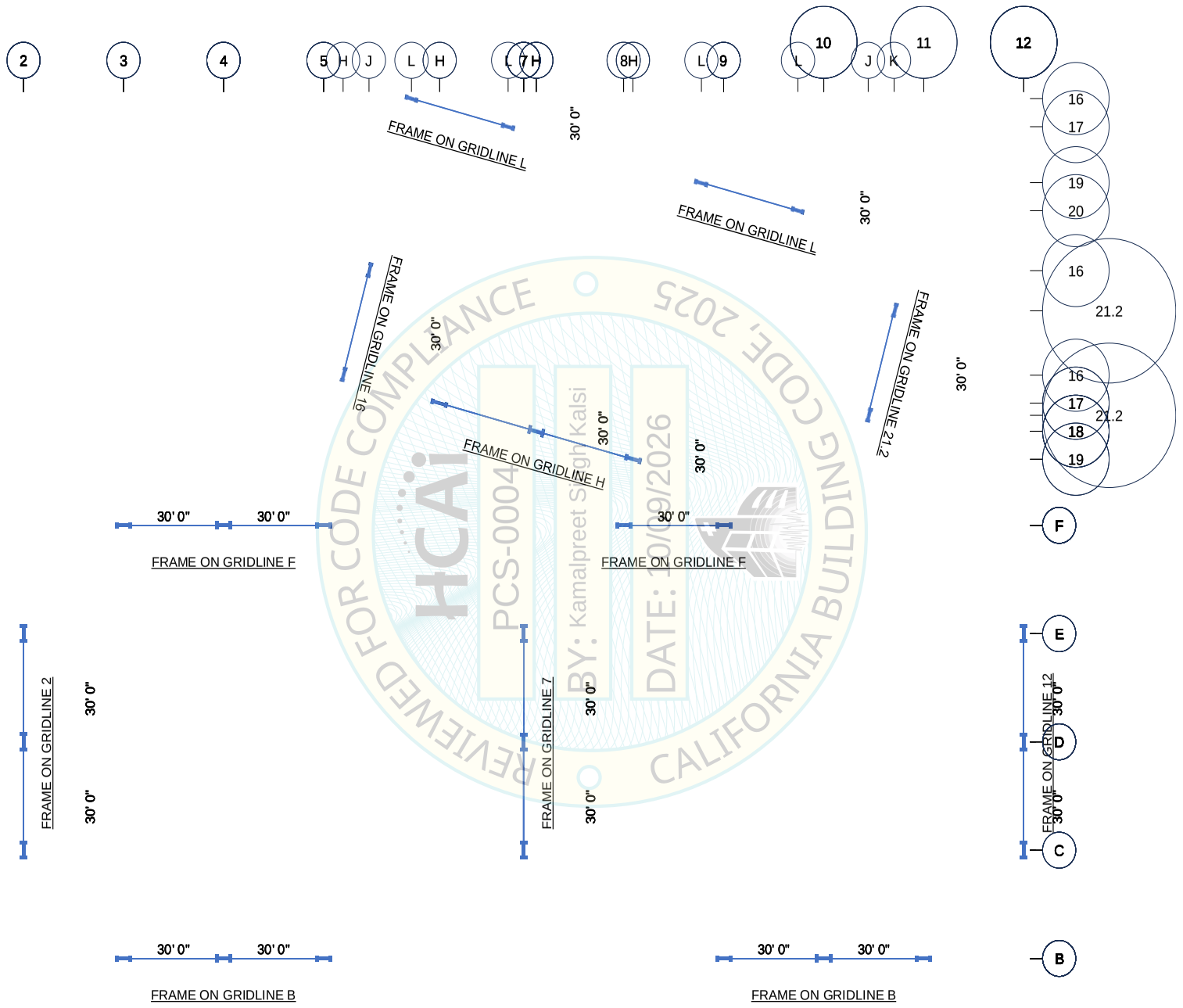
DuraFuse Frames Elevations

Project Name: HCAI CALCULATIONS

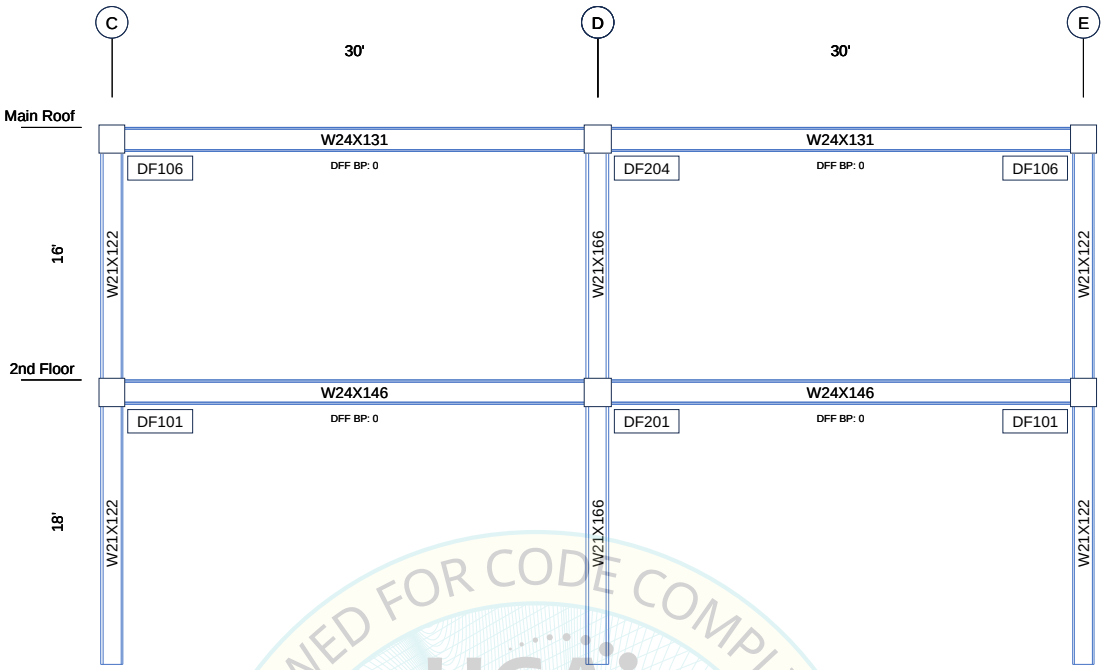
Project Number: XXXXXX-00

Date: 12/17/2024

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



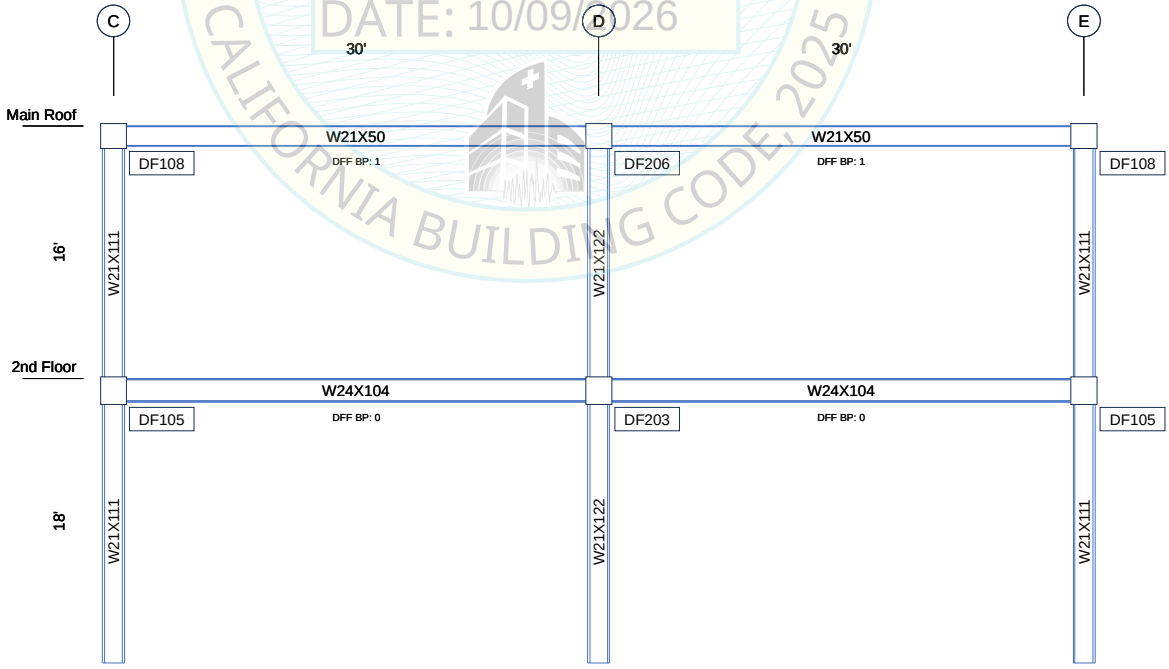
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026



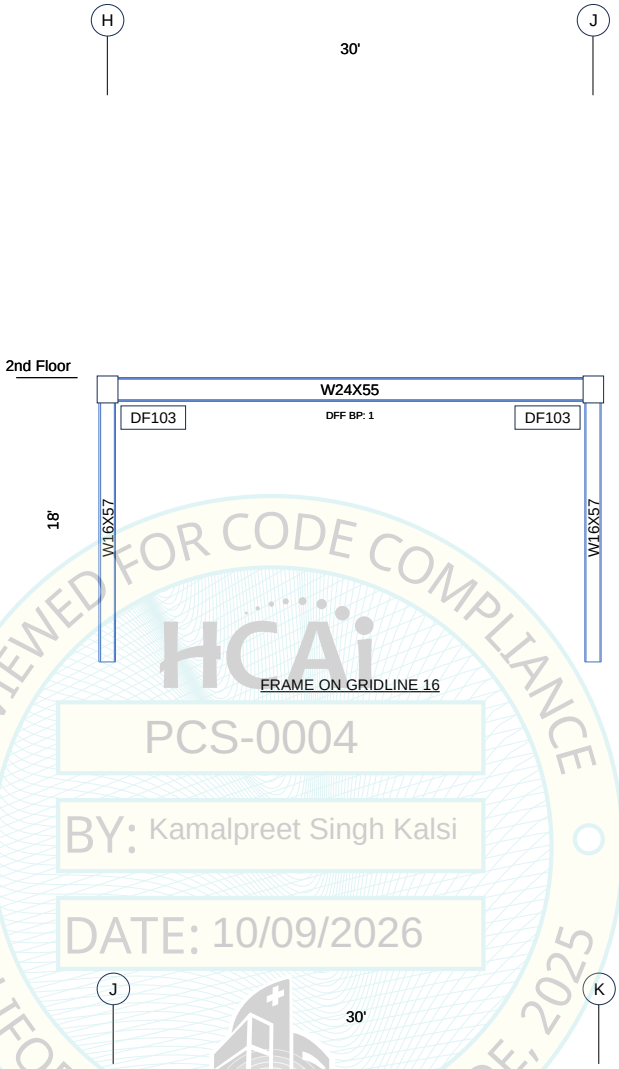
FRAME ON GRIDLINE 2

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

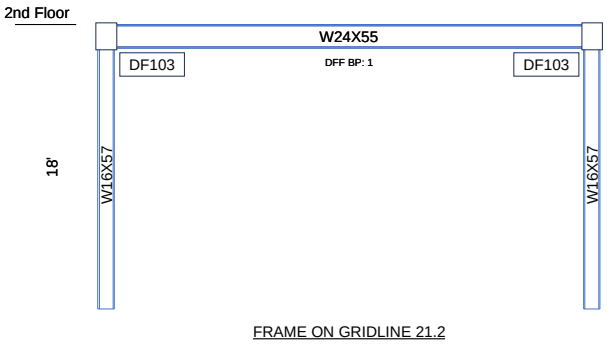
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026



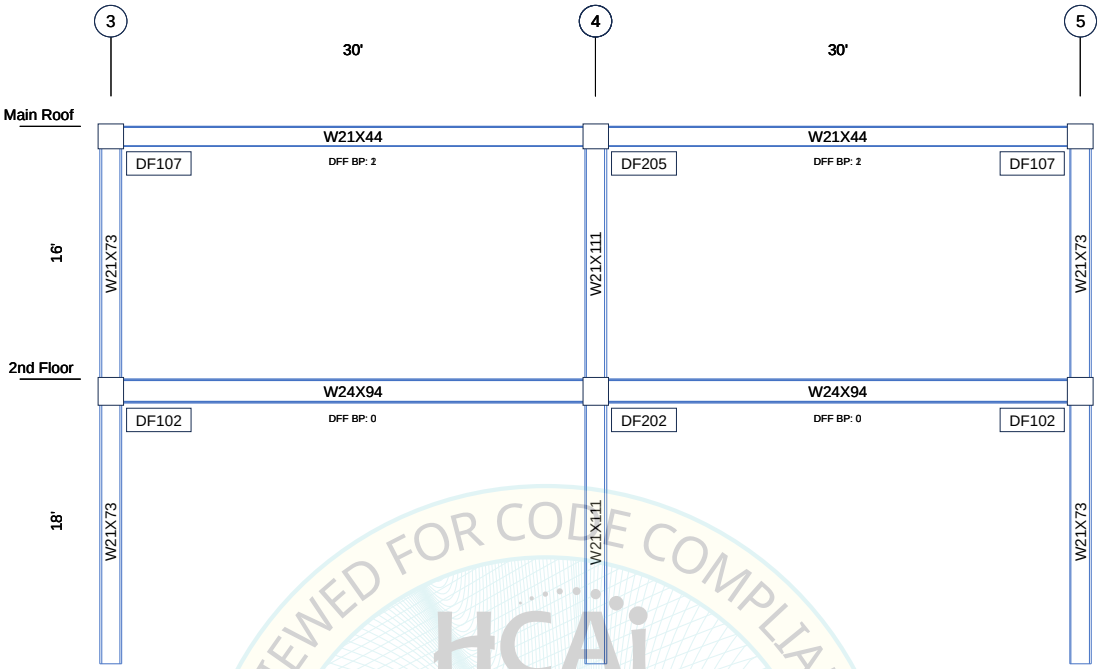
FRAME ON GRIDLINE 12



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

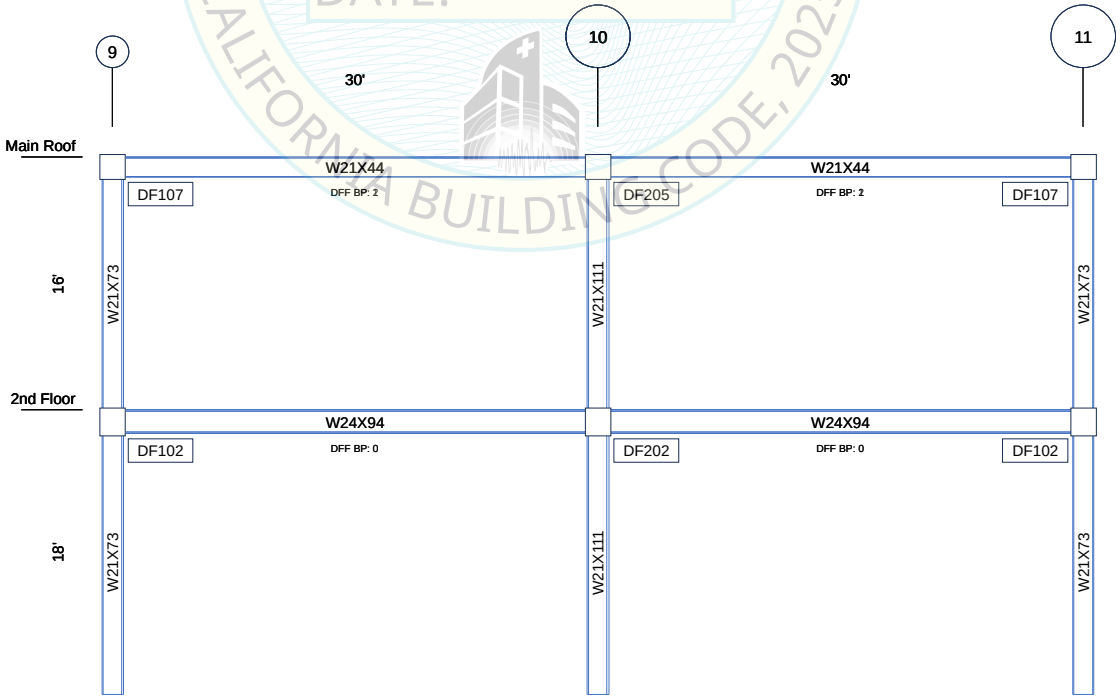


FRAME ON GRIDLINE 21.2

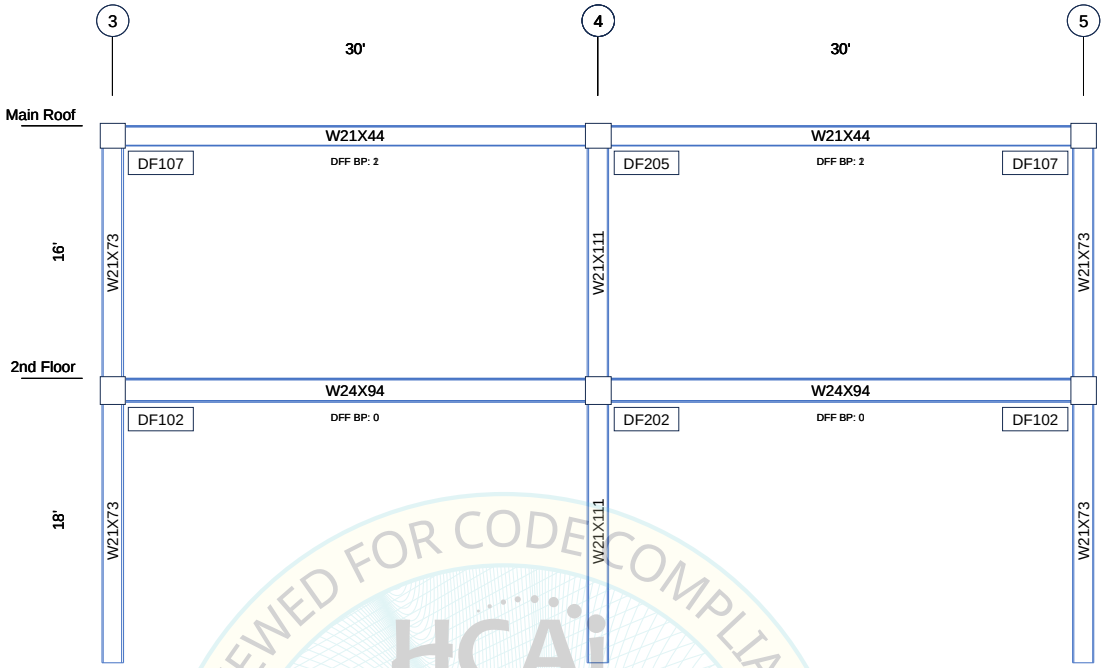


FRAME ON GRIDLINE B

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

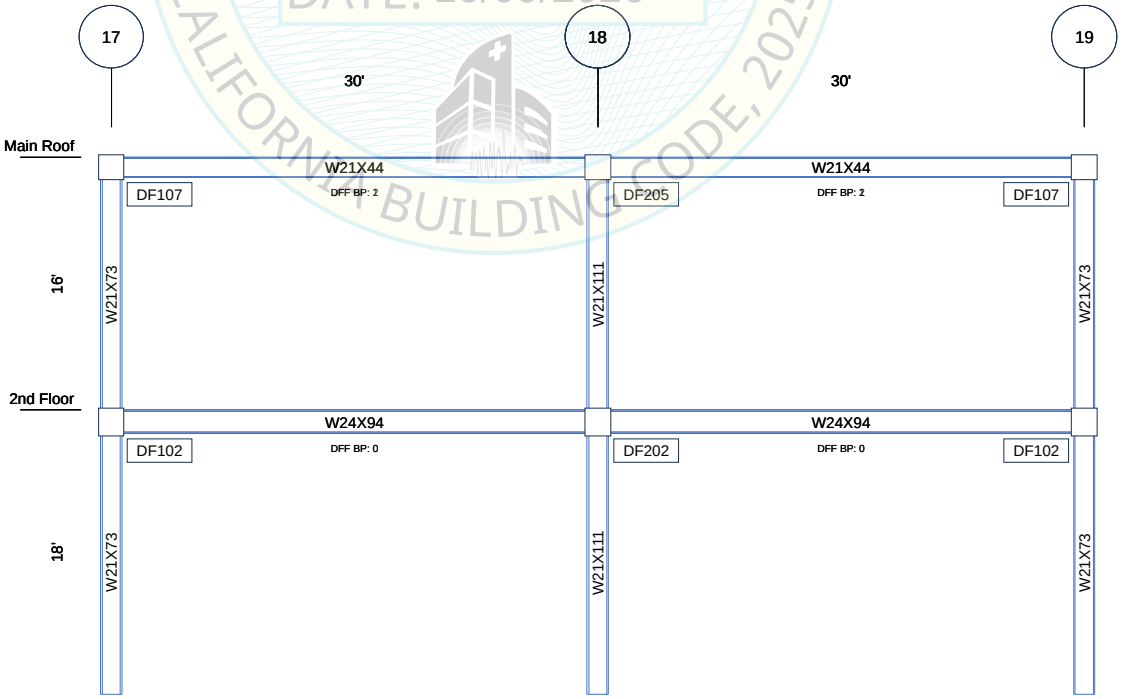


FRAME ON GRIDLINE B

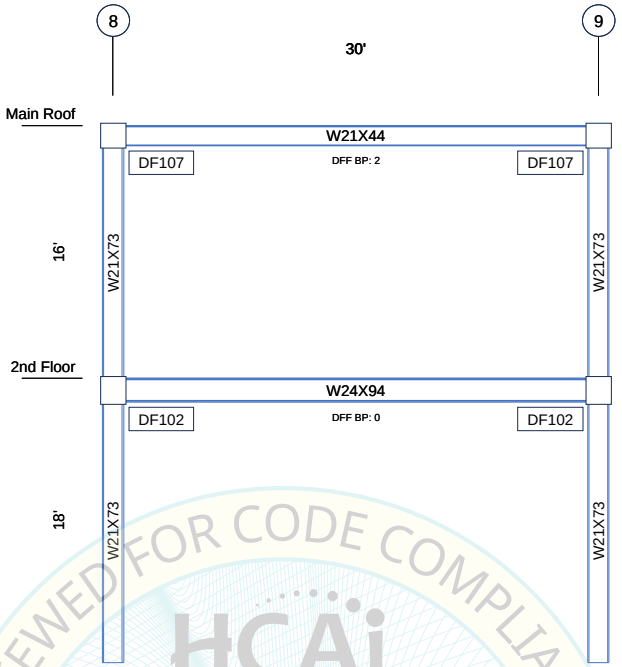


FRAME ON GRIDLINE F

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



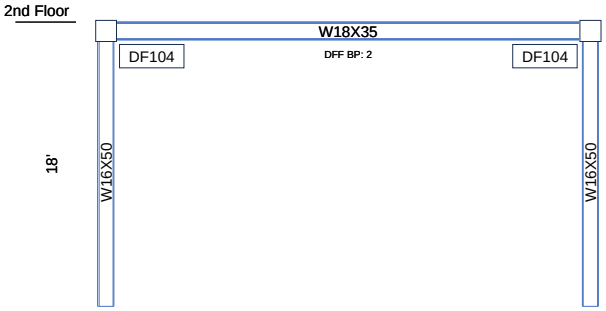
FRAME ON GRIDLINE H



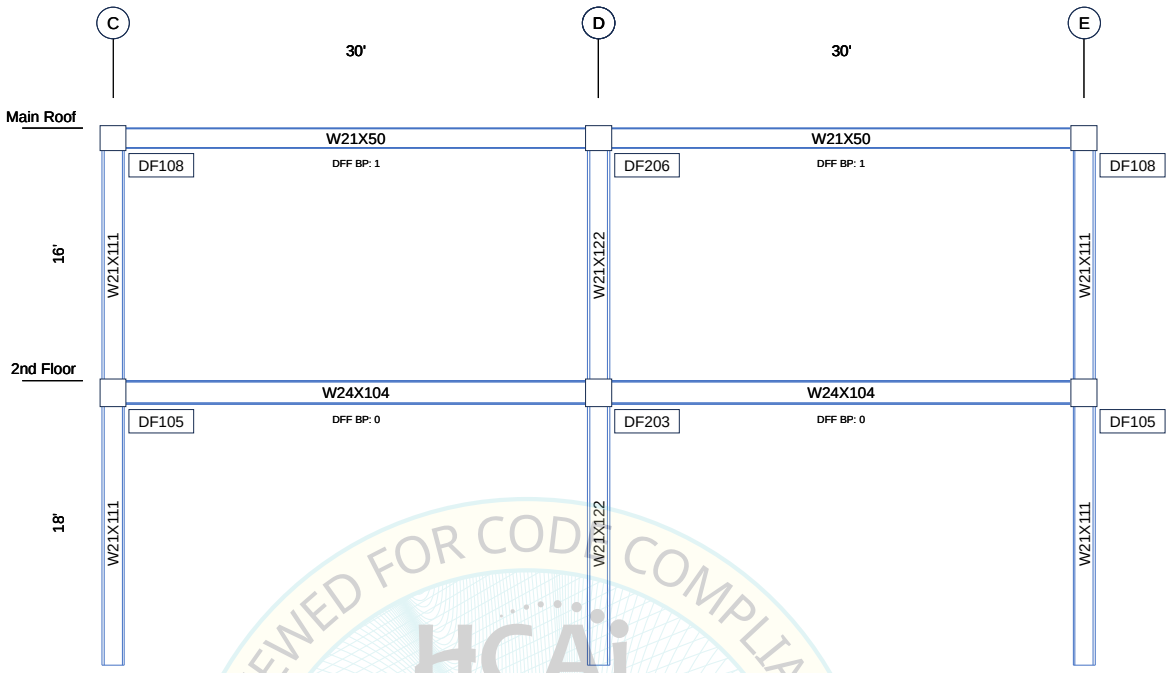
FRAME ON GRIDLINE F

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025





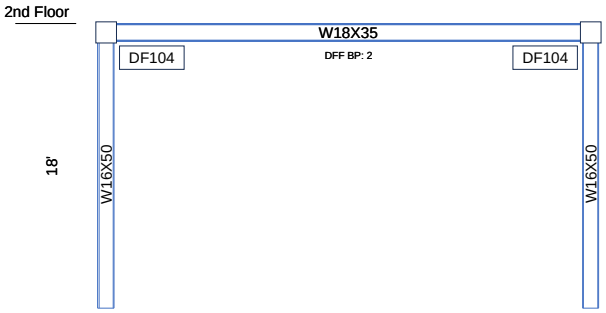
FRAME ON GRIDLINE L



FRAME ON GRIDLINE 7

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025





FRAME ON GRIDLINE L

2 DURAFUSE FRAMES DESIGN INFORMATION

Demands from EOR	2.1
Dimension Diagram	2.2
Project Design Summary	2.3
Limit State Summary	2.4
Beam Local Buckling & Lateral Bracing Report	2.5

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DEMANDS FROM EOR

HCAI PCS SUBMITTAL V4.0

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 labeled "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 + 0.5Lr or 0.3S or 0.5R
Load Combination 3: 1.2D + 1.6Lr or 1.0S or 1.6R + (L or 0.5W)
Load Combination 4: 1.2D + 1.0(W or WT) + LL + 0.5LLr or 0.3S or 0.5R
Load Combination 5: 0.9D + 1.0(W or WT)
Load Combination 6: 1.2D + E_v + E_h + L + 0.15S, where $E_v = 0.2S_{DS}$*D, $E_h = \rho Q_E$
Load Combination 7: 0.9D - E_v + E_h
Load Combination 6 (for column axial loads): 1.2D + E_v + E_{mh} + L + 0.15S, where $E_{mh} = \Omega_0 Q_E$
Load Combination 7 (for column axial loads): 0.9D - E_v + E_{mh}, where $E_{mh} = \Omega_0 Q_E$
* Collector design accounts for EOR demand loads.

GRAVITY & EARTHQUAKE DEMANDS			LC 1,2,3	LC 1,2,3	LC 1,2,3	LC 1,2,3	LC 6,7	LC 6,7	LC 1,2,3,6,7	
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**		**380.5**	**565.8**		
M102	2nd Floor	2 - E	40.1	40.1	86.2		374.7	563.5		
DF101			**43.8**	**43.8**	**90.7**		**380.5**	**565.8**		**5.0**
M103	2nd Floor	3 - B	**58.1**	**50.3**	46.5		220.5	437.9		
M104	2nd Floor	3 - F	48.8	45.8	40.8		215.3	423.3		
M105	2nd Floor	5 - B	50.3	50.3	**50.3**		205.6	420.2		
M106	2nd Floor	5 - F	50.1	50.3	44.1		**234.1**	432.2		
M110	2nd Floor	H - 17	15.7	14.7	14.7		153.8	300.0		
M114	2nd Floor	8 - F	51.0	52.1	48.1		222.9	**466.5**		
M115	2nd Floor	H - 19	15.7	15.7	15.7		142.7	293.7		
M117	2nd Floor	9 - B	49.4	49.4	49.4		202.5	405.9		
M118	2nd Floor	9 - F	51.0	46.5	42.4		203.5	463.6		
M122	2nd Floor	11 - B	50.1	50.1	50.1		206.6	415.7		
DF102			**58.1**	**50.3**	**50.3**		**234.1**	**466.5**		**6.0**
M107	2nd Floor	H - 16	5.4		14.7		**127.3**	68.3		
M108	2nd Floor	J - 16	5.4	23.6	22.6		53.6	**71.1**		
M120	2nd Floor	J - 21.2	**31.5**		**31.5**		37.9	60.3		
M121	2nd Floor	K - 21.2	4.3		26.3		34.1	59.7		
DF103			**31.5**		**31.5**		**127.3**	**71.1**		**7.0**
M109	2nd Floor	L - 16	**27.6**		**27.6**		**38.3**	62.5		
M111	2nd Floor	L - 17	8.4	13.2	4.6		26.6	62.1		
M116	2nd Floor	L - 19	8.4	13.3	4.6		27.0	**64.4**		
M119	2nd Floor	L - 20	8.4	8.3	4.6		23.5	63.5		
DF104			**27.6**		**27.6**		**38.3**	**64.4**		**8.0**
M112	2nd Floor	7 - C	**45.9**	**42.4**	**89.5**		335.4	459.7		
M113	2nd Floor	7 - E	45.9	45.9	89.5		**335.6**	**459.7**		
M123	2nd Floor	12 - C	27.8	26.8	89.5		228.5	394.8		
M124	2nd Floor	12 - E	27.8	27.8	89.5		228.9	397.8		
DF105			**45.9**	**42.4**	**89.5**		**335.6**	**459.7**		**9.0**

REVIEWED FOR CODE COMPLIANCE HCAI PCS-0004 BY: Kamalpreet Singh Kalsi DATE: 10/09/2026 CALIFORNIA BUILDING CODE 2022

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	75.0	**226.6**		
M126	Main Roof	2 - E	**7.3**	7.8	**36.7**	**146.9**	**82.2**	224.3		
DF106			**7.3**	**4.1**	**36.7**	**146.9**	**82.2**	**226.6**		**5.0**
M127	Main Roof	3 - B	16.4		14.4		41.5	136.7		
M128	Main Roof	3 - F	25.2	33.5	7.8		62.2	167.0		
M129	Main Roof	5 - B	16.2	16.2	16.2		42.8	137.5		
M130	Main Roof	5 - F	**34.1**	33.9	7.3		69.2	**183.6**		
M131	Main Roof	H - 17	29.9	19.2	**29.9**	**239.1**	**72.3**	155.3		
M134	Main Roof	8 - F	18.5	19.6	7.3		47.4	146.4		
M135	Main Roof	H - 19	12.5	28.1	28.1	224.8	61.4	140.6		
M136	Main Roof	9 - B	14.8	12.7	12.7		42.3	130.6		
M137	Main Roof	9 - F	16.9	17.1	7.3		38.4	140.9		
M138	Main Roof	11 - B	14.8	14.8	14.8		44.6	127.4		
DF107			**34.1**		**29.9**	**239.1**	**72.3**	**183.6**		**6.0**
M132	Main Roof	7 - C	**8.7**	**7.3**	22.7		66.7	136.6		
M133	Main Roof	7 - E	8.7	8.7	**24.2**		**70.1**	**136.8**		
M139	Main Roof	12 - C	7.3	7.2	23.0		51.2	122.1		
M140	Main Roof	12 - E	7.3	7.3	23.0		51.4	124.7		
DF108			**8.7**	**7.3**	**24.2**		**70.1**	**136.8**		**7.0**
M201	2nd Floor	2 - D	**40.1**	**40.1**	**89.5**		**262.4**	**629.8**	**625.8**	
DF201			**40.1**	**40.1**	**89.5**		**262.4**	**629.8**	**625.8**	**8.0**
M202	2nd Floor	4 - B	**52.7**	**52.7**	**52.7**		156.7	**535.1**	**529.0**	
M203	2nd Floor	4 - F	49.5	50.1	44.1		**181.8**	519.2	518.9	
M205	2nd Floor	H - 18	15.7	15.7	15.7		82.9	311.5	314.5	
M206	2nd Floor	10 - B	45.0	50.1	45.0		140.6	497.0	517.0	
DF202			**52.7**	**52.7**	**52.7**		**181.8**	**535.1**	**529.0**	**9.0**
M204	2nd Floor	7 - D	**45.9**	**45.9**	**89.5**		**258.5**	**492.1**	**491.8**	
M207	2nd Floor	12 - D	27.8	27.8	89.5		149.1	402.9	405.3	
DF203			**45.9**	**45.9**	**89.5**		**258.5**	**492.1**	**491.8**	**5.0**
M208	Main Roof	2 - D	**7.8**	**7.3**	**36.6**	**146.4**	**45.1**	**222.4**	**217.8**	
DF204			**7.8**	**7.3**	**36.6**	**146.4**	**45.1**	**222.4**	**217.8**	**6.0**
M209	Main Roof	4 - B	16.4	17.1	**7.3**		34.0	145.0	152.1	
M210	Main Roof	4 - F	**39.7**	27.0	7.3		**62.2**	**243.2**	**216.3**	
M212	Main Roof	H - 18	15.8	**28.7**	28.7	229.6	48.1	169.7	161.4	
M213	Main Roof	10 - B	14.8	14.8	14.8		31.1	137.1	140.4	
DF205			**39.7**	**28.7**	**7.3**		**62.2**	**243.2**	**216.3**	**7.0**
M211	Main Roof	7 - D	**8.7**	**8.7**	**24.6**		**46.2**	**132.7**	**132.9**	
M214	Main Roof	12 - D	7.3	7.3	23.1		31.9	126.3	124.3	
DF206			**8.7**	**8.7**	**24.6**		**46.2**	**132.7**	**132.9**	**8.0**

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

WIND DEMANDS			LC 4,5	LC 4,5	LC 4,5	LC 4,5
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) *
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**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

WIND DEMANDS (Continued)

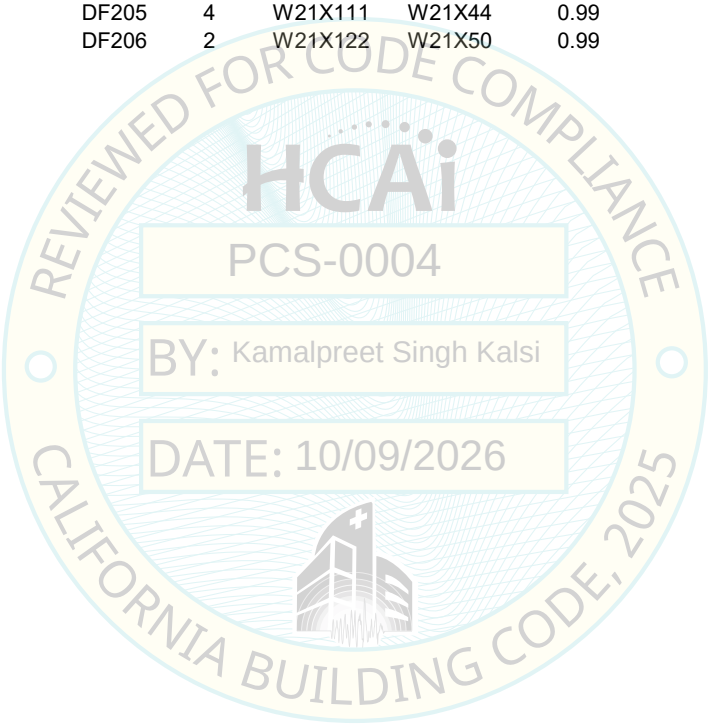
Model ID **DF ID**	Framing Level	Grid	Column Axial Compression (kips)	DFF Beam 1 Moment @ Face of Column (k-ft)	Beam 2 Moment @ Face of Column (SMF or Gravity - Cantilever beam) (k-ft)	In-Plane Collector Force (From EOR) (kips) *
M202	2nd Floor	4 - B	132.4	**232.1**		
M203	2nd Floor	4 - F	**153.9**	217.4		
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**		

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

PROJECT SUMMARY

HCAI PCS SUBMITTAL V4.0

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

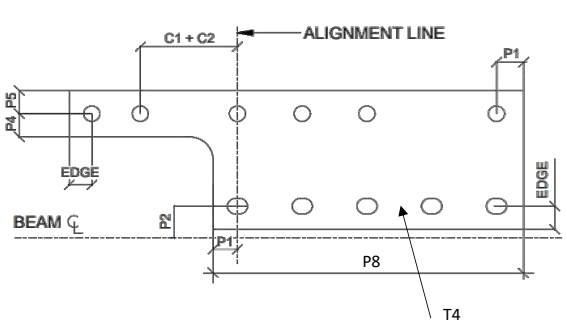


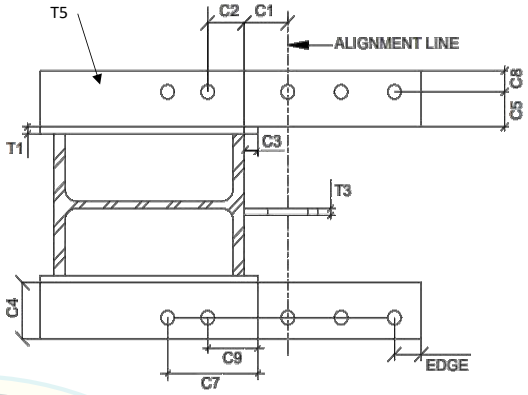
* The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

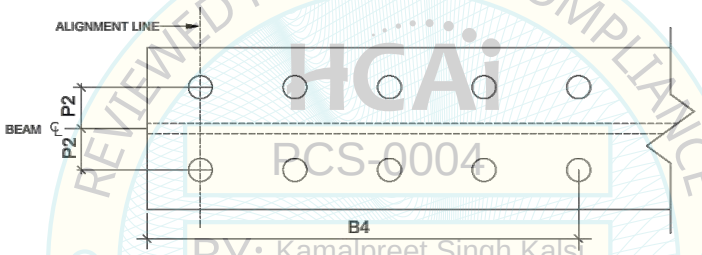
See following pages for more information

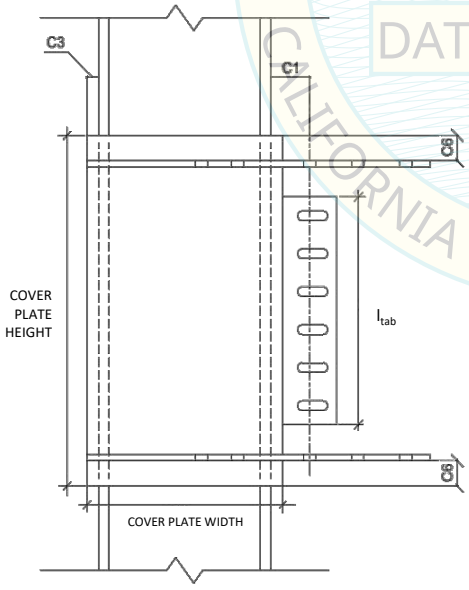
DIMENSION DIAGRAMS

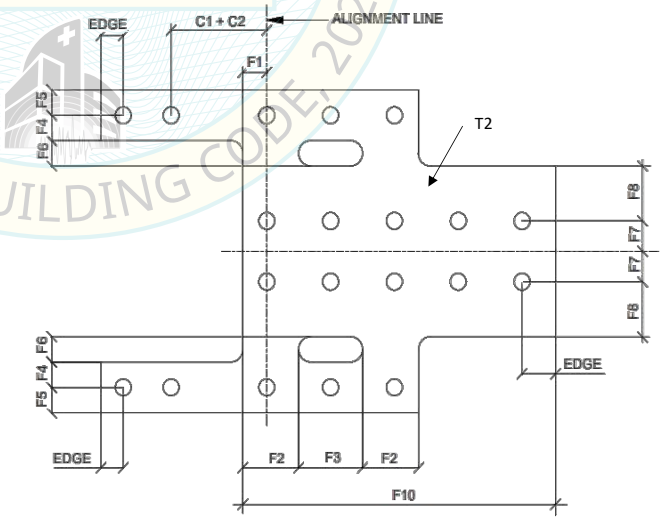
HCAI PCS SUBMITTAL V4.0







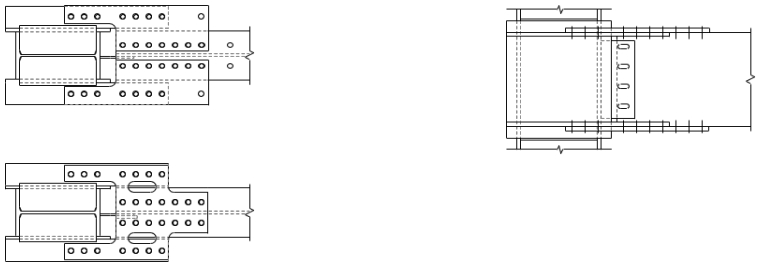




REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

DF101 Limit State Summary

Column: W21X122 - Beam: W24X146



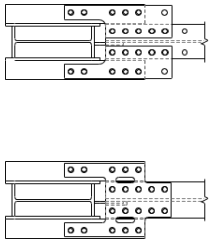
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 5.9	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 6.4	OK
	Column-beam moment ratio	ΣM^*_{pc} = 3.06E+04	ΣM^*_{pb} = 2.27E+04	OK
2	Maximum probable moment	M_u = 6713 k-in	M_{pr} = 20900 k-in	0.32
3	Cover plate shear yielding	R_u = 528.4 kips	ϕR_n = 805.9 kips	0.66
	Cover plate thickness	t_{cp} = 0.875 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 = 407.1 kips	ϕR_n = 438.4 kips	0.93
	Bolt slip for wind load	R_u = 20.2 kips	ϕR_n = 161.4 kips	0.13
6	Alignment line location, C1	$C1_{req}$ = 5.8 in	C1 = 5.7 in	OK
7	Weld 1 strength	r_u = 9.3 kip/in	ϕr_n = 15.3 kip/in	0.61
8	Weld 2 strength	r_u = 20.6 kip/in	ϕr_n = 22.3 kip/in	0.93
9	External continuity plate rupture: Mode 1	P_u = 232.7 kips	ϕP_n = 243.4 kips	0.96
	External continuity plate rupture: Mode 2	P_u = 290.8 kips	ϕP_n = 295.4 kips	0.98
	External continuity plate rupture: Mode 3	P_u = 349 kips	ϕP_n = 451.6 kips	0.77
	External continuity plate rupture: Mode 4	R_u = 290.8 kips	ϕR_n = 328.8 kips	0.88
	External continuity plate rupture: Mode 5	R_u = 349 kips	ϕR_n = 414.2 kips	0.84
10	Beam shear	V_u = 167.4 kips	ϕV_n = 481.6 kips	0.35
11	Beam block shear	R_u = 814.3 kips	ϕR_n = 1174.3 kips	0.69
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	0.72
	Shear tab weld failure	r_u = 28.7 kips	ϕr_n = 30.6 kips	0.94
15	Top plate for shear yielding	R_u = 407.1 kips	ϕR_n = 801.6 kips	0.51
	Top plate for shear rupture	R_u = 407.1 kips	ϕR_n = 436 kips	0.93
16	Top plate for tensile rupture in extensions	P_u = 174.5 kips	ϕP_n = 178.2 kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.4	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 346.8 kips	ϕP_n = 551.8 kips	0.63
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 346.8 kips	ϕP_n = 519.9 kips	0.67
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 3.4 in**	**F2 = 3.2 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 0.8	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 346.8 kips	ϕR_n = 430.3 kips	0.81
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 346.8 kips	ϕR_n = 362 kips	0.96
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	R_u = 192.7 kips	ϕR_n = 296.7 kips	0.65
	Fully Restrained (FR) assumption	K_{req} = 6.64E+06	K_S = 1.11E+07	0.60
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

DF102 Limit State Summary

Column: W21X73 - Beam: W24X94



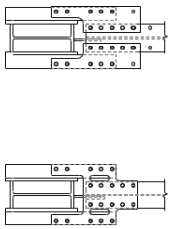
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 5.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5.6	OK
	Column-beam moment ratio	ΣM^*_{pc} = 1.70E+04	ΣM^*_{pb} = 1.41E+04	OK
2	Maximum probable moment	M_u = 5374 k-in	M_{pr} = 12700 k-in	0.42
3	Cover plate shear yielding	R_u = 326.5 kips	ϕR_n = 568.1 kips	0.57
	Cover plate thickness	t_{cp} = 0.625 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 = 254.6 kips	ϕR_n = 313.1 kips	0.81
	Bolt slip for wind load	R_u = 37.9 kips	ϕR_n = 115.3 kips	0.33
6	Alignment line location, C1	$C1_{req}$ = 4.6 in	C1 = 4.6 in	OK
7	Weld 1 strength	r_u = 12.6 kip/in	ϕr_n = 13.7 kip/in	0.92
8	Weld 2 strength	r_u = 18.4 kip/in	ϕr_n = 19.5 kip/in	0.94
9	External continuity plate rupture: Mode 1	P_u = 152.8 kips	ϕP_n = 157.3 kips	0.97
	External continuity plate rupture: Mode 2	P_u = 203.7 kips	ϕP_n = 217.8 kips	0.94
	External continuity plate rupture: Mode 3	P_u = 254.6 kips	ϕP_n = 357.2 kips	0.71
	External continuity plate rupture: Mode 4	R_u = 203.7 kips	ϕR_n = 238.5 kips	0.85
	External continuity plate rupture: Mode 5	R_u = 254.6 kips	ϕR_n = 305 kips	0.83
10	Beam shear	V_u = 133.1 kips	ϕV_n = 375.4 kips	0.35
11	Beam block shear	R_u = 509.2 kips	ϕR_n = 568.2 kips	0.90
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 = 22.6 kips	ϕr_n = 25.1 kips	0.90
15	Top plate for shear yielding	R_u = 254.6 kips	ϕR_n = 510 kips	0.50
	Top plate for shear rupture	R_u = 254.6 kips	ϕR_n = 277.9 kips	0.92
16	Top plate for tensile rupture in extensions	P_u = 101.8 kips	ϕP_n = 103.6 kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 1.7	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 201.3 kips	ϕP_n = 346.5 kips	0.58
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 201.3 kips	ϕP_n = 296.9 kips	0.68
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 2.6 in**	**F2 = 2.6 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 0.6	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 201.3 kips	ϕR_n = 275.6 kips	0.73
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 201.3 kips	ϕR_n = 204.8 kips	0.98
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} = 3.91E+06	K_S = 1.13E+07	0.35
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

DF103 Limit State Summary

Column: W16X57 - Beam: W24X55



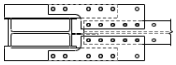
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 6.9	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	M_u = 832 k-in	M_{pr} = 5895 k-in	0.14
3	Cover plate shear yielding	R_u = 198.1 kips	ϕR_n = 681.8 kips	0.29
	Cover plate thickness	t_{cp} = 0.875 in	t_{req} = 0.417 in	OK
4	Beam net section check	M_{pe} = 7370 k-in	M_{fr} = 8017 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 125.2 kips	ϕR_n = 139.2 kips	0.90
	Bolt slip for wind load	R_u = 3 kips	ϕR_n = 50.4 kips	0.06
6	Alignment line location, C1	$C1_{req}$ = 4.2 in	C1 = 2.5 in	OK
7	Weld 1 strength	r_u = 7.5 kip/in	ϕr_n = 7.9 kip/in	0.95
8	Weld 2 strength	r_u = 10.3 kip/in	ϕr_n = 11.1 kip/in	0.93
9	External continuity plate rupture: Mode 1	P_u = 75.1 kips	ϕP_n = 76.2 kips	0.99
	External continuity plate rupture: Mode 2	P_u = 100.1 kips	ϕP_n = 101.8 kips	0.98
	External continuity plate rupture: Mode 3	P_u = 125.2 kips	ϕP_n = 178.6 kips	0.70
	External continuity plate rupture: Mode 4	R_u = 100.1 kips	ϕR_n = 114.8 kips	0.87
	External continuity plate rupture: Mode 5	R_u = 125.2 kips	ϕR_n = 153.5 kips	0.82
10	Beam shear	V_u = 65.9 kips	ϕV_n = 279.7 kips	0.24
11	Beam block shear	R_u = 250.3 kips	ϕR_n = 268 kips	0.93
12	Web bolt shear	R_u = 65.9 kips	ϕR_n = 111.3 kips	0.59
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.04
	Shear tab weld failure	r_u = 11.3 kips	ϕr_n = 13.9 kips	0.81
15	Top plate for shear yielding	R_u = 125.2 kips	ϕR_n = 276.6 kips	0.45
	Top plate for shear rupture	R_u = 125.2 kips	ϕR_n = 178.2 kips	0.70
16	Top plate for tensile rupture in extensions	P_u = 50.1 kips	ϕP_n = 53.3 kips	0.94
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2.5 in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.4	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 97.3 kips	ϕP_n = 207.3 kips	0.47
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 97.3 kips	ϕP_n = 196.3 kips	0.50
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 1.8 in**	**F2 = 1.8 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 0.8	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 97.3 kips	ϕR_n = 147.7 kips	0.66
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 97.3 kips	ϕR_n = 118.8 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} = 1.96E+06	K_S = 5.45E+06	0.36
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

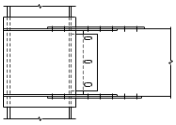
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

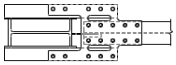
** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

DF104 Limit State Summary

Column: W16X50 - Beam: W18X35







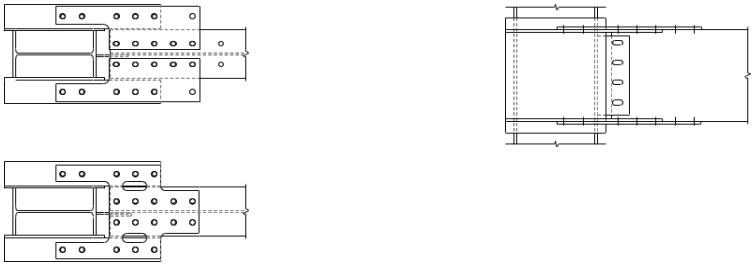
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 7.1	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5.6	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	M_u = 752 k-in	M_{pr} = 3325 k-in	0.23
3	Cover plate shear yielding	R_u = 118 kips	ϕR_n = 227.2 kips	0.52
	Cover plate thickness	t_{cp} = 0.375 in	t_{req} = 0.354 in	OK
4	Beam net section check	M_{pe} = 3658 k-in	M_{fr} = 3770 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 95.3 kips	ϕR_n = 139.2 kips	0.69
	Bolt slip for wind load	R_u = 8.5 kips	ϕR_n = 50.4 kips	0.17
6	Alignment line location, C1	$C1_{req}$ = 4.2 in	C1 = 2.1 in	OK
7	Weld 1 strength	r_u = 6.4 kip/in	ϕr_n = 7.9 kip/in	0.81
8	Weld 2 strength	r_u = 9.2 kip/in	ϕr_n = 11.1 kip/in	0.83
9	External continuity plate rupture: Mode 1	P_u = 57.2 kips	ϕP_n = 60.9 kips	0.94
	External continuity plate rupture: Mode 2	P_u = 76.3 kips	ϕP_n = 81.4 kips	0.94
	External continuity plate rupture: Mode 3	P_u = 95.3 kips	ϕP_n = 154.4 kips	0.62
	External continuity plate rupture: Mode 4	R_u = 76.3 kips	ϕR_n = 91.9 kips	0.83
	External continuity plate rupture: Mode 5	R_u = 95.3 kips	ϕR_n = 127 kips	0.75
10	Beam shear	V_u = 46.9 kips	ϕV_n = 159.3 kips	0.29
11	Beam block shear	R_u = 190.7 kips	ϕR_n = 262.6 kips	0.73
12	Web bolt shear	R_u = 46.9 kips	ϕR_n = 79 kips	0.59
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.08
	Shear tab weld failure	r_u = 13.7 kips	ϕr_n = 16.7 kips	0.82
15	Top plate for shear yielding	R_u = 95.3 kips	ϕR_n = 243.8 kips	0.39
	Top plate for shear rupture	R_u = 95.3 kips	ϕR_n = 164.5 kips	0.58
16	Top plate for tensile rupture in extensions	P_u = 38.1 kips	ϕP_n = 39.6 kips	0.96
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2 in	OK
18	Fuse plate width-thickness ratio of yielding 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 = 126.4 kips	0.58
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 1.7 in**	**F2 = 1.6 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 0.9	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 73.1 kips	ϕR_n = 118.1 kips	0.62
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 73.1 kips	ϕR_n = 95.1 kips	0.77
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} = 7.39E+05	K_S = 2.41E+06	0.31
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

DF105 Limit State Summary

Column: W21X111 - Beam: W24X104



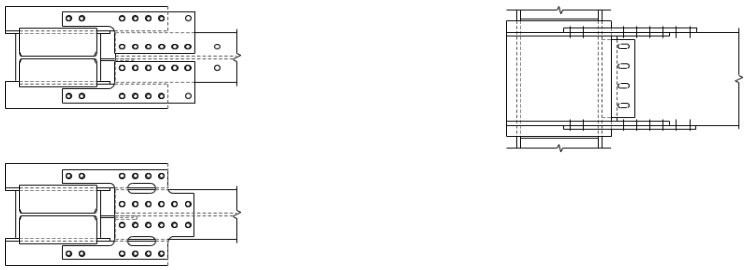
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 8.5	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 7	OK
	Column-beam moment ratio	ΣM^*_{pc} = 2.80E+04	ΣM^*_{pb} = 1.59E+04	OK
2	Maximum probable moment	M_u = 5428 k-in	M_{pr} = 14450 k-in	0.38
3	Cover plate shear yielding	R_u = 383.4 kips	ϕR_n = 564.4 kips	0.68
	Cover plate thickness	t_{cp} = 0.625 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 = 295.2 kips	ϕR_n = 313.1 kips	0.94
	Bolt slip for wind load	R_u = 22 kips	ϕR_n = 115.3 kips	0.19
6	Alignment line location, C1	$C1_{req}$ = 5.1 in	C1 = 5.1 in	OK
7	Weld 1 strength	r_u = 15 kip/in	ϕr_n = 16 kip/in	0.94
8	Weld 2 strength	r_u = 18.1 kip/in	ϕr_n = 19.5 kip/in	0.93
9	External continuity plate rupture: Mode 1	P_u = 177.1 kips	ϕP_n = 180.5 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 236.2 kips	ϕP_n = 277.2 kips	0.85
	External continuity plate rupture: Mode 3	P_u = 295.2 kips	ϕP_n = 510.7 kips	0.58
	External continuity plate rupture: Mode 4	R_u = 236.2 kips	ϕR_n = 277.6 kips	0.85
	External continuity plate rupture: Mode 5	R_u = 295.2 kips	ϕR_n = 359.8 kips	0.82
10	Beam shear	V_u = 131.3 kips	ϕV_n = 361.5 kips	0.36
11	Beam block shear	R_u = 590.5 kips	ϕR_n = 886.6 kips	0.67
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 = 23.5 kips	ϕr_n = 25.1 kips	0.94
15	Top plate for shear yielding	R_u = 295.2 kips	ϕR_n = 523.1 kips	0.56
	Top plate for shear rupture	R_u = 295.2 kips	ϕR_n = 345.5 kips	0.85
16	Top plate for tensile rupture in extensions	P_u = 118.1 kips	ϕP_n = 118.8 kips	0.99
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0.5 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 3.2	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 232.6 kips	ϕP_n = 372.9 kips	0.62
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 232.6 kips	ϕP_n = 353.2 kips	0.66
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 3.6 in**	**F2 = 3.5 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 0.7	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 232.6 kips	ϕR_n = 286.9 kips	0.81
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 232.6 kips	ϕR_n = 241.3 kips	0.96
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} = 4.49E+06	K_S = 8.09E+06	0.56
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

DF106 Limit State Summary

Column: W21X122 - Beam: W24X131



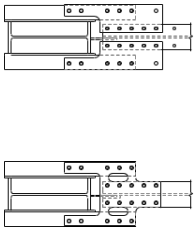
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 6.7	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 6.4	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	M_u = 2709 k-in	M_{pr} = 18500 k-in	0.15
3	Cover plate shear yielding	R_u = 446.1 kips	ϕR_n = 596.2 kips	0.75
	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 = 364.4 kips	ϕR_n = 375.7 kips	0.97
	Bolt slip for wind load	R_u = 6 kips	ϕR_n = 138.3 kips	0.04
6	Alignment line location, C1	$C1_{req}$ = 5.6 in	C1 = 5.6 in	OK
7	Weld 1 strength	r_u = 9.8 kip/in	ϕr_n = 16.8 kip/in	0.58
8	Weld 2 strength	r_u = 19.9 kip/in	ϕr_n = 22.3 kip/in	0.89
9	External continuity plate rupture: Mode 1	P_u = 242.9 kips	ϕP_n = 246.8 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 303.6 kips	ϕP_n = 443.9 kips	0.68
	External continuity plate rupture: Mode 3	P_u = 364.4 kips	ϕP_n = 670.1 kips	0.54
	External continuity plate rupture: Mode 4	R_u = 303.6 kips	ϕR_n = 409.9 kips	0.74
	External continuity plate rupture: Mode 5	R_u = 364.4 kips	ϕR_n = 485.9 kips	0.75
10	Beam shear	V_u = 116.7 kips	ϕV_n = 444.7 kips	0.26
11	Beam block shear	R_u = 728.7 kips	ϕR_n = 918.4 kips	0.79
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 = 24.9 kips	ϕr_n = 27.8 kips	0.90
15	Top plate for shear yielding	R_u = 364.4 kips	ϕR_n = 696.1 kips	0.52
	Top plate for shear rupture	R_u = 364.4 kips	ϕR_n = 382.5 kips	0.95
16	Top plate for tensile rupture in extensions	P_u = 121.5 kips	ϕP_n = 123.4 kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0.4 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.5	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 241.2 kips	ϕP_n = 513 kips	0.47
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 241.2 kips	ϕP_n = 491.4 kips	0.49
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 3.4 in**	**F2 = 3.2 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 0.7	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 241.2 kips	ϕR_n = 326.2 kips	0.74
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 241.2 kips	ϕR_n = 248.6 kips	0.97
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	R_u = 192.7 kips	ϕR_n = 296.7 kips	0.65
	Fully Restrained (FR) assumption	K_{req} = 5.83E+06	K_S = 1.03E+07	0.57
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

DF107 Limit State Summary

Column: W21X73 - Beam: W21X44



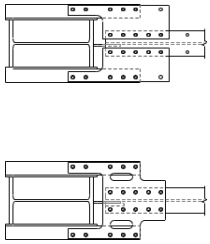
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 7.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 5.6	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	M_u = 2004 k-in	M_{pr} = 4770 k-in	0.42
3	Cover plate shear yielding	R_u = 128.9 kips	ϕR_n = 348 kips	0.37
	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} = 5599 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 115.5 kips	ϕR_n = 139.2 kips	0.83
	Bolt slip for wind load	R_u = 26.9 kips	ϕR_n = 50.4 kips	0.53
6	Alignment line location, C1	$C1_{req}$ = 4.1 in	C1 = 2.3 in	OK
7	Weld 1 strength	r_u = 6.2 kip/in	ϕr_n = 7.9 kip/in	0.78
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 = 69.3 kips	ϕP_n = 70.1 kips	0.99
	External continuity plate rupture: Mode 2	P_u = 92.4 kips	ϕP_n = 103.3 kips	0.89
	External continuity plate rupture: Mode 3	P_u = 115.5 kips	ϕP_n = 188.9 kips	0.61
	External continuity plate rupture: Mode 4	R_u = 92.4 kips	ϕR_n = 109.5 kips	0.84
	External continuity plate rupture: Mode 5	R_u = 115.5 kips	ϕR_n = 144.6 kips	0.80
10	Beam shear	V_u = 62.2 kips	ϕV_n = 217.4 kips	0.29
11	Beam block shear	R_u = 231 kips	ϕR_n = 278.1 kips	0.83
12	Web bolt shear	R_u = 62.2 kips	ϕR_n = 111.3 kips	0.56
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.11
	Shear tab weld failure	r_u = 12.6 kips	ϕr_n = 13.9 kips	0.91
15	Top plate for shear yielding	R_u = 115.5 kips	ϕR_n = 240 kips	0.48
	Top plate for shear rupture	R_u = 115.5 kips	ϕR_n = 160.9 kips	0.72
16	Top plate for tensile rupture in extensions	P_u = 46.2 kips	ϕP_n = 48.8 kips	0.95
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2.2 in	OK
18	Fuse plate width-thickness ratio of yielding 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 = 153 kips	0.59
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 140.4 kips	0.64
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 2.1 in**	**F2 = 2 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 118.1 kips	0.76
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 95.1 kips	0.95
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} = 1.22E+06	K_S = 2.71E+06	0.45
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

DF108 Limit State Summary

Column: W21X111 - Beam: W21X50



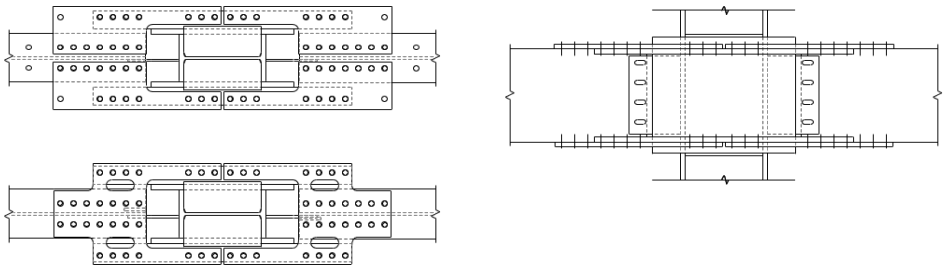
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 6.1	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 7	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	M_u = 1637 k-in	M_{pr} = 5500 k-in	0.30
3	Cover plate shear yielding	R_u = 141.8 kips	ϕR_n = 347.6 kips	0.41
	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} = 6412 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 131.9 kips	ϕR_n = 139.2 kips	0.95
	Bolt slip for wind load	R_u = 6.9 kips	ϕR_n = 50.4 kips	0.14
6	Alignment line location, C1	$C1_{req}$ = 4.4 in	C1 = 2.3 in	OK
7	Weld 1 strength	r_u = 6.9 kip/in	ϕr_n = 7.8 kip/in	0.89
8	Weld 2 strength	r_u = 8.7 kip/in	ϕr_n = 11.1 kip/in	0.78
9	External continuity plate rupture: Mode 1	P_u = 79.1 kips	ϕP_n = 80 kips	0.99
	External continuity plate rupture: Mode 2	P_u = 105.5 kips	ϕP_n = 110.4 kips	0.96
	External continuity plate rupture: Mode 3	P_u = 131.9 kips	ϕP_n = 212.2 kips	0.62
	External continuity plate rupture: Mode 4	R_u = 105.5 kips	ϕR_n = 122.2 kips	0.86
	External continuity plate rupture: Mode 5	R_u = 131.9 kips	ϕR_n = 167.9 kips	0.79
10	Beam shear	V_u = 41.2 kips	ϕV_n = 237.1 kips	0.17
11	Beam block shear	R_u = 263.7 kips	ϕR_n = 347 kips	0.76
12	Web bolt shear	R_u = 41.2 kips	ϕR_n = 111.3 kips	0.37
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.05
	Shear tab weld failure	r_u = 10.5 kips	ϕr_n = 11.1 kips	0.94
15	Top plate for shear yielding	R_u = 131.9 kips	ϕR_n = 318.8 kips	0.41
	Top plate for shear rupture	R_u = 131.9 kips	ϕR_n = 219.4 kips	0.60
16	Top plate for tensile rupture in extensions	P_u = 52.7 kips	ϕP_n = 53.3 kips	0.99
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2.2 in	OK
18	Fuse plate width-thickness ratio of yielding 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 = 289.7 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 (controlling design check)****	**$F2_{max}$ = 1.9 in**	**F2 = 1.9 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 1.1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 102.7 kips	ϕR_n = 154.7 kips	0.66
	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	K_{req} = 1.43E+06	K_S = 2.70E+06	0.53
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

DF201 Limit State Summary

Column: W21X166 - Beam: W24X146



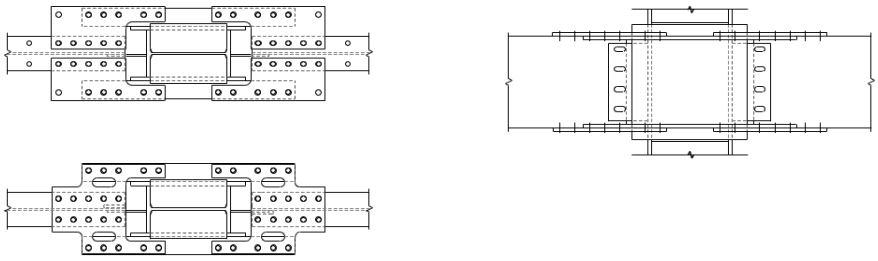
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 5.9	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 4.6	OK
	Column-beam moment ratio	ΣM^*_{pc} = 4.87E+04	ΣM^*_{pb} = 4.55E+04	OK
2	Maximum probable moment	M_u = 7320 k-in	M_{pr} = 20900 k-in	0.35
3	Cover plate shear yielding	R_u = 975.9 kips	ϕR_n = 1381.5 kips	0.71
	Cover plate thickness	t_{cp} = 1.5 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 = 408.6 kips	ϕR_n = 438.4 kips	0.93
	Bolt slip for wind load	R_u = 37.3 kips	ϕR_n = 161.4 kips	0.23
6	Alignment line location, C1	$C1_{req}$ = 10.1 in	C1 = 10.1 in	OK
7	Weld 1 strength	r_u = 22.4 kip/in	ϕr_n = 30.1 kip/in	0.74
8	Weld 2 strength	r_u = 24.7 kip/in	ϕr_n = 27.8 kip/in	0.89
9	External continuity plate rupture: Mode 1	P_u = 233.5 kips	ϕP_n = 251.4 kips	0.93
	External continuity plate rupture: Mode 2	P_u = 291.9 kips	ϕP_n = 779.3 kips	0.37
	External continuity plate rupture: Mode 3	P_u = 350.3 kips	ϕP_n = 1221.1 kips	0.29
	External continuity plate rupture: Mode 4	R_u = 291.9 kips	ϕR_n = 572 kips	0.51
	External continuity plate rupture: Mode 5	R_u = 350.3 kips	ϕR_n = 685.9 kips	0.51
10	Beam shear	V_u = 163.9 kips	ϕV_n = 481.6 kips	0.34
11	Beam block shear	R_u = 817.3 kips	ϕR_n = 1174.3 kips	0.70
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 = 39.8 kips	ϕr_n = 41.8 kips	0.95
15	Top plate for shear yielding	R_u = 408.6 kips	ϕR_n = 810 kips	0.50
	Top plate for shear rupture	R_u = 408.6 kips	ϕR_n = 444.2 kips	0.92
16	Top plate for tensile rupture in extensions	P_u = 175.1 kips	ϕP_n = 178.2 kips	0.98
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.4	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 346.8 kips	ϕP_n = 615.1 kips	0.56
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 346.8 kips	ϕP_n = 602.2 kips	0.58
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 3.4 in**	**F2 = 3.2 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 0.8	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 346.8 kips	ϕR_n = 430.3 kips	0.81
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 346.8 kips	ϕR_n = 362 kips	0.96
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	R_u = 247.8 kips	ϕR_n = 370.9 kips	0.67
	Fully Restrained (FR) assumption	K_{req} = 6.64E+06	K_S = 1.04E+07	0.64
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

DF202 Limit State Summary

Column: W21X111 - Beam: W24X94



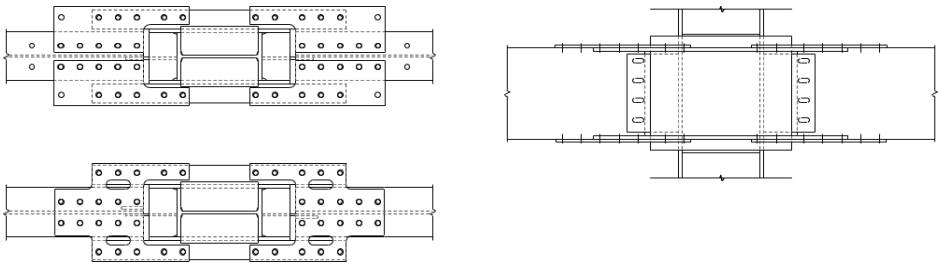
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 5.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 7	OK
	Column-beam moment ratio	ΣM^*_{pc} = 3.11E+04	ΣM^*_{pb} = 2.81E+04	OK
2	Maximum probable moment	M_u = 6086 k-in	M_{pr} = 12700 k-in	0.48
3	Cover plate shear yielding	R_u = 619.7 kips	ϕR_n = 909 kips	0.68
	Cover plate thickness	t_{cp} = 1 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 = 256.7 kips	ϕR_n = 313.1 kips	0.82
	Bolt slip for wind load	R_u = 55.3 kips	ϕR_n = 115.3 kips	0.48
6	Alignment line location, C1	$C1_{req}$ = 7.1 in	C1 = 3.1 in	OK
7	Weld 1 strength	r_u = 14.1 kip/in	ϕr_n = 20.8 kip/in	0.68
8	Weld 2 strength	r_u = 21.2 kip/in	ϕr_n = 22.3 kip/in	0.95
9	External continuity plate rupture: Mode 1	P_u = 154 kips	ϕP_n = 167.6 kips	0.92
	External continuity plate rupture: Mode 2	P_u = 205.4 kips	ϕP_n = 210.5 kips	0.98
	External continuity plate rupture: Mode 3	P_u = 256.7 kips	ϕP_n = 373.8 kips	0.69
	External continuity plate rupture: Mode 4	R_u = 205.4 kips	ϕR_n = 240.7 kips	0.85
	External continuity plate rupture: Mode 5	R_u = 256.7 kips	ϕR_n = 327.9 kips	0.78
10	Beam shear	V_u = 127.7 kips	ϕV_n = 375.4 kips	0.34
11	Beam block shear	R_u = 513.5 kips	ϕR_n = 670.6 kips	0.77
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 = 27.2 kips	ϕr_n = 27.8 kips	0.98
15	Top plate for shear yielding	R_u = 256.7 kips	ϕR_n = 498.8 kips	0.51
	Top plate for shear rupture	R_u = 256.7 kips	ϕR_n = 294.3 kips	0.87
16	Top plate for tensile rupture in extensions	P_u = 102.7 kips	ϕP_n = 106.6 kips	0.96
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0.8 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.7	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 201.8 kips	ϕP_n = 464.6 kips	0.43
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 201.8 kips	ϕP_n = 450.4 kips	0.45
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 2.6 in**	**F2 = 2.6 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 0.9	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 201.8 kips	ϕR_n = 275.6 kips	0.73
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 201.8 kips	ϕR_n = 204.8 kips	0.99
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	R_u = 164.9 kips	ϕR_n = 296.2 kips	0.56
	Fully Restrained (FR) assumption	K_{req} = 3.91E+06	K_S = 6.77E+06	0.58
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

DF203 Limit State Summary

Column: W21X122 - Beam: W24X104



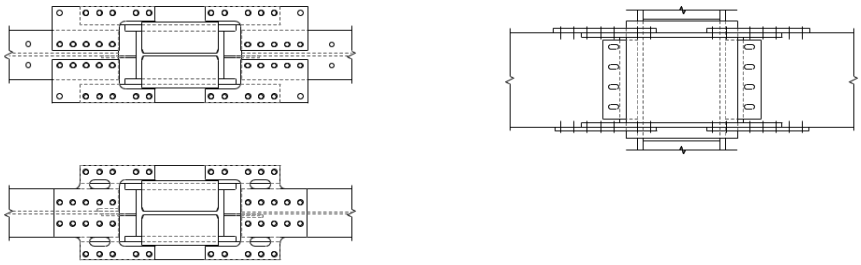
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 8.5	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 6.4	OK
	Column-beam moment ratio	ΣM^*_{pc} = 3.32E+04	ΣM^*_{pb} = 3.17E+04	OK
2	Maximum probable moment	M_u = 5713 k-in	M_{pr} = 14450 k-in	0.40
3	Cover plate shear yielding	R_u = 712.1 kips	ϕR_n = 903 kips	0.79
	Cover plate thickness	t_{cp} = 1 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 = 293.2 kips	ϕR_n = 313.1 kips	0.94
	Bolt slip for wind load	R_u = 35 kips	ϕR_n = 115.3 kips	0.30
6	Alignment line location, C1	$C1_{req}$ = 10.4 in	C1 = 10.3 in	OK
7	Weld 1 strength	r_u = 12.4 kip/in	ϕr_n = 13.4 kip/in	0.93
8	Weld 2 strength	r_u = 17.8 kip/in	ϕr_n = 19.5 kip/in	0.91
9	External continuity plate rupture: Mode 1	P_u = 175.9 kips	ϕP_n = 179.8 kips	0.98
	External continuity plate rupture: Mode 2	P_u = 234.6 kips	ϕP_n = 240.8 kips	0.97
	External continuity plate rupture: Mode 3	P_u = 293.2 kips	ϕP_n = 490.8 kips	0.60
	External continuity plate rupture: Mode 4	R_u = 234.6 kips	ϕR_n = 267 kips	0.88
	External continuity plate rupture: Mode 5	R_u = 293.2 kips	ϕR_n = 376.6 kips	0.78
10	Beam shear	V_u = 131.3 kips	ϕV_n = 361.5 kips	0.36
11	Beam block shear	R_u = 586.5 kips	ϕR_n = 886.6 kips	0.66
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	0.23
	Shear tab weld failure	r_u = 9 kips	ϕr_n = 11.1 kips	0.80
15	Top plate for shear yielding	R_u = 293.2 kips	ϕR_n = 517.5 kips	0.57
	Top plate for shear rupture	R_u = 293.2 kips	ϕR_n = 340 kips	0.86
16	Top plate for tensile rupture in extensions	P_u = 117.3 kips	ϕP_n = 118.8 kips	0.99
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 1 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 3.2	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 232.6 kips	ϕP_n = 401.6 kips	0.58
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 232.6 kips	ϕP_n = 390.5 kips	0.60
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 3.6 in**	**F2 = 3.5 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 0.7	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 232.6 kips	ϕR_n = 286.9 kips	0.81
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 232.6 kips	ϕR_n = 241.3 kips	0.96
Misc	Bridge plate tension failure	P_u = 30.4 kips	ϕP_n = 105.5 kips	0.29
	Bridge plate to cover plate weld failure	R_u = 30.4 kips	ϕR_n = 78.3 kips	0.39
	Shear tab to bridge plate weld failure	R_u = 56.5 kips	ϕR_n = 60.5 kips	0.93
	Bridge plate to column flange weld failure	R_u = 56.5 kips	ϕR_n = 65.1 kips	0.87
	Shear plate shear failure	R_u = 192.7 kips	ϕR_n = 296.7 kips	0.65
	Fully Restrained (FR) assumption	K_{req} = 4.49E+06	K_S = 7.83E+06	0.57
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODES

DF204 Limit State Summary

Column: W21X166 - Beam: W24X131



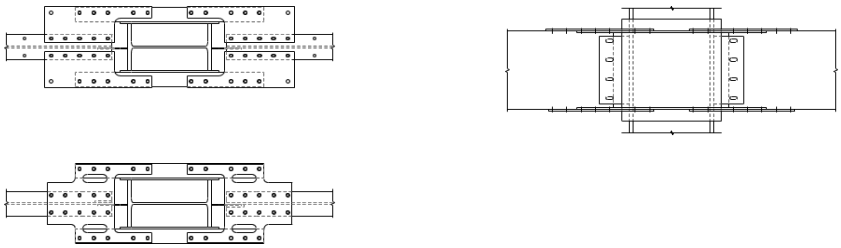
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 6.7	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 4.6	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	M_u = 2644 k-in	M_{pr} = 14031 k-in	0.19
3	Cover plate shear yielding	R_u = 643.6 kips	ϕR_n = 1186.9 kips	0.54
	Cover plate thickness	t_{cp} = 1.5 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 = 276.8 kips	ϕR_n = 313.1 kips	0.88
	Bolt slip for wind load	R_u = 7.3 kips	ϕR_n = 115.3 kips	0.06
6	Alignment line location, C1	$C1_{req}$ = 6.1 in	C1 = 6.1 in	OK
7	Weld 1 strength	r_u = 15.6 kip/in	ϕr_n = 23.9 kip/in	0.65
8	Weld 2 strength	r_u = 24.3 kip/in	ϕr_n = 25.1 kip/in	0.97
9	External continuity plate rupture: Mode 1	P_u = 166.1 kips	ϕP_n = 188.5 kips	0.88
	External continuity plate rupture: Mode 2	P_u = 221.4 kips	ϕP_n = 224.8 kips	0.98
	External continuity plate rupture: Mode 3	P_u = 276.8 kips	ϕP_n = 359.9 kips	0.77
	External continuity plate rupture: Mode 4	R_u = 221.4 kips	ϕR_n = 261.3 kips	0.85
	External continuity plate rupture: Mode 5	R_u = 276.8 kips	ϕR_n = 346.7 kips	0.80
10	Beam shear	V_u = 91 kips	ϕV_n = 444.7 kips	0.20
11	Beam block shear	R_u = 553.5 kips	ϕR_n = 802.6 kips	0.69
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 = 22.6 kips	ϕr_n = 27.8 kips	0.81
15	Top plate for shear yielding	R_u = 276.8 kips	ϕR_n = 646.9 kips	0.43
	Top plate for shear rupture	R_u = 276.8 kips	ϕR_n = 356.5 kips	0.78
16	Top plate for tensile rupture in extensions	P_u = 110.7 kips	ϕP_n = 129.5 kips	0.85
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2.8 in	OK
18	Fuse plate width-thickness ratio of yielding region	$(F6/t_p)_{max}$ = 4.2	$F6/t_p$ = 2.5	OK
	Fuse plate yielding in net section ahead of first yielding region	$R_{u,fp}$ = 219 kips	ϕP_n = 569.2 kips	0.38
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 219 kips	ϕP_n = 564.5 kips	0.39
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 2.5 in**	**F2 = 2.5 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 219 kips	ϕR_n = 326.2 kips	0.67
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 219 kips	ϕR_n = 248.6 kips	0.88
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	R_u = 247.8 kips	ϕR_n = 370.9 kips	0.67
	Fully Restrained (FR) assumption	K_{req} = 5.83E+06	K_S = 6.96E+06	0.84
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

DF205 Limit State Summary

Column: W21X111 - Beam: W21X44



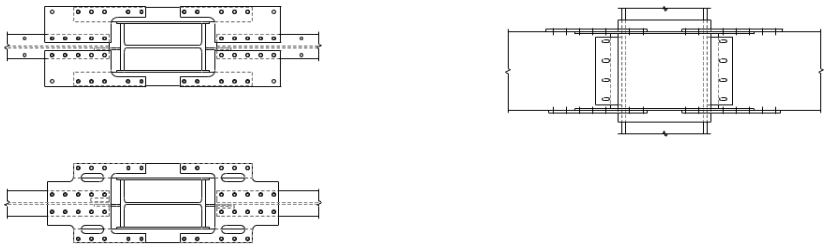
Step No.	Limit State/Check	Demand/Limit	Capacity	D/C
1	Beam flange local buckling check	$\lambda_{p,flange}$ = 9.2	λ_{flange} = 7.2	OK
	Column flange local buckling check	$\lambda_{hd,flange}$ = 7.3	λ_{flange} = 7	OK
	Column-beam moment ratio	N/A	N/A	N/A
2	Maximum probable moment	M_u = 2598 k-in	M_{pr} = 4770 k-in	0.54
3	Cover plate shear yielding	R_u = 227.2 kips	ϕR_n = 348 kips	0.65
	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} = 5599 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 116 kips	ϕR_n = 139.2 kips	0.83
	Bolt slip for wind load	R_u = 42 kips	ϕR_n = 50.4 kips	0.83
6	Alignment line location, C1	$C1_{req}$ = 5.1 in	C1 = 2.3 in	OK
7	Weld 1 strength	r_u = 11.8 kip/in	ϕr_n = 12.6 kip/in	0.94
8	Weld 2 strength	r_u = 12.1 kip/in	ϕr_n = 13.9 kip/in	0.87
9	External continuity plate rupture: Mode 1	P_u = 69.6 kips	ϕP_n = 70.1 kips	0.99
	External continuity plate rupture: Mode 2	P_u = 92.8 kips	ϕP_n = 101.5 kips	0.91
	External continuity plate rupture: Mode 3	P_u = 116 kips	ϕP_n = 200.3 kips	0.58
	External continuity plate rupture: Mode 4	R_u = 92.8 kips	ϕR_n = 109.5 kips	0.85
	External continuity plate rupture: Mode 5	R_u = 116 kips	ϕR_n = 150.2 kips	0.77
10	Beam shear	V_u = 67.9 kips	ϕV_n = 217.4 kips	0.31
11	Beam block shear	R_u = 232 kips	ϕR_n = 330.7 kips	0.70
12	Web bolt shear	R_u = 67.9 kips	ϕR_n = 111.3 kips	0.61
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.12
	Shear tab weld failure	r_u = 14.6 kips	ϕr_n = 16.7 kips	0.87
15	Top plate for shear yielding	R_u = 116 kips	ϕR_n = 270 kips	0.43
	Top plate for shear rupture	R_u = 116 kips	ϕR_n = 190.1 kips	0.61
16	Top plate for tensile rupture in extensions	P_u = 46.4 kips	ϕP_n = 48.8 kips	0.95
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2.2 in	OK
18	Fuse plate width-thickness ratio of yielding 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 = 248.6 kips	0.36
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 90 kips	ϕP_n = 264.7 kips	0.34
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 2.1 in**	**F2 = 2 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 118.1 kips	0.76
	Fuse plate tensile rupture in net section in extensions	$R_{u,fpn}$ = 90 kips	ϕR_n = 95.1 kips	0.95
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} = 1.22E+06	K_S = 2.34E+06	0.52
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

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	N/A	N/A	N/A
2	Maximum probable moment	M_u = 1586 k-in	M_{pr} = 5500 k-in	0.29
3	Cover plate shear yielding	R_u = 267.7 kips	ϕR_n = 347.6 kips	0.77
	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} = 6412 k-in	OK
5	Beam flange bolt (bearing and tearout) failure	R_u = 132.4 kips	ϕR_n = 139.2 kips	0.95
	Bolt slip for wind load	R_u = 8 kips	ϕR_n = 50.4 kips	0.16
6	Alignment line location, C1	$C1_{req}$ = 4.1 in	C1 = 2.3 in	OK
7	Weld 1 strength	r_u = 14.1 kip/in	ϕr_n = 15 kip/in	0.94
8	Weld 2 strength	r_u = 19 kip/in	ϕr_n = 19.5 kip/in	0.98
9	External continuity plate rupture: Mode 1	P_u = 79.4 kips	ϕP_n = 83.8 kips	0.95
	External continuity plate rupture: Mode 2	P_u = 105.9 kips	ϕP_n = 111.2 kips	0.95
	External continuity plate rupture: Mode 3	P_u = 132.4 kips	ϕP_n = 207.6 kips	0.64
	External continuity plate rupture: Mode 4	R_u = 105.9 kips	ϕR_n = 126 kips	0.84
	External continuity plate rupture: Mode 5	R_u = 132.4 kips	ϕR_n = 173.4 kips	0.76
10	Beam shear	V_u = 41.2 kips	ϕV_n = 237.1 kips	0.17
11	Beam block shear	R_u = 264.7 kips	ϕR_n = 362.7 kips	0.73
12	Web bolt shear	R_u = 41.2 kips	ϕR_n = 111.3 kips	0.37
13	Shear tab rupture through net section	P_u, Vu Varies	ϕR_{nn}, ϕRnv Varies	0.05
	Shear tab weld failure	r_u = 10.3 kips	ϕr_n = 11.1 kips	0.93
15	Top plate for shear yielding	R_u = 132.4 kips	ϕR_n = 318.8 kips	0.42
	Top plate for shear rupture	R_u = 132.4 kips	ϕR_n = 219.4 kips	0.60
16	Top plate for tensile rupture in extensions	P_u = 52.9 kips	ϕP_n = 53.3 kips	0.99
17	Top plate yielding due to combined flexure and shear	$P2_{req}$ = 0 in	P2 = 2.2 in	OK
18	Fuse plate width-thickness ratio of yielding 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 = 320.6 kips	0.32
	Fuse plate rupture in net section ahead of first yielding region	$R_{u,fp}$ = 102.7 kips	ϕP_n = 343.7 kips	0.30
19	**Fuse yielding region dimension (controlling design check)****	**$F2_{max}$ = 1.9 in**	**F2 = 1.9 in**	**OK**
	Fuse yielding region width-depth ratio	$(F6/F2)_{max}$ = 1.8	F6 / F2 = 1.1	OK
20	Fuse plate tensile yield in extensions	$R_{u,fpn}$ = 102.7 kips	ϕR_n = 140.6 kips	0.73
	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	K_{req} = 1.43E+06	K_S = 2.66E+06	0.54
	Rigid panel zone assumption	N/A	N/A	N/A
	External continuity plates with out-of-plane forces	N/A	N/A	N/A

** The governing limit state is presented in Step 19. All other checks are relative to the fuse plate's expected capacity as the controlling mechanism.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE

BEAM LOCAL BUCKLING & LATERAL BRACING REPORT

HCAI PCS SUBMITTAL V4.0

The purpose of this check is to demonstrate that the beams meet local buckling and bracing 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)[b]	λ	λ_p	λ Check	M_u (kip-ft)[c]	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	24.5	5.92	9.15	OK	1742	0	1.47	32.0	3053	0.57
W24X131	30.0	25.7	6.70	9.15	OK	1542	0	1.66	30.7	2560	0.60
W24X104	30.0	25.5	8.50	9.15	OK	1204	0	1.65	30.5	1915	0.63
W24X94	30.0	26.1	5.18	9.15	OK	1058	0	2.22	26.1	1084	0.98
W24X55	30.0	26.8	6.94	9.15	OK	558	1	1.80	13.5	574	0.97
W21X50	30.0	26.2	6.10	9.15	OK	458	1	1.80	13.5	474	0.97
W21X44	30.0	26.0	7.22	9.15	OK	398	2	1.73	11.5	435	0.91
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. Bar to bar length is the distance between the two ends of the external continuity plates on each side of the beam.
c. M_u equals the plastic moment of the beam (M_p), representing the maximum demand on the beam.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

3 DURAFUSE FRAMES DETAILED CONNECTION DESIGN

DF101 Detailed Connection Design	3.1
DF102 Detailed Connection Design	3.2
DF103 Detailed Connection Design	3.3
DF201 Detailed Connection Design	3.4

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Gurmeet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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.)	44	
Beam depth	d_b (in.)	24.7	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	146.0	**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}	5.9	**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.)	44	
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)*	347.8	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	50.2	
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), Factor based on code and member type, $F(E / (R_{yc} F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	
	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	6.45	**OK**

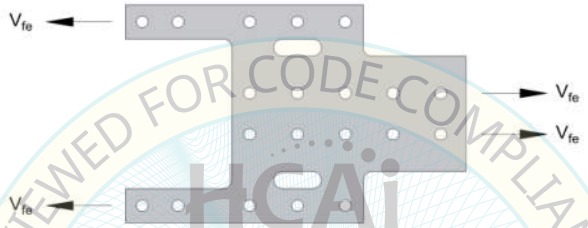
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	307	
Column design axial load	P_u (kips)	347.8	
Column area	A_{gc} (in^2)	35.9	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.7	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	3.06E+04	
Beam plastic section modulus	Z_b (in^3)	418	
Minimum 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)	2.27E+04	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.35	**OK**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)	6713	
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**
Fuse plate thickness	$T2$ (in)	1.125	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	1.125	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	1.125	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	24.7	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	404.6	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	$P_{d,total}$ (kips)	5.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	2.5	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	407.1	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	91	
Fuse plate thickness	$T2$ (in)	1.125	
Top plate thickness	$T4$ (in)	1.125	
External continuity plate thickness	$T5$ (in)	1.125	
Beam depth	d_b (in)	24.7	
Column depth	d_c (in)	21.7	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	528.4	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	0.875	
Cover plate overhange dimension	$C3$ (in)	2.625	
Column depth	d_c (in)	21.7	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	26.95	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	707.4	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	707.4	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.58	**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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	30.70	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (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	0.66	**OK**

Step 3. (continued)

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, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	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 5a. Determine the number of bolts for each bolt line on the 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)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	2.5	
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)	62.6	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	6.5	
Number of bolts per line (AISC 358 15.2(1))	n_b	7	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Top plate thickness	$T4$ (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	3.843	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.2031	
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	3	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	407.1	
Beam flange thickness	t_{fb} (in)	1.09	
Top plate thickness	$T4$ (in)	1.125	
Governing material thickness, min$(t_{fb}, T4)$	t_{min} (in)	1.1	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1339.1	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	988.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1065.4	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	584.5	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	584.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	438.4	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.93	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1043	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	1.125	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	20.2	
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	7	
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)	189.8	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	161.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.13	**OK**

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

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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	4.8	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	7	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Fuse plate thickness	$T2$ (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	3.843	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.2	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	4	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	3	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	404.6	
Beam flange thickness	t_{fb} (in)	1.09	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1339.1	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	988.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1065.4	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	584.5	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	584.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	438.4	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.92	**OK**

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kemalettin Cengizkalsi
DATE: 10/09/2026

Step 5b. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1043	
Bolt group demand from wind per line, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	20.2	
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 on the bottom flange	$n_{b,Bot}$	7	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	189.8	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	161.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.13	**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 (in)	1.625	
Beam flange width factor, 0 *if* $b_{fb} < b_{fc}$, 1.0 *otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	2.625	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	5.732	
Distance from alignment line to column	*C1* (in)	5.75	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

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)	$P_{d,total}$ (kips)	5.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	2.5	
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	
Fuse plate thickness	$T2$ (in)	1.125	
Top plate thickness	$T4$ (in)	1.125	
External continuity plate thickness	$T5$ (in)	1.125	
Cover plate thickness	$T1$ (in)	0.875	
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2T5$ for cap plate and ortho. cant./drag condition	h_{cp} (in)	30.7	
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)	0.6250	
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.45	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $T5 + T1 + [C6]$	l_{We1} (in)	5.000	
Weld 1 size	D_{W1} (1/16 in)	8	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
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)	1.78	
Vertical distance from center of rotation to geometric center	e_y (in)	0.04	

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.

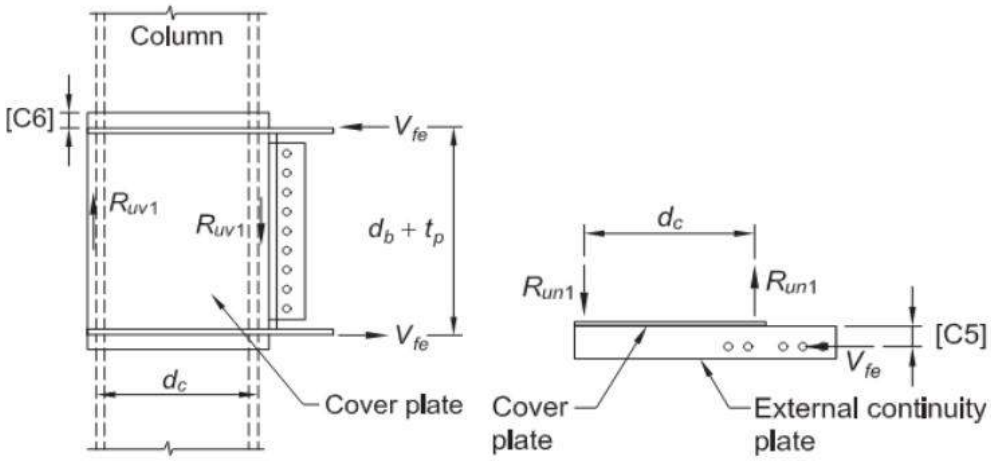
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	23035	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	17.08	12.01	57.91	205.2
W4TL	18.99	6.13	37.15	116.4
W1LB	19.15	11.33	48.74	216.9
W4BR	17.02	5.94	27.73	101.1
W1RC	9.08	7.00	2.10	63.5
W4TC	15.27	5.09	7.43	77.8
W1LC	12.63	7.48	1.51	94.5
W4BC	15.19	5.09	7.47	77.4
		12.01	57.91	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	12.0	
Orientation of Resultant Force	θ_r (degrees)	57.9	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.59	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	9.3	
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)	9.3	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.27	
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)	15.30	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.61	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/06/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

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)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	2.5	
Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang dimension	*C3* (in)	2.625	
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} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	3.96	
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)	14.0	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	20.6	
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*	0.93	**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)	1221.4	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	407.1	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	909.6	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}\,T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	9192.3	
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)*	404.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	2.5	
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)	89.5	
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)	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)	1328.2	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	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)	2100.0	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1575.0	
Weld 2 demand/capacity check for cap plate configuration, $R_u\,/\,\phi R_n$	*D/C*	0.26	**OK**

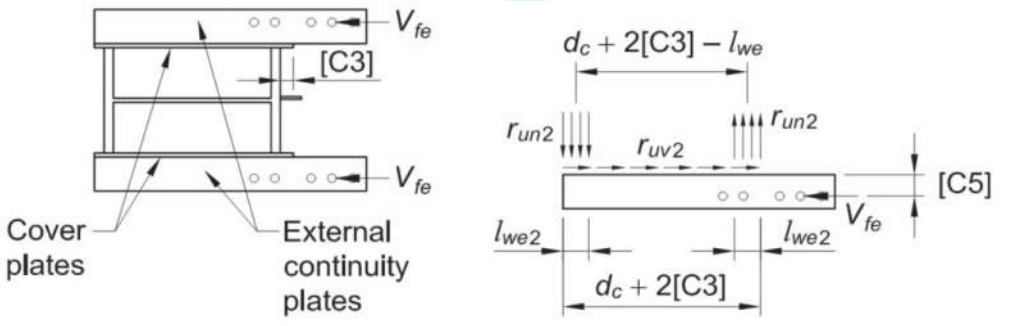


Fig. C-15.5. Weld demands for external continuity plate welds.

DESIGNED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	2.5	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	4	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	232.7	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate width	*C4* (in)	5.75	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, *T5([C4]-($d_{hole,std}$+1/16))*	A_n (in^2)	4.992	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	324.5	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	243.4	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.96	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	5	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	290.8	
External continuity plate dimension	*C2 (in)*	0.75	
Cover plate overhang dimension	*C3* (in)	2.625	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	3.375	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate width	*C4* (in)	5.75	
Diagonal bonus (AISC 360 B4.3b), *[C9]^2 (T5)/4[C5]*	*Bonus* (in)	1.068	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, T5*([C4]-$d_{hole,std}$-1/16) + Bonus*	A_n (in^2)	6.060	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	393.9	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	295.4	
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, $n_{rup1} + 2$	n_{rup}	6	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	349.0	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	0.75	
Cover plate overhang dimension	*C3* (in)	2.625	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	6.75	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate width	*C4* (in)	5.75	
Diagonal bonus (AISC 360 B4.3b), *[C7]^2 (T5)/(4[C5])*	*Bonus* (in)	4.271	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, *(T5)([C4]-$d_{hole,std}$-1/16) + Bonus*	A_n (in^2)	9.264	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	602.1	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	451.6	
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, $n_{rup1} + 1$	n_{rup}	5	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	290.8	
External continuity plate width	*C4* (in)	5.75	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	3.375	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	6.469	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	3.797	
Net tension area, A_{gt} - *((T5)*($d_{hole,std}$+1/16))*	A_{nt} (in^2)	4.992	
Net shear area, A_{gv}	A_{nv} (in^2)	3.797	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	438.4	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	328.8	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.88	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	6	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	349.0	
Bolt spacing (see schedule)	s (in)	3.375	
External continuity plate dimension	*C2 (in)*	0.75	
Cover plate overhang dimension	*C3* (in)	2.625	
External continuity plate width	*C4* (in)	5.75	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	6.75	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	6.469	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	7.594	
Net tension area, A_{gt}-*((T4)(d_{hole}+1/16))*	A_{nt} (in^2)	4.992	
Net shear area, A_{gv}	A_{nv} (in^2)	7.594	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	552.3	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	414.2	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.84	**OK**

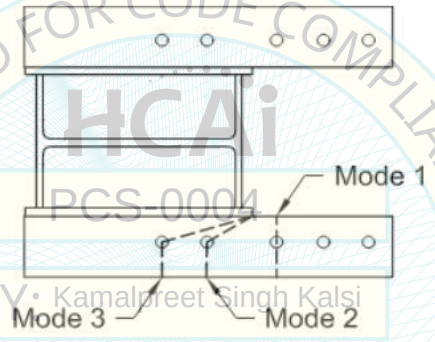


Fig. C-15.6. Tension rupture limit state for external continuity plate.

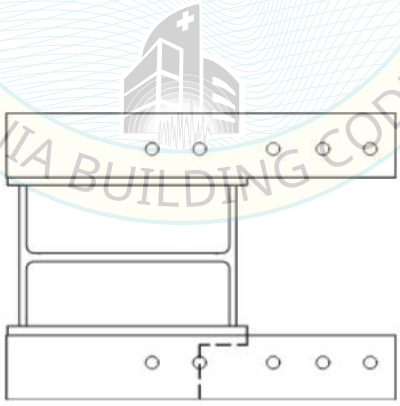


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	2.5	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	814.3	
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	7	
Length of connection on beam	$B4$ (in)	21.875	
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)	47.688	
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)	29.089	
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)	1565.8	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1174.3	
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.69	**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 provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	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 (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	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1}\ t_{min}\ F_{u,min}\ (n_n - 1)$	R_{nt1} (kips)	477.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2}\ t_{min}\ F_{u,min}$	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	0.67	**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), $1.5 T_b \mu D_u h_f n_s$	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)	5.75	
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)	962.5	

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	$T3$ (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 (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, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) T3$	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), $(0.75) 0.6 F_{uST} A_{nv}$	ϕR_{nv} (kips)	201.09	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75) F_{uST} A_e$	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	0.72	**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)	962.49	
Shear tab gross area, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	45.13	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0) 0.6 F_{yST} A_{gv}$	ϕR_{nv} (kips)	285.00	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9) F_{yST} A_{gv}$	ϕR_{nn} (kips)	427.50	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9) F_{yST} Z_{tab}$	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**

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	19	
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)	8.8	
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)	5.75	
Size of weld in 1/16^{ths} of an inch	D_{w3}	11	
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)	962.5	
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)	27.3	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	28.7	
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)	30.6	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.94	**OK**
This check is N/A when bridge plates do not apply			

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 (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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (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)

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	2.5	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	407.1	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	1.125	
Top plate dimension (see schedule)	$P8$ (in)	23.75	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	26.719	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	801.6	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	801.6	
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	7	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	14.906	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	581.3	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	436.0	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.93	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

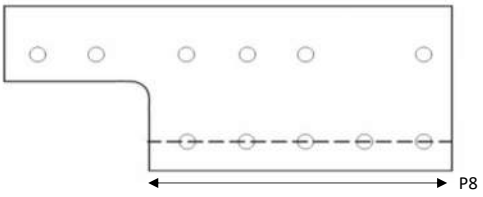


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile 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)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	2.5	
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	407.1	

Yielding check (LS 1)

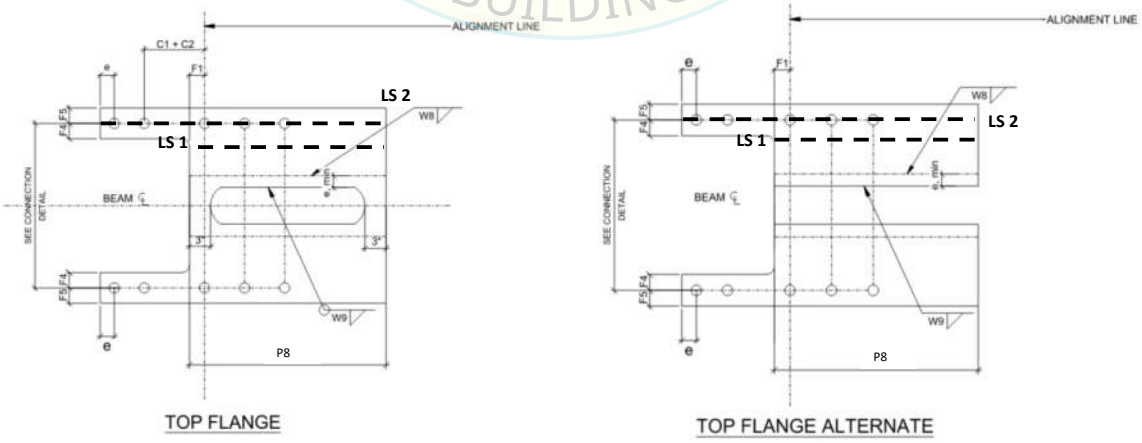
Top plate thickness (see step 2)	*T4* (in)	1.125	
Top plate dimension (see schedule)	*P8* (in)	19.5	
Fuse plate dimension	*F1 (in)*	1.75	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	21.938	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	658.1	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	658.1	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.62	**OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Number of oversized bolts in Zone M (see step 5)	n_m	3	
Number of oversized bolts on top plate (see step 5)	n_b	7	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.5	
Bolt spacing (see schedule)	*s* (in)	3.375	
Fuse plate dimension	*F1 (in)*	1.75	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	32.5	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	36.563	
Number of oversized bolts on top plate (see step 5)	n_b	7	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	25.242	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	984.4	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	738.3	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.55	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	407.1	
Beam flange width	b_{fb} (in)	12.9	
Weld 8 size	D_{W8} (1/16 in)	10	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.63	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	18.25	
Weld 9 size	D_{W9} (1/16 in)	10	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	13.50	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	12.8	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	13.9	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.92	**OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	2.5	
Total number of bolts (see step 5)	n_b	7	
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)	174.5	

Yielding check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	2.875	
Top plate thickness (see step 2s)	*T4* (in)	1.125	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	5.344	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	267.2	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	240.5	
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)	*T4* (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)(T4)*	A_e (in^2)	3.656	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	237.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	178.2	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.98	**OK**

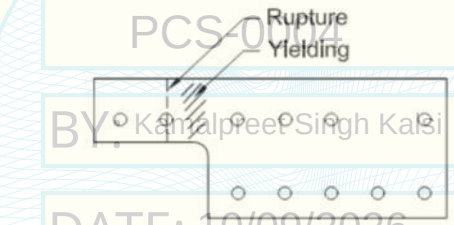


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-00024
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

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)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	2.5	
Top plate thickness (see step 2)	*T4* (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)	23.75	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	13.404	
Dimension, *([P8] - m) / 2*	*e* (in)	5.173	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.0	
Provided P2 dimension	*P2* (in)	2.75	**OK**

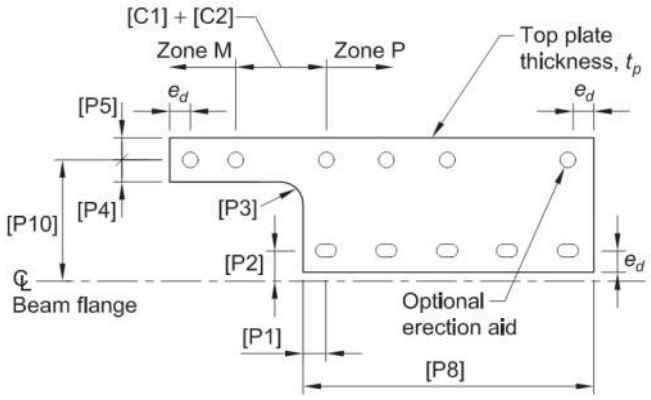


Fig. 15.6. Top plate detail.

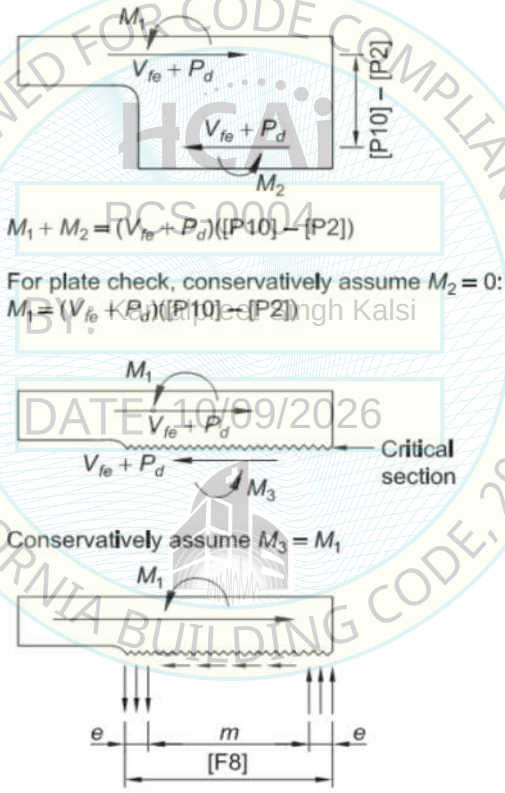


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS 0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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**
Fuse plate thickness (see step 2)	$T2$ (in)	1.125	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	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)	$n_{b,\ Bot}$	7	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	346.8	
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)	$T2$ (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, $2([F7]+[F8])T2$	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	0.63	**OK**

Step 18. (continued)

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)	$T2$ (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, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	8.888	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse}R_{tp}A_e$ [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	0.67	**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

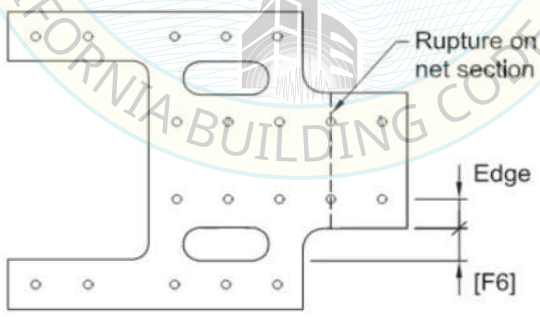


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaljeet Singh Kalsi
UPDATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	2.75	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900	
Fuse plate thickness (see step 2)	*T2* (in)	1.125	
Top plate thickness (see step 2)	*T4* (in)	1.125	
Beam depth	d_b (in)	24.7	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	167.4	
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_n	4	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	108.5	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	113.9	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	45.6	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	3.350	
Provided yielding region depth	*F2* (in)	3.250	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	219.4	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	404.6	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.8	**OK**

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.82	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

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)	$n_{b,\ Bot}$	7
Number of bolts in the M region	$n_{m,\ Bot}$	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)	346.8

Yielding check

Fuse plate dimension	*F4* (in)	1.875	
Fuse plate dimension	*F5* (in)	2.375	
Fuse plate thickness (see step 2)	*T2* (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])(T2)*	A_g (in^2)	9.5625	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	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*	0.81	**OK**

Rupture check

Fuse plate dimension	*F4* (in)	1.875	
Fuse plate dimension	*F5* (in)	2.375	
Fuse plate thickness (see step 2)	*T2* (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)(T2)*	A_e (in^2)	6.1875	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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*	0.96	**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

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

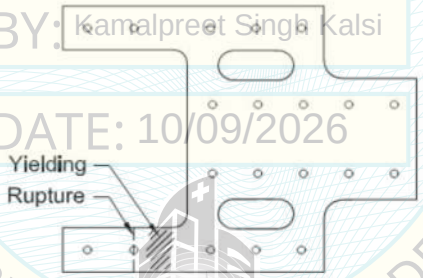
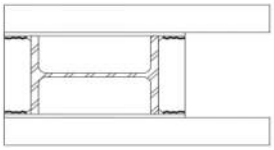


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

MISCELLANEOUS CALCULATIONS

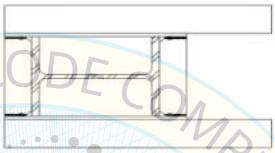
Bridge plate calculations

Bridge plate tension failure check



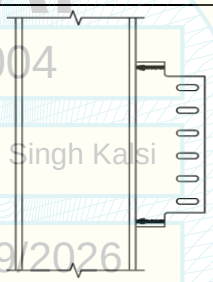
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	5.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	2.5	
External continuity plate dimension	*C5* (in)	3	
Column depth	d_c (in)	21.7	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	2.625	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}$ *T4([C3]-1")*	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**

Bridge plate to cover plate weld failure check



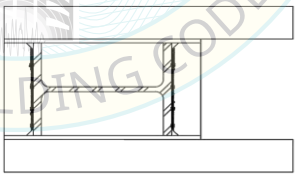
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16$ *([C3]-1")(2 sides)*	ϕR_n (kips)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

Shear tab to bridge plate weld failure check



Minimum 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, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	167.4	
Distance from alignment line to column	*C1* (in)	5.75	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2.625	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

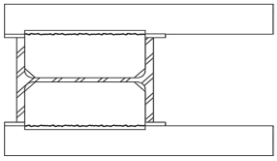
Bridge plate to column flange weld failure check



Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	5.75	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2.625	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	5.25	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

*These checks are N/A when bridge plates do not apply

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), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	296.7	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**

*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	4580
Column flange width	b_{fc} (in)	12.4
Beam depth	d_b (in)	24.7
Bay width	B (in)	360.0
Number of bolts in zone M	M	3
Number of bolts in zone P	P	4
Bolt spacing	s (in)	3.375
Cover plate thickness	$T1$ (in)	0.875
Fuse plate thickness	T2 (in)	1.125
Top plate thickness	$T4$ (in)	1.125
External continuity plate dimension	$C5$ (in)	3
Fuse plate dimension	$F2$ (in)	3.25
Fuse plate dimension	$F3$ (in)	7
Fuse plate dimension	$F4$ (in)	1.9
Fuse plate dimension	$F6$ (in)	2.75
Fuse plate dimension	$F8$ (in)	2.7
Distance from beam centerline to bolt line	$P2$ (in)	2.75
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	9.07E+04
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	2.98E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	3.99E+04
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	6.30E+04
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	2.34E+04
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	5.81E+04
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	1.11E+07
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	6.64E+06
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.60 **OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

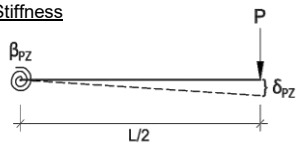
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

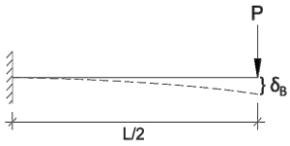


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.7
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)	2.6
Cover plate thickness	$T1$ (in)	0.875
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	1486.5
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)	2.40E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.44E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	N/A
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)	N/A
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	N/A **N/A**

*This check is N/A when connection stiffness is applied in the model analysis

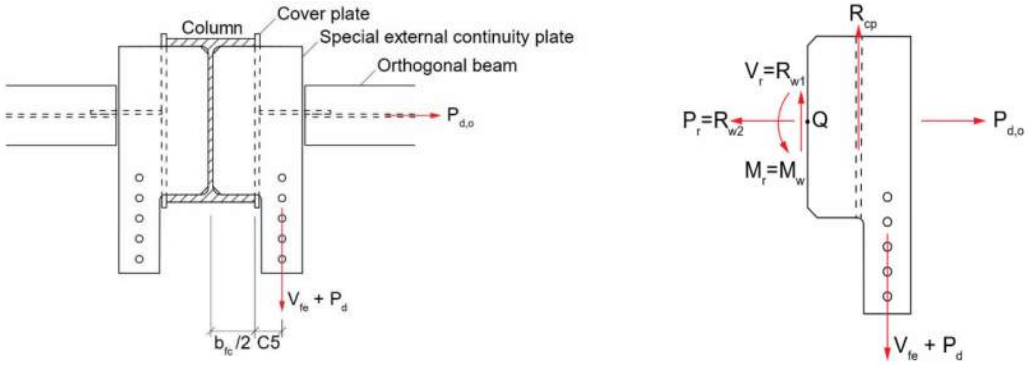
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	404.6
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	5.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	2.5
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
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
External continuity plate thickness	$T5$ (in)	1.125
External continuity plate gage dimension	$C5$ (in.)	3.00
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	110.04	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	22.25	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
	Special detailing not required		

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	1.46	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	99.2	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	21.13	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply	Special detailing not required		



Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size			
Orthogonal beam depth	$d_{b,o}$ (in)		
In plane moment frame beam 1		W24X146	
In plane moment frame beam 2		NA	
Max in-plane moment frame beam depth	d_b (in)	24.7	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0	
New continuity plate required for deeper cantilever beam		N/A	
New continuity plate width past column flange	C4 (in)	5.75	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A	**N/A**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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.)	44	
Beam depth	d_b (in.)	24.3	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	94.0	**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}	5.2	**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.)	44	
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)*	212.5	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	50.1	
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), Factor based on code and member type, $F(E/(R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	
	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	5.6	**OK**

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

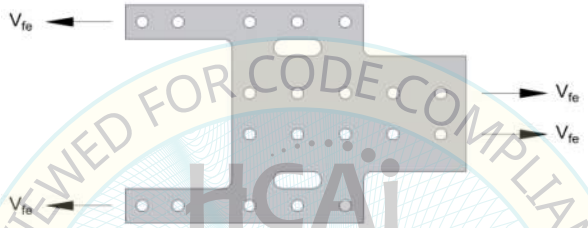
Column plastic section modulus	Z_c (in^3)	172	
Column design axial load	P_u (kips)	212.5	
Column area	A_{gc} (in^2)	21.5	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.2	
Beam depth	d_b (in)	24.3	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c*(F_{yc}-P_u/A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	1.70E+04	
Beam plastic section modulus	Z_b (in^3)	254	
Minimum 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)	1.41E+04	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.21	**OK**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)	5374	
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**
Fuse plate thickness	$T2$ (in)	0.875	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	1	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	0.875	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	24.3	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	251.6	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	251.6	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	251.6	
Beam area	A_{gb} (in^2)	27.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	3.0	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	254.6	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	50	
Fuse plate thickness	$T2$ (in)	0.875	
Top plate thickness	$T4$ (in)	1	
External continuity plate thickness	$T5$ (in)	0.875	
Beam depth	d_b (in)	24.3	
Column depth	d_c (in)	21.2	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	326.5	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	0.625	
Cover plate overhange dimension	$C3$ (in)	1.5	
Column depth	d_c (in)	21.2	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	24.2	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	453.8	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	453.8	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.56	**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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	30.30	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	568.1	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	568.1	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.57	**OK**

Step 3. (continued)

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, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	1.33	**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 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.0	
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)	62.6	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	4.1	
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 (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Top plate thickness	$T4$ (in)	1	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	2.688	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.5189	
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)	254.6	
Beam flange thickness	t_{fb} (in)	0.875	
Top plate thickness	$T4$ (in)	1	
Governing material thickness, $\min(t_{fb}, T4)$	t_{min} (in)	0.9	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	767.8	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	528.9	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	61.9	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	590.8	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	417.5	
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	417.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	313.1	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.81	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1915	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	1	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	37.9	
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	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)	135.6	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	115.3	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.33	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	251.6	
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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	3.0	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	5	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Fuse plate thickness	$T2$ (in)	0.875	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	3.072	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.7	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	3	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	2	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	251.6	
Beam flange thickness	t_{fb} (in)	0.875	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	767.8	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	528.9	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	61.9	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	590.8	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	417.5	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	417.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	313.1	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.80	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kemalettin Cagatayalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 5b. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1915	
Bolt group demand from wind per line, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	37.9	
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 on the bottom flange	$n_{b,Bot}$	5	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	135.6	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	115.3	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.33	**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 (in)	1.625	
Beam flange width factor, 0 *if* $b_{fb} < b_{fc}$, 1.0 *otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	1.5	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	4.583	
Distance from alignment line to column	*C1* (in)	4.625	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	3	
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	
Fuse plate thickness	*T2* (in)	0.875	
Top plate thickness	*T4* (in)	1	
External continuity plate thickness	*T5* (in)	0.875	
Cover plate thickness	*T1* (in)	0.625	
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3	
Cover plate height, d_b + 2[C6], d_b + [C6] - T5 or just d_b - 2T5 for cap plate and ortho. cant./drag condition	h_{cp} (in)	30.3	
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)	29.18	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), *T5 + T1 + [C6]*	l_{We1} (in)	4.500	
Weld 1 size	D_{W1} (1/16 in)	7	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
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)	3.03	
Vertical distance from center of rotation to geometric center	e_y (in)	0.05	

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.

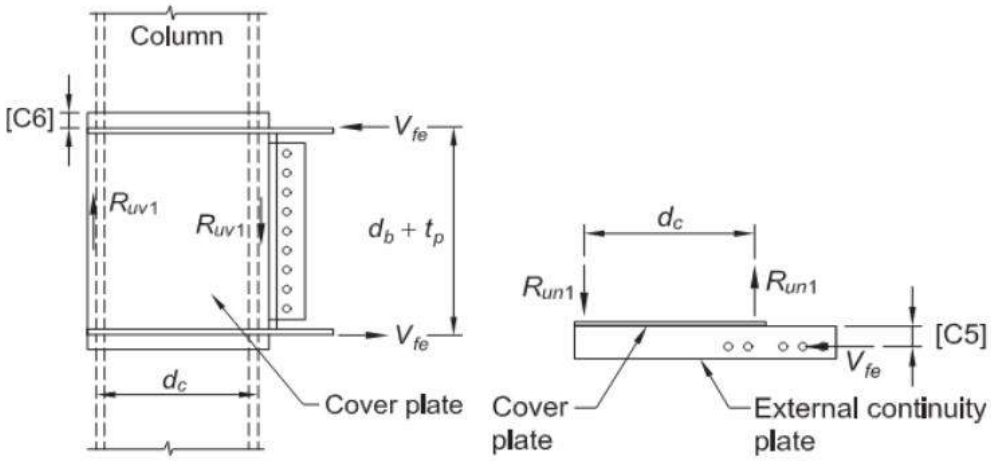
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	12028	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	16.22	10.65	62.18	172.8
W4TL	19.61	0.00	40.32	0.0
W1LB	19.71	9.65	46.26	190.3
W4BR	16.26	0.00	24.07	0.0
W1RC	7.58	5.85	2.61	44.3
W4TC	15.30	0.00	12.18	0.0
W1LC	13.63	6.59	1.45	89.8
W4BC	15.19	0.00	12.26	0.0
		10.65	62.18	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	10.7	
Orientation of Resultant Force	θ_r (degrees)	62.2	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.58	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	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)	8.0	
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	12.6	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.23	
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)	13.69	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	*D/C*	0.92	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3	
Column depth	d_c (in)	21.2	
Column flange thickness	t_{fc} (in)	0.74	
Cover plate overhang dimension	*C3* (in)	1.5	
External continuity plate dimension	*C5* (in)	3	
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} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	2.43	
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)	15.1	
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)*	251.6	
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	763.8	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	254.6	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	635.3	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	5764.9	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.53	**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)*	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.0	
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)	84.1	
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)	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)	1092.4	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	385.8	
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)	1507.3	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1130.5	
Weld 2 demand/capacity check for cap plate configuration, $R_u/\phi R_n$	*D/C*	0.23	**OK**

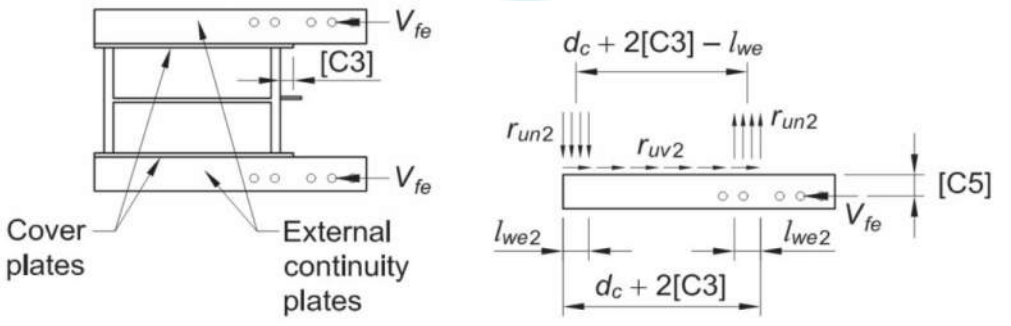


Fig. C-15.5. Weld demands for external continuity plate welds.

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
DESIGNED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.0	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	3	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	152.8	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
External continuity plate width	*C4* (in)	5	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, $T5([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	3.227	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	209.7	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	157.3	
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, $n_{rup1} + 1$	n_{rup}	4	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	203.7	
External continuity plate dimension	*C2 (in)*	2.625	
Cover plate overhang dimension	*C3* (in)	1.5	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	4.125	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
External continuity plate width	*C4* (in)	5	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (T5)/4[C5]$	*Bonus* (in)	1.241	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $T5([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	4.467	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	290.4	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	217.8	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.94	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	5	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	254.6	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	2.625	
Cover plate overhang dimension	*C3* (in)	1.5	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	7.5	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
External continuity plate width	*C4* (in)	5	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (T5)/(4[C5])$	*Bonus* (in)	4.102	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $(T5)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	7.328	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	476.3	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	357.2	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.71	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	4	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	203.7	
External continuity plate width	*C4* (in)	5	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	4.125	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	4.375	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	3.609	
Net tension area, $A_{gt} - ((T5)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	3.227	
Net shear area, A_{gv}	A_{nv} (in^2)	3.609	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	318.0	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	238.5	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.85	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	5	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	254.6	
Bolt spacing (see schedule)	s (in)	3.375	
External continuity plate dimension	*C2 (in)*	2.625	
Cover plate overhang dimension	*C3* (in)	1.5	
External continuity plate width	*C4* (in)	5	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	7.5	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	4.375	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	6.563	
Net tension area, A_{gt}-*((T4)*(d_{hole}+1/16))*	A_{nt} (in^2)	3.227	
Net shear area, A_{gv}	A_{nv} (in^2)	6.563	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	406.6	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	305.0	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.83	**OK**

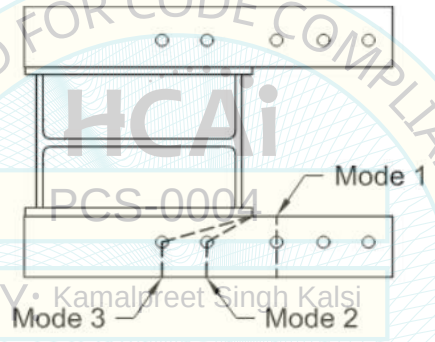


Fig. C-15.6. Tension rupture limit state for external continuity plate.

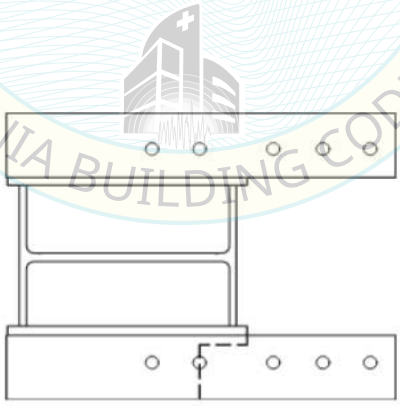


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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), $(1.0)(0.6)F_{yb}A_w C_v$	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)	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.0	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	509.2	
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	5	
Length of connection on beam	$B4$ (in)	15.125	
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)	26.469	
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)	16.133	
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)	757.6	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	568.2	
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.90	**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 provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	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 (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	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{u,min}\ (n_n\ -1)$	R_{nt1} (kips)	477.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{u,min}$	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)	4.625	
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)	615.4	

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	$T3$ (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 (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, $T3l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16)T3$	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), $(0.75)0.6F_{uST}A_{nv}$	ϕR_{nv} (kips)	201.09	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{uST}A_e$	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)	615.36	
Shear tab gross area, $T3l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab plastic section modulus, $(l_{tab}/2)^2\ T3$	Z_{tab} (in^3)	45.13	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yST}A_{gv}$	ϕR_{nv} (kips)	285.00	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yST}A_{gv}$	ϕR_{nn} (kips)	427.50	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_{yST}Z_{tab}$	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	0.50	**OK**

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	19	
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)	7.0	
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)	4.625	
Size of weld in 1/16^{ths} of an inch	D_{W3}	9	
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)	615.4	
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)	21.5	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	22.6	
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)	25.1	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.90	**OK**
This check is N/A when bridge plates do not apply			

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 (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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (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)

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	3.0	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	254.6	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	1	
Top plate dimension (see schedule)	$P8$ (in)	17	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	17.000	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	510.0	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	510.0	
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	5	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	9.500	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	370.5	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	277.9	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.92	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

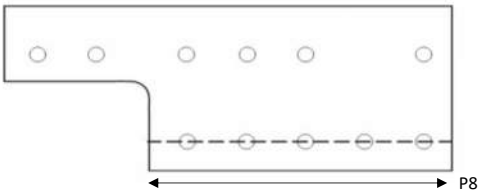


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.6
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	3.0
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	254.6

Yielding check (LS 1)

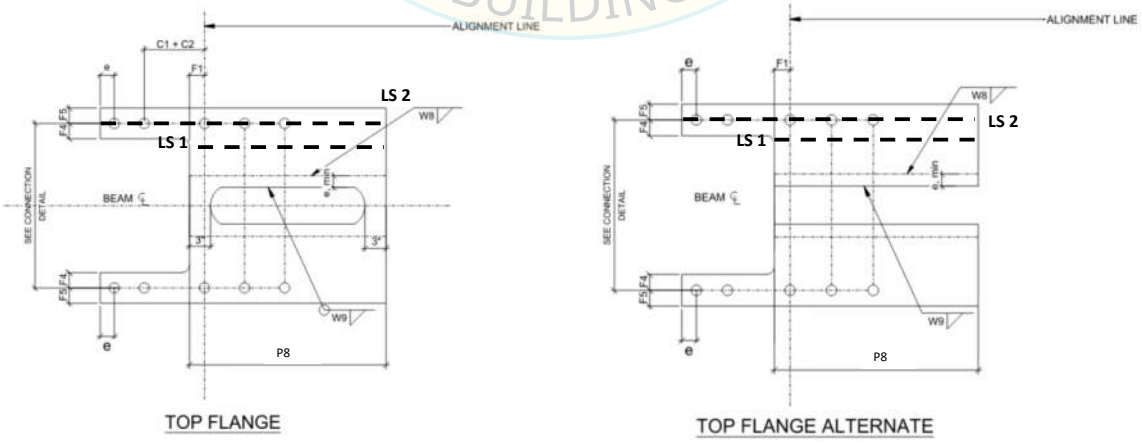
Top plate thickness (see step 2)	*T4* (in)	1	
Top plate dimension (see schedule)	*P8* (in)	12.75	
Fuse plate dimension	*F1 (in)*	1.75	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	12.750	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	382.5	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	382.5	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.67	**OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Number of oversized bolts in Zone M (see step 5)	n_m	2	
Number of oversized bolts on top plate (see step 5)	n_b	5	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.5	
Bolt spacing (see schedule)	*s* (in)	3.375	
Fuse plate dimension	*F1 (in)*	1.75	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	23.125	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	23.125	
Number of oversized bolts on top plate (see step 5)	n_b	5	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	15.938	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	621.6	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	466.2	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.55	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	254.6	
Beam flange width	b_{fb} (in)	9.07	
Weld 8 size	D_{W8} (1/16 in)	11	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.69	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	11.38	
Weld 9 size	D_{W9} (1/16 in)	11	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	6.75	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	14.0	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	15.3	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.92	**OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	3.0	
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)	101.8	

Yielding check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	1.75	
Top plate thickness (see step 2s)	*T4* (in)	1	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	3.625	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	181.3	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	163.1	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.62	**OK**

Rupture check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	1.75	
Top plate thickness (see step 2)	*T4* (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)(T4)*	A_e (in^2)	2.125	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	138.1	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	103.6	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.98	**OK**

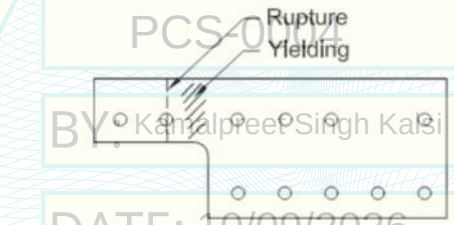


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

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)	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	3.0	
Top plate thickness (see step 2)	*T4* (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)	17.00	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	9.430	
Dimension, *([P8] - m) / 2*	*e* (in)	3.785	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.0	
Provided P2 dimension	*P2* (in)	2.75	**OK**

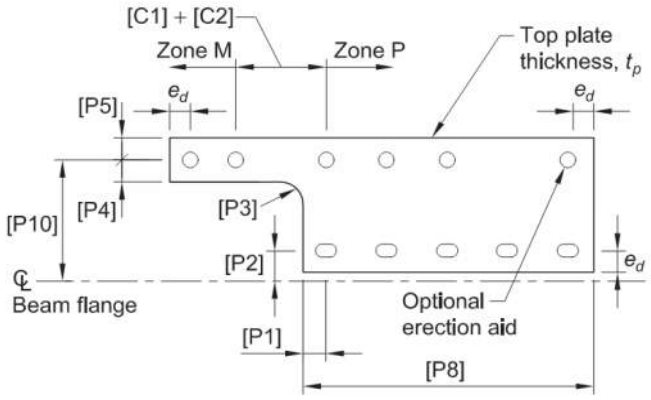


Fig. 15.6. Top plate detail.

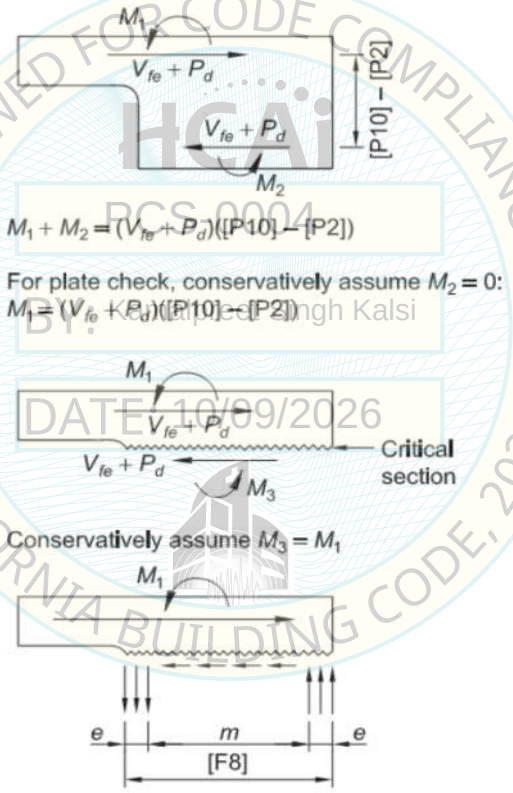


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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**
Fuse plate thickness (see step 2)	$T2$ (in)	0.875	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	1.7	**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)	251.6	
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)	$n_{b,\ Bot}$	5	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	201.3	
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)	$T2$ (in)	0.875	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Gross area for yielding, $2([F7]+[F8])T2$	A_g (in^2)	7.700	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	385.0	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	346.5	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.58	**OK**

Step 18. (continued)

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)	$T2$ (in)	0.875	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.438	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	5.075	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	P_n (kips)	395.9	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	296.9	
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.68	**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

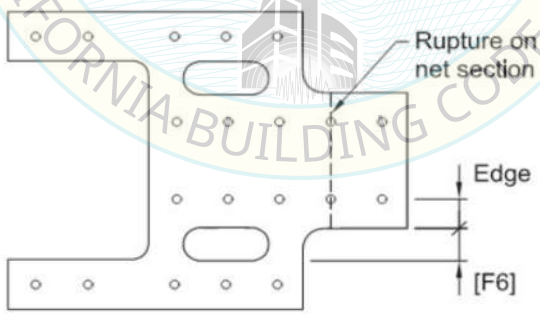


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kanpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	1.5	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	12700	
Fuse plate thickness (see step 2)	*T2* (in)	0.875	
Top plate thickness (see step 2)	*T4* (in)	1	
Beam depth	d_b (in)	24.3	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	133.1	
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_n	4	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	108.5	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	113.9	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	45.6	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	2.626	
Provided yielding region depth	*F2* (in)	2.625	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	137.8	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	251.6	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.8	**OK**

Fuse plate yielding region geometry			
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.57	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.6
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)	$n_{b,\ Bot}$	5
Number of bolts in the M region	$n_{m,\ Bot}$	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)	201.3

Yielding check

Fuse plate dimension	*F4* (in)	1.875	
Fuse plate dimension	*F5* (in)	1.625	
Fuse plate thickness (see step 2)	*T2* (in)	0.875	
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])(T2)*	A_g (in^2)	6.125	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	R_n (kips)	306.25	
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	275.63	
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.73	**OK**

Rupture check

Fuse plate dimension	*F4* (in)	1.875	
Fuse plate dimension	*F5* (in)	1.625	
Fuse plate thickness (see step 2)	*T2* (in)	0.875	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	3.5	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	R_n (kips)	273.0	
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	204.8	
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

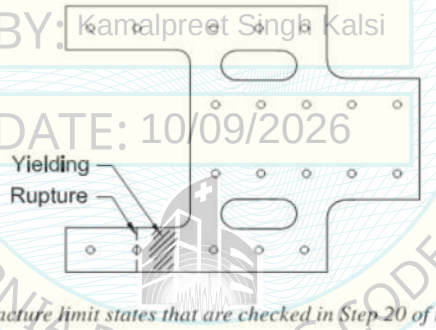


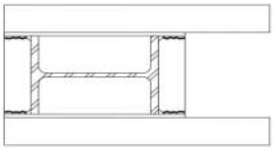
Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

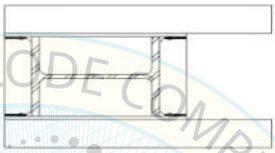
Bridge plate calculations

Bridge plate tension failure check



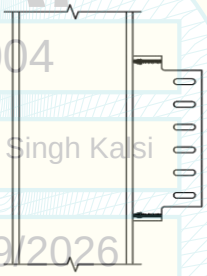
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	251.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	3	
External continuity plate dimension	*C5* (in)	3	
Column depth	d_c (in)	21.2	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	1.5	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}$ *T4([C3]-1")*	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**

Bridge plate to cover plate weld failure check



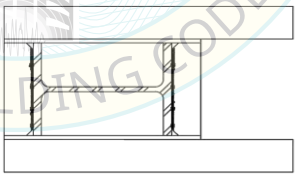
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16$ *([C3]-1")(2 sides)*	ϕR_n (kips)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

Shear tab to bridge plate weld failure check



Minimum 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)	4.625	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	1.5	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

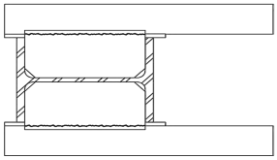
Bridge plate to column flange weld failure check



Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	4.625	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	1.5	
Column flange width	b_{fc} (in)	8.3	
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	3	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

*These checks are N/A when bridge plates do not apply

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



Column depth	d_c (in)	21.2	
Column flange thickness	t_{fc} (in)	0.74	
Weld 4 size	D_{W4} (1/16 in)	0	
Weld 4 effective length, d_c - 2 t_{fc}	l_{eff} (in)	19.72	
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)	N/A	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.000	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	N/A	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	8.3
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)	3.375
Cover plate thickness	$T1$ (in)	0.625
Fuse plate thickness	T2 (in)	0.875
Top plate thickness	$T4$ (in)	1
External continuity plate dimension	$C5$ (in)	3
Fuse plate dimension	$F2$ (in)	2.625
Fuse plate dimension	$F3$ (in)	4.875
Fuse plate dimension	$F4$ (in)	1.9
Fuse plate dimension	$F6$ (in)	1.5
Fuse plate dimension	$F8$ (in)	1.65
Distance from beam centerline to bolt line	$P2$ (in)	2.75
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	5.29E+04
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	3.43E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	1.01E+05
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	6.01E+04
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	2.68E+04
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	5.27E+04
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	1.13E+07
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	3.91E+06
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.35 **OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

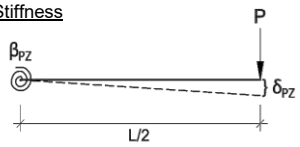
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

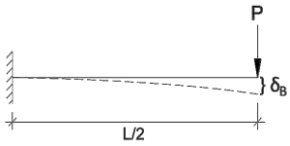


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (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)	21.2
Column flange width	b_{fc} (in)	8.3
Column web thickness	t_{wc} (in)	0.455
Column flange thickness	t_{fc} (in)	0.74
Cover plate overhang	$C3$ (in)	1.5
Cover plate thickness	$T1$ (in)	0.625
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	969.5
Column ratio, $(d_c - t_{fc}) / B$	α	0.057
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)	1.56E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	5.70E+04
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	N/A
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)	N/A
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	N/A **N/A**

*This check is N/A when connection stiffness is applied in the model analysis

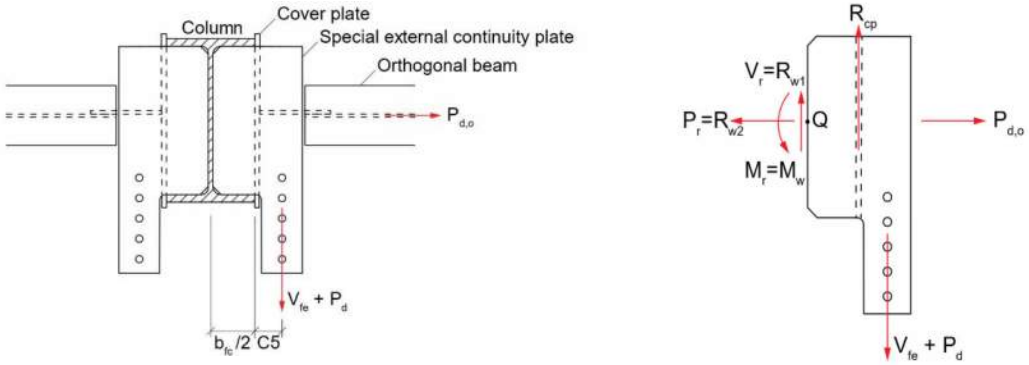
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	251.6
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	6.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	3.0
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
Column flange width	b_{fc} (in.)	8.3
Column depth	d_c (in.)	21.2
Column flange thickness	t_{fc} (in.)	0.74
Column web thickness	t_{wc} (in.)	0.46
External continuity plate thickness	$T5$ (in)	0.875
External continuity plate gage dimension	$C5$ (in.)	3.00
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	85.07	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	17.26	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
		Special detailing not required	

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	1.24	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	76.7	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	16.38	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply		Special detailing not required	



Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size			
Orthogonal beam depth	$d_{b,o}$ (in)		
In plane moment frame beam 1		W24X94	
In plane moment frame beam 2		NA	
Max in-plane moment frame beam depth	d_b (in)	24.3	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0	
New continuity plate required for deeper cantilever beam		N/A	
New continuity plate width past column flange	C4 (in)	5	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A	**N/A**

This check is N/A when OOP forces do not apply

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

Connection ID DF103

W16X57 Column and W24X55 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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Beam limitation checks			
Beam size		W24X55	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	44	
Beam depth	d_b (in.)	23.6	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	55.0	**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}	54.6	**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}	6.9	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		W16X57	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	44	
Column depth	d_c (in.)	16.4	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	57	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	0.715	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	108.8	
Gross area of the column section	A_g (in^2)	16.8	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	51.5	
Column web width-to-thickness ratio	λ_{web}	33	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), Factor based on code and member type, $F(E/(R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	
	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	4.98	**OK**

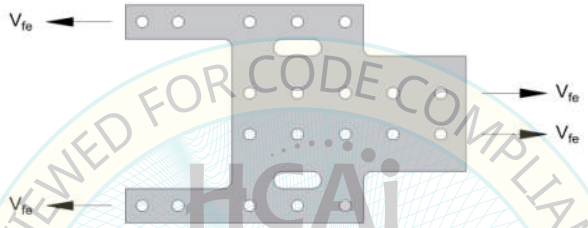
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	105	
Column design axial load	P_u (kips)	108.8	
Column area	A_{gc} (in^2)	16.8	
Story Height	H (in)	216.0	
Column depth	d_c (in)	16.4	
Beam depth	d_b (in)	23.6	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c{*}(F_{yc}-P_u/A_{gc}){*}(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	N/A	
Beam plastic section modulus	Z_b (in^3)	134	
Minimum bay width	B (in)	360.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	5895	
Clear distance between column faces, $B-d_c$	L_h (in)	343.6	
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)	31.5	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	65.9	
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	540	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	N/A	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	N/A	**N/A**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)	832	
Beam plastic section modulus	Z_b (in^3)	134	
Beam plastic bending moment , $F_{yb} Z_b$	M_p (k-in)	6700	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	5895	**OK**
Fuse plate thickness	$T2$ (in)	0.625	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	0.625	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	0.625	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	23.6	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	121.7	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	121.7	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	121.7	
Beam area	A_{gb} (in^2)	16.2	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	3.5	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	125.2	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	32	
Fuse plate thickness	$T2$ (in)	0.625	
Top plate thickness	$T4$ (in)	0.625	
External continuity plate thickness	$T5$ (in)	0.625	
Beam depth	d_b (in)	23.6	
Column depth	d_c (in)	16.4	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	198.1	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	0.875	
Cover plate overhange dimension	$C3$ (in)	1	
Column depth	d_c (in)	16.4	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	18.4	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	483.0	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	483.0	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.26	**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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	25.98	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (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	0.29	**OK**

Step 3. (continued)

Cover plate thickness check (AISC 341 E3.6e.2)			
Beam depth	d_b (in)	23.6	
Beam flange thickness	t_{fb} (in)	0.505	
Column flange thickness	t_{fc} (in)	0.715	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	22.59	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	14.97	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.417	
Thickness check, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	2.10	**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)	134	
Beam flange thickness	t_{fb} (in)	0.505	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	0.473	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	21.9	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	112.1	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	7370	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	8017	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.92	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.5	
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)	27.8	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	4.5	
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 (in)	1.0625	
Bolt spacing (see schedule)	s (in)	2.75	
Top plate thickness	$T4$ (in)	0.625	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	2.080	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.8307	
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)	125.2	
Beam flange thickness	t_{fb} (in)	0.505	
Top plate thickness	$T4$ (in)	0.625	
Governing material thickness, min$(t_{fb}, T4)$	t_{min} (in)	0.5	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	295.4	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	285.6	
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, $e - d_O / 2$	l_{c2} (in)	0.6	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	23.4	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	309.0	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	185.6	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	185.6	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	139.2	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.90	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	148	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	0.625	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	3.0	
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	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)	59.3	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	50.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.06	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	121.7	
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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	3.3	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	5	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.0625	
Bolt spacing (see schedule)	s (in)	2.75	
Fuse plate thickness	$T2$ (in)	0.625	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	2.080	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.8	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	3	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	2	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	121.7	
Beam flange thickness	t_{fb} (in)	0.505	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	295.4	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	285.6	
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, $e - d_O / 2$	l_{c2} (in)	0.6	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	23.4	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	309.0	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	185.6	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	185.6	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	139.2	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.87	**OK**

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kemalettin Cagatayoglu
DATE: 10/09/2026

Step 5b. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	148	
Bolt group demand from wind per line, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	3.0	
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 on the bottom flange	$n_{b,Bot}$	5	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	59.3	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	50.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.06	**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)	23.6	
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e (in)	1.0625	
Beam flange width factor, 0 *if* $b_{fb} < b_{fc}$, 1.0 *otherwise*	α	0	
Cover plate overhang dimension	*C3* (in)	1	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	2.4785	
Distance from alignment line to column	*C1* (in)	4.25	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	3.5	
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	32	
Column depth	d_c (in)	16.4	
Column flange thickness	t_{fc} (in)	0.715	
Beam depth	d_b (in)	23.6	
Fuse plate thickness	$T2$ (in)	0.625	
Top plate thickness	$T4$ (in)	0.625	
External continuity plate thickness	$T5$ (in)	0.625	
Cover plate thickness	$T1$ (in)	0.875	
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2T5$ for cap plate and ortho. cant./drag condition	h_{cp} (in)	25.975	
Gage dimension from the cover plate to the line of bolts on the external continuity plates	$C5$ (in)	2.25	
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.3750	
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	25.23	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $T5 + T1 + [C6]$	l_{We1} (in)	4.500	
Weld 1 size	D_{W1} (1/16 in)	4	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	14.97	
Horizontal distance from center of rotation to geometric center	e_x (in)	2.62	
Vertical distance from center of rotation to geometric center	e_y (in)	0.10	

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.

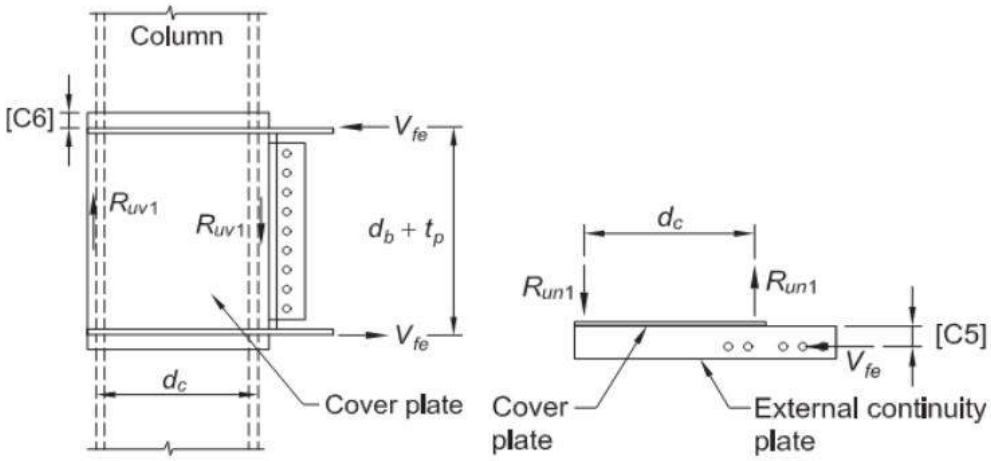
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	4876	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	13.66	5.34	65.87	72.9
W4TL	17.71	0.00	34.18	0.0
W1LB	16.35	4.86	48.57	79.4
W4BR	15.20	0.00	18.09	0.0
W1RC	5.60	2.83	3.64	15.9
W4TC	14.91	0.00	10.69	0.0
W1LC	10.82	3.23	1.88	34.9
W4BC	14.71	0.00	10.84	0.0
		5.34	65.87	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	5.3	
Orientation of Resultant Force	θ_r (degrees)	65.9	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.50	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe} + P_d/2)*(d_b + T2/2 + T4/2)/\Sigma RL)$	r_{uv} (kip/in)	6.5	
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)	3.8	
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	7.5	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.23	
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)	7.94	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.95	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.5	
Column depth	d_c (in)	16.4	
Column flange thickness	t_{fc} (in)	0.715	
Cover plate overhang dimension	*C3* (in)	1	
External continuity plate dimension	*C5* (in)	2.25	
Weld 2 size	D_{W2} (1/16 in)	4	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.25	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), t_{fc} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	2.34	
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)	6.8	
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)	7.7	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	10.3	
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)	11.1	
External continuity plate-to-cover plate weld tearing check, $r_u\,/\,\phi r_n$	*D/C*	0.93	**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)*	121.7	
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	281.6	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	125.2	
External continuity plate thickness (see step 2)	*T5* (in)	0.625	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	345.0	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}\,T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	2380.5	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.48	**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)*	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.5	
Column fillet dimension	*k* (in)	1.1	
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)	64.1	
Column flange width	b_{fc} (in)	7.12	
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	24.98	
Weld 2 size	D_{W2} (1/16 in)	4	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	476.0	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	185.4	
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)	682.8	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	512.1	
Weld 2 demand/capacity check for cap plate configuration, $R_u\,/\,\phi R_n$	*D/C*	0.24	**OK**

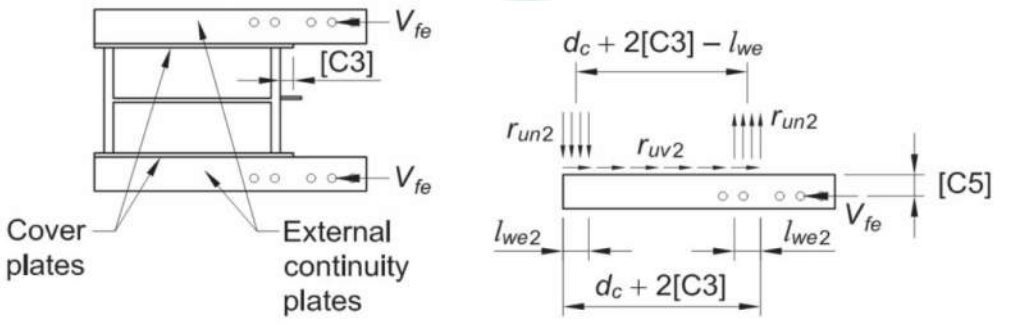


Fig. C-15.5. Weld demands for external continuity plate welds.

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
DESIGNED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.5	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	3	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	75.1	
External continuity plate thickness (see step 2)	*T5* (in)	0.625	
External continuity plate width	*C4* (in)	3.375	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Net area of external continuity plate cross section, *T5([C4]-($d_{hole,std}$ +1/16))*	A_n (in^2)	1.563	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	101.6	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	76.2	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.99	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	4	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	100.1	
External continuity plate dimension	*C2 (in)*	1.75	
Cover plate overhang dimension	*C3* (in)	1	
External continuity plate dimension	*C5* (in)	2.25	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	2.75	
External continuity plate thickness (see step 2)	*T5* (in)	0.625	
External continuity plate width	*C4* (in)	3.375	
Diagonal bonus (AISC 360 B4.3b), *[C9]^2 (T5)/4[C5]*	*Bonus* (in)	0.525	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Area of external continuity plate cross section with bonus, T5*([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	2.088	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	135.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	101.8	
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, $n_{rup1} + 2$	n_{rup}	5	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	125.2	
Bolt spacing (see schedule)	*s* (in)	2.75	
External continuity plate dimension	*C2 (in)*	1.75	
Cover plate overhang dimension	*C3* (in)	1	
External continuity plate dimension	*C5* (in)	2.25	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5.5	
External continuity plate thickness (see step 2)	*T5* (in)	0.625	
External continuity plate width	*C4* (in)	3.375	
Diagonal bonus (AISC 360 B4.3b), *[C7]^2 (T5)/(4[C5])*	*Bonus* (in)	2.101	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Area of external continuity plate cross section with bonus, *(T5)([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	3.663	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	238.1	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	178.6	
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, $n_{rup1} + 1$	n_{rup}	4	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	100.1	
External continuity plate width	*C4* (in)	3.375	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	2.75	
External continuity plate thickness (see step 2)	*T5* (in)	0.625	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	2.109	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	1.719	
Net tension area, A_{gt} - *((T5)*($d_{hole,std}$ +1/16))*	A_{nt} (in^2)	1.563	
Net shear area, A_{gv}	A_{nv} (in^2)	1.719	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	153.1	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	114.8	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.87	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	5	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	125.2	
Bolt spacing (see schedule)	s (in)	2.75	
External continuity plate dimension	*C2 (in)*	1.75	
Cover plate overhang dimension	*C3* (in)	1	
External continuity plate width	*C4* (in)	3.375	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5.5	
External continuity plate thickness (see step 2)	*T5* (in)	0.625	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	2.109	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	3.438	
Net tension area, A_{gt}-*((T4)(d_{hole}+1/16))*	A_{nt} (in^2)	1.563	
Net shear area, A_{gv}	A_{nv} (in^2)	3.438	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	204.7	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	153.5	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.82	**OK**

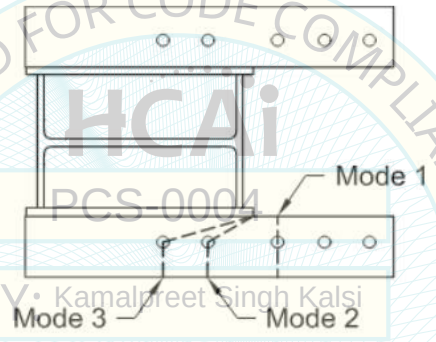


Fig. C-15.6. Tension rupture limit state for external continuity plate.

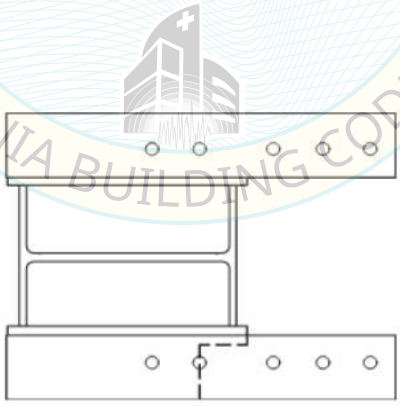


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum bay width	B (in)	360.0	
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	31.5	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	5895	
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	65.9	
Beam depth	d_b (in)	23.6	
Beam web thickness	t_{wb} (in)	0.395	
Beam web area, $d_b t_{wb}$	A_w (in)	9.322	
Beam web width-to-thickness ratio, h / t_w	l_{web}	54.6	
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)	279.7	
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.24	**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)	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.5	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	250.3	
Beam flange thickness	t_{fb} (in)	0.505	
Beam flange width	b_{fb} (in)	7.01	
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	5	
Length of connection on beam	$B4$ (in)	12.0625	
Distance from beam centerline to bolt line	$P2$ (in)	2.5	
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	1.015	
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	12.183	
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	0.573	
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	8.206	
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)	357.3	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	268.0	
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)	65.9	
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), $R_u / \phi r_n$	$n_{n,req}$	2	
Provided number of shear tab bolts	n_n	4	**OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	20.75	
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	20	
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	19.75	**OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	0.395	
Shear tab provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	R_{nb} (kips)	184.9	
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 (in)	1.000	
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	5.917	
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)	5.1	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{u,min}\ (n_n\ -1)$	R_{nt1} (kips)	471.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.6	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{u,min}$	R_{nt2} (kips)	18.3	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	490.1	
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	148.4	
Controlling strength, $min(R_{nb}, R_{nt,} R_{nv})$	R_n (kips)	148.4	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	111.3	
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.59	**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	4	
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	71.2	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	65.9	
Distance from alignment line to column	$C1$ (in)	4.25	
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)	344.6	

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	$T3$ (in)	0.5	
Shear tab length (see step 12)	l_{tab} (in)	20	
Beam T dimension limit	T_{beam} (in)	20.75	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1	
Slotted bolt hole spacing, $(T_{beam}\ - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	5.8125	
Gross area of shear tab, $T3l_{tab}$	A_{gv} (in^2)	9.88	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16)T3$	A_{nv} (in^2)	8.13	
Effective net area, $A_e = A_{nv}$	A_e (in^2)	8.13	
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{uST}A_{nv}$	ϕR_{nv} (kips)	237.66	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{uST}A_e$	ϕR_{nn} (kips)	396.09	
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	0.04	**OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	65.85	
Required axial strength of the shear tab (see above)	P_u (kips)	71.19	
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)	344.63	
Shear tab gross area, $T3l_{tab}$	A_{gv} (in^2)	9.88	
Shear tab plastic section modulus, $(l_{tab}/2)^2\ T3$	Z_{tab} (in^3)	48.76	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yST}A_{gv}$	ϕR_{nv} (kips)	296.25	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yST}A_{gv}$	ϕR_{nn} (kips)	444.38	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_{yST}Z_{tab}$	ϕM_n (kips)	2194.10	
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	0.19	**OK**

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	19.75	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	65.9	
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	3.3	
Number of slotted bolt holes in shear tab	n_{vb}	4	
Slotted bolt hole spacing	s_{slot} (in)	5.8125	
Distance from line of bolts to shear tab weld	$C1$ (in)	4.25	
Size of weld in 1/16^{ths} of an inch	D_{w3}	5	
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)	344.6	
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)	10.8	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	11.3	
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)	13.9	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.81	**OK**
This check is N/A when bridge plates do not apply			

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 (in)	1.0625	
Beam depth	d_b (in)	23.6	
Length of the shear tab (see step 12)	l_{tab} (in)	19.75	
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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (in)	1.61175	
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	1.625	**OK**
Provided outside shear tab slot dimension, [S2] = [S1]	$S2$ (in)	1.625	

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

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	3.5	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	125.2	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	0.625	
Top plate dimension (see schedule)	$P8$ (in)	14.75	
Gross shear area of top plate, (P8)(T4)	A_g (in^2)	9.219	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	276.6	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	276.6	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.45	**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	5	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	6.094	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	237.7	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	178.2	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.70	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

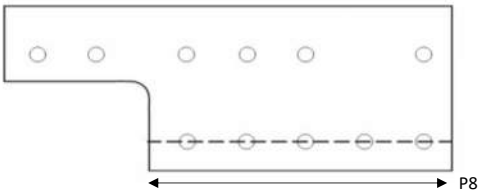


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	3.5	
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	125.2	

Yielding check (LS 1)

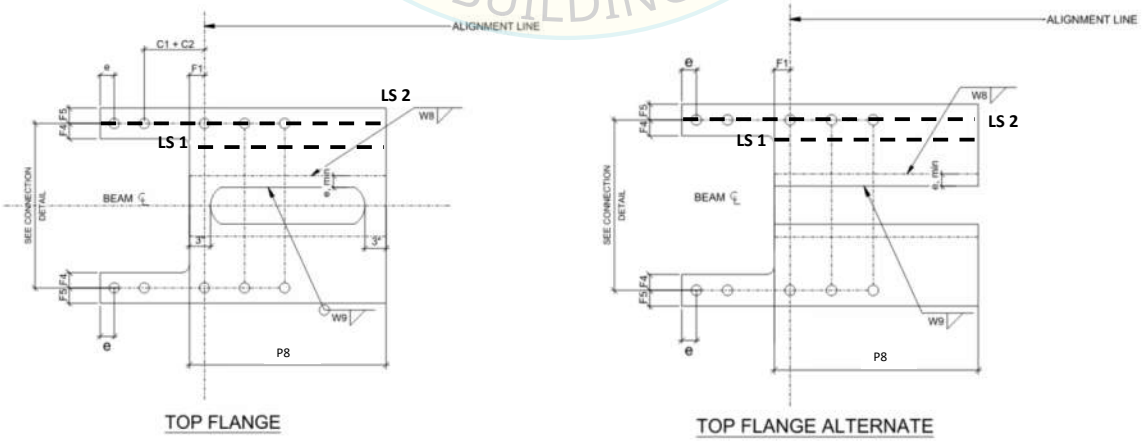
Top plate thickness (see step 2)	*T4* (in)	0.625	
Top plate dimension (see schedule)	*P8* (in)	9.125	
Fuse plate dimension	*F1 (in)*	1.88	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	5.703	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	171.1	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	171.1	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.73	**OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	0.75	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	0.9375	
Number of oversized bolts in Zone M (see step 5)	n_m	2	
Number of oversized bolts on top plate (see step 5)	n_b	5	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1	
Bolt spacing (see schedule)	*s* (in)	2.75	
Fuse plate dimension	*F1 (in)*	1.88	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	17	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	10.625	
Number of oversized bolts on top plate (see step 5)	n_b	5	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	7.695	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	300.1	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	225.1	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.56	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	125.2	
Beam flange width	b_{fb} (in)	7.01	
Weld 8 size	D_{W8} (1/16 in)	9	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.56	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	8.00	
Weld 9 size	D_{W9} (1/16 in)	9	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	3.13	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	11.3	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	12.5	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.90	**OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	3.5	
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)	50.1	

Yielding check

Top plate dimension	*P4* (in)	1.5	
Top plate dimension	*P5* (in)	1.25	
Top plate thickness (see step 2s)	*T4* (in)	0.625	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	1.719	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	85.9	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	77.3	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.65	**OK**

Rupture check

Top plate dimension	*P4* (in)	1.5	
Top plate dimension	*P5* (in)	1.25	
Top plate thickness (see step 2)	*T4* (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)(T4)*	A_e (in^2)	1.094	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	71.1	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	53.3	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.94	**OK**

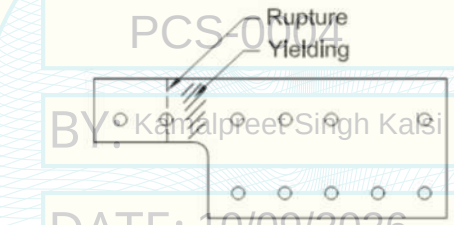


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-00024
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

Entering and tightening check

Beam flange fillet dimension	k_1	1.0	
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)	1.9375	
Provided P2 dimension	*P2* (in)	2.5	**OK**

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	3.5	
Top plate thickness (see step 2)	*T4* (in)	0.625	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	*P10* (in)	6.685	
Inside length of the top plate	*P8* (in)	14.75	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	7.417	
Dimension, *([P8] - m) / 2*	*e* (in)	3.666	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.0	
Provided P2 dimension	*P2* (in)	2.5	**OK**

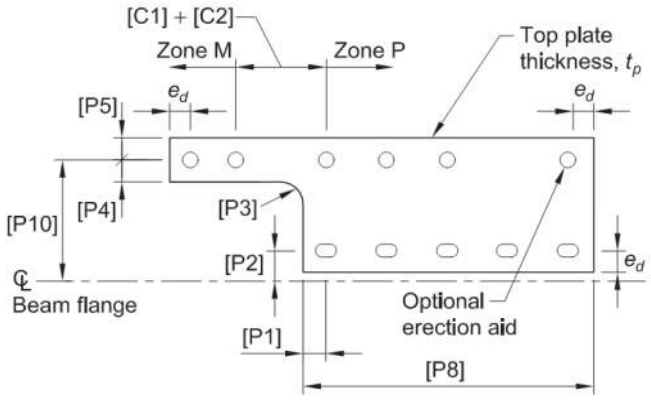


Fig. 15.6. Top plate detail.

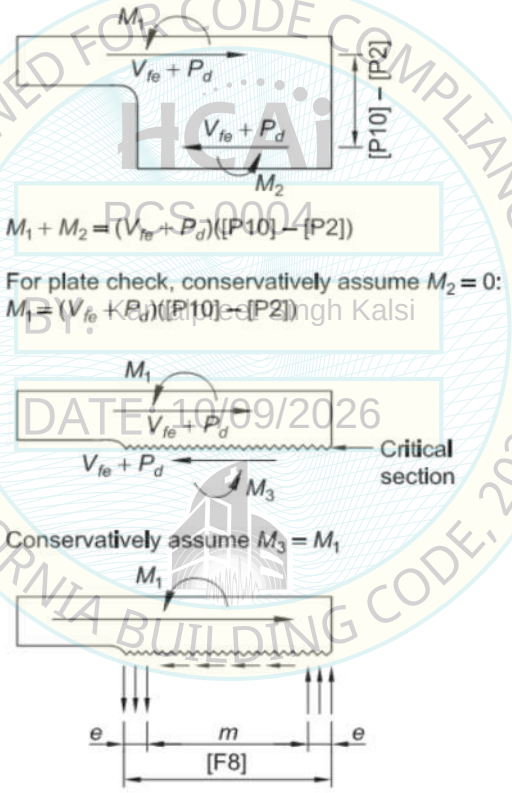


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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**
Fuse plate thickness (see step 2)	$T2$ (in)	0.625	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	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)	121.7	
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)	$n_{b,\ Bot}$	5	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	97.3	
Fuse plate dimension, centerline to gage line	$F7$ (in)	2.5	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	1.185	
Fuse plate thickness (see step 2)	$T2$ (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, $2([F7]+[F8])T2$	A_g (in^2)	4.606	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	230.3	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	207.3	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.47	**OK**

Step 18. (continued)

Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Fuse plate dimension	$F7$ (in)	2.5	
Fuse plate dimension	$F8$ (in)	1.185	
Fuse plate thickness (see step 2)	$T2$ (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, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	3.356	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	P_n (kips)	261.8	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	196.3	
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.50	**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

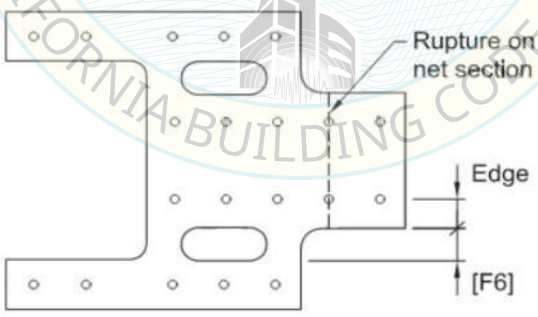


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaljeet Singh Kalsi
UPDATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	1.5	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	5895	
Fuse plate thickness (see step 2)	*T2* (in)	0.625	
Top plate thickness (see step 2)	*T4* (in)	0.625	
Beam depth	d_b (in)	23.6	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	65.9	
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_n	4	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	47.5	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	49.8	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	19.9	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	1.787	
Provided yielding region depth	*F2* (in)	1.750	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	65.6	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	121.7	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.9	**OK**

Fuse plate yielding region geometry			
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.84	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7	
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)	$n_{b,\ Bot}$	5	
Number of bolts in the M region	$n_{m,\ Bot}$	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)	97.3	

Yielding check

Fuse plate dimension	*F4* (in)	1.5	
Fuse plate dimension	*F5* (in)	1.125	
Fuse plate thickness (see step 2)	*T2* (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])(T2)*	A_g (in^2)	3.28125	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	R_n (kips)	164.06	
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	147.66	
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.66	**OK**

Rupture check

Fuse plate dimension	*F4* (in)	1.5	
Fuse plate dimension	*F5* (in)	1.125	
Fuse plate thickness (see step 2)	*T2* (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)(T2)*	A_e (in^2)	2.03125	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	R_n (kips)	158.4	
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	118.8	
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.82	**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

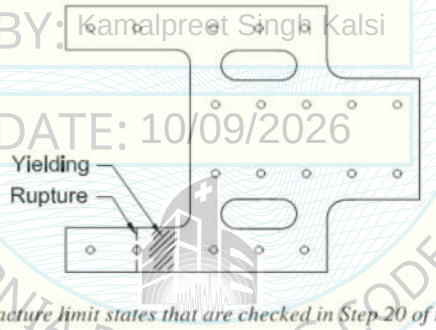


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

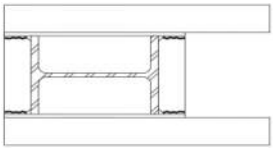
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

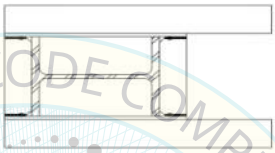
Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	121.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	3.5	
External continuity plate dimension	*C5* (in)	2.25	
Column depth	d_c (in)	16.4	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	1	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}\ T4([C3]-1")$	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**



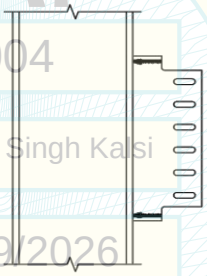
Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



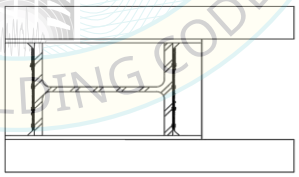
Shear tab to bridge plate weld failure check

Minimum bay width	*B* (in)	360.0	
Column depth	d_c (in)	16.4	
Beam depth	d_b (in)	23.6	
Shear force from gravity	$V_{gravity}$ (kips)	31.5	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	5895	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	65.9	
Distance from alignment line to column	*C1* (in)	4.25	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	1	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



Bridge plate to column flange weld failure check

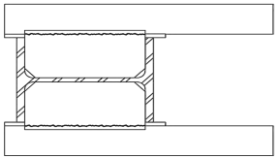
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	4.25	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	1	
Column flange width	b_{fc} (in)	7.12	
Weld 6 effective length, $MIN(b_{fc}-2", 2[C3])$	l_{eff} (in)	2	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



*These checks are N/A when bridge plates do not apply

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

Column depth	d_c (in)	16.4	
Column flange thickness	t_{fc} (in)	0.715	
Weld 4 size	D_{W4} (1/16 in)	0	
Weld 4 effective length, $d_c - 2\ t_{fc}$	l_{eff} (in)	14.97	
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)	N/A	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.000	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{ySP}\ t_{sp}\ (d_c-2t_{fc})$	ϕR_n (kips)	N/A	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	1350	
Column flange width	b_{fc} (in)	7.12	
Beam depth	d_b (in)	23.6	
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)	2.75	
Cover plate thickness	$T1$ (in)	0.875	
Fuse plate thickness	T2 (in)	0.625	
Top plate thickness	$T4$ (in)	0.625	
External continuity plate dimension	$C5$ (in)	2.25	
Fuse plate dimension	$F2$ (in)	1.75	
Fuse plate dimension	$F3$ (in)	5	
Fuse plate dimension	$F4$ (in)	1.5	
Fuse plate dimension	$F6$ (in)	1.5	
Fuse plate dimension	$F8$ (in)	1.185	
Distance from beam centerline to bolt line	$P2$ (in)	2.5	
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	3.97E+04	
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	1.63E+04	
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	2.13E+04	
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	5.02E+04	
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	1.30E+04	
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	3.22E+04	
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	5.45E+06	
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	1.96E+06	
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.36	**OK**

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CERTIFIED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

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

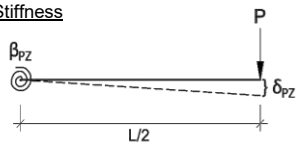
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

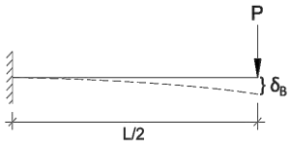


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	23.6
Beam flange thickness	t_{fb} (in)	0.505
Column depth	d_c (in)	16.4
Column flange width	b_{fc} (in)	7.12
Column web thickness	t_{wc} (in)	0.43
Column flange thickness	t_{fc} (in)	0.715
Cover plate overhang	$C3$ (in)	1.0
Cover plate thickness	$T1$ (in)	0.875
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	926.3
Column ratio, $(d_c - t_{fc}) / B$	α	0.044
Beam ratio, $(d_b - t_{fb}) / H$	β	0.107
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	1.44E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	4.41E+04
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	N/A
Beam moment of inertia	I_{xb} (in^3)	1350
Required panel zone spring stiffness , *$60EI_{xb} / B$ for one-sided, $120EI_{xb} / B$ for two-sided*	K_{req} (k-in)	N/A
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	N/A **N/A**

*This check is N/A when connection stiffness is applied in the model analysis

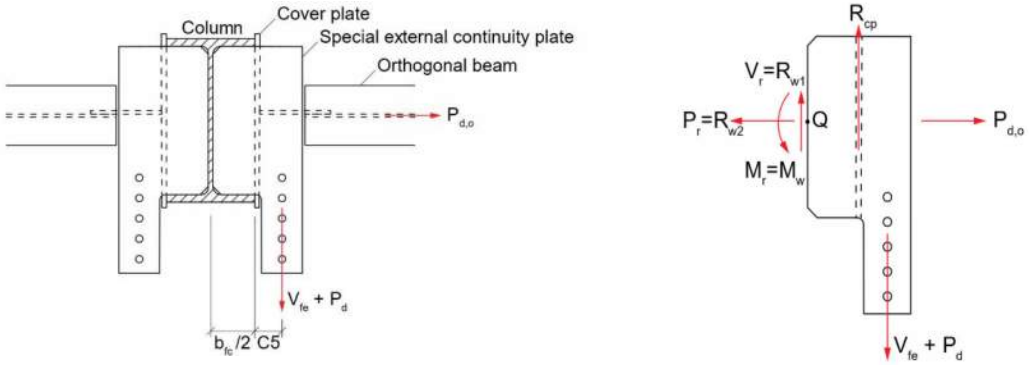
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	121.7
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	7.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	3.5
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
Column flange width	b_{fc} (in.)	7.1
Column depth	d_c (in.)	16.4
Column flange thickness	t_{fc} (in.)	0.72
Column web thickness	t_{wc} (in.)	0.43
External continuity plate thickness	$T5$ (in)	0.625
External continuity plate gage dimension	$C5$ (in.)	2.25
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	35.02	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	9.36	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
	Special detailing not required		

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	1.12	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	31.3	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	8.85	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply	Special detailing not required		



Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size			
Orthogonal beam depth	$d_{b,o}$ (in)		
In plane moment frame beam 1		W24X55	
In plane moment frame beam 2		NA	
Max in-plane moment frame beam depth	d_b (in)	23.6	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0	
New continuity plate required for deeper cantilever beam		N/A	
New continuity plate width past column flange	C4 (in)	3.375	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A	**N/A**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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.)	44	
Beam depth	d_b (in.)	24.7	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	146.0	**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}	5.9	**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.)	44	
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)*	224.7	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	53.3	
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), Factor based on code and member type, $F(E / (R_{yc} F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	
	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	4.57	**OK**

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

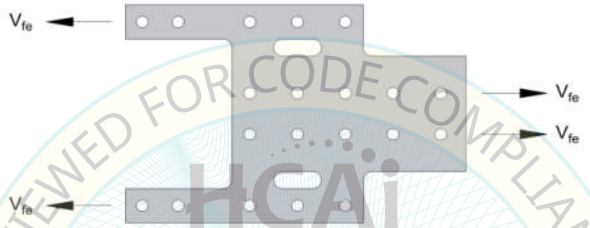
Column plastic section modulus	Z_c (in^3)	432	
Column design axial load	P_u (kips)	224.7	
Column area	A_{gc} (in^2)	48.8	
Story Height	H (in)	216.0	
Column depth	d_c (in)	22.5	
Beam depth	d_b (in)	24.7	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c (F_{yc} - P_u/A_{gc})(H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	4.87E+04	
Beam plastic section modulus	Z_b (in^3)	418	
Minimum 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)	4.55E+04	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.07	**OK**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)	7320	
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**
Fuse plate thickness	$T2$ (in)	1.125	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	1.125	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	1.5	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	24.7	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	404.6	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	809.3	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	809.3	
Beam area	A_{gb} (in^2)	43.0	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	8.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	4.0	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	813.3	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	89	
Fuse plate thickness	$T2$ (in)	1.125	
Top plate thickness	$T4$ (in)	1.125	
External continuity plate thickness	$T5$ (in)	1.5	
Beam depth	d_b (in)	24.7	
Column depth	d_c (in)	22.5	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	975.9	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	1.5	
Cover plate overhange dimension	$C3$ (in)	7	
Column depth	d_c (in)	22.5	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	36.5	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	1642.5	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	1642.5	
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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	30.70	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	1381.5	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	1381.5	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.71	**OK**

Step 3. (continued)

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, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	3.19	**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 5a. Determine the number of bolts for each bolt line on the 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)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	4.0	
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)	62.6	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	6.5	
Number of bolts per line (AISC 358 15.2(1))	n_b	7	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Top plate thickness	$T4$ (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	3.843	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.2031	
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	3	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	408.6	
Beam flange thickness	t_{fb} (in)	1.09	
Top plate thickness	$T4$ (in)	1.125	
Governing material thickness, $\min(t_{fb}, T4)$	t_{min} (in)	1.1	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1339.1	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	988.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1065.4	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	584.5	
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	584.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	438.4	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.93	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1926	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	1.125	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	37.3	
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	7	
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)	189.8	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	161.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.23	**OK**

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

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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	4.8	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	7	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Fuse plate thickness	$T2$ (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	3.843	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.2	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	4	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	3	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	404.6	
Beam flange thickness	t_{fb} (in)	1.09	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1339.1	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	988.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1065.4	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	584.5	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	584.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	438.4	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.92	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kemalettin Dağlıalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 5b. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1926	
Bolt group demand from wind per line, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	37.3	
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 on the bottom flange	$n_{b,Bot}$	7	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	189.8	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	161.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.23	**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 (in)	1.625	
Beam flange width factor, 0 *if* $b_{fb} < b_{fc}$, 1.0 *otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	7	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	10.107	
Distance from alignment line to column	*C1* (in)	10.125	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	809.3	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	8.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	4	
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	
Fuse plate thickness	*T2* (in)	1.125	
Top plate thickness	*T4* (in)	1.125	
External continuity plate thickness	*T5* (in)	1.5	
Cover plate thickness	*T1* (in)	1.5	
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3	
Cover plate height, d_b + 2[C6], d_b + [C6] - T5 or just d_b - 2T5 for cap plate and ortho. cant./drag condition	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)*	1.1250	
Weld 1 length, h_{cp} - 2l_{hb1}	l_{W1} (in)	28.45	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), *T5 + T1 + [C6]*	l_{We1} (in)	6.000	
Weld 1 size	D_{W1} (1/16 in)	16	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
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)	1.17	
Vertical distance from center of rotation to geometric center	e_y (in)	0.03	

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.

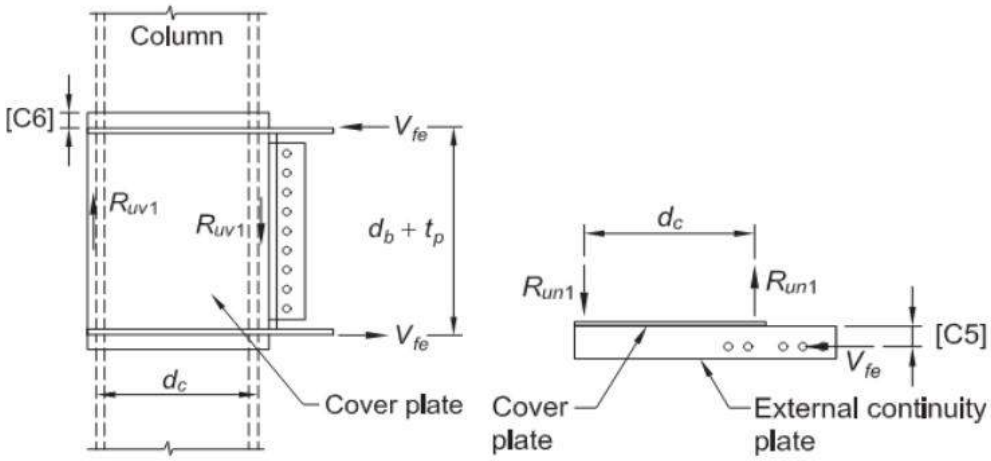
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	37537	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	17.23	22.87	54.20	393.9
W4TL	18.63	7.69	35.68	143.2
W1LB	18.65	22.22	48.23	414.5
W4BR	17.31	7.64	29.48	132.2
W1RC	10.08	12.54	1.80	126.4
W4TC	15.19	6.47	5.18	98.3
W1LC	12.43	13.13	1.46	163.2
W4BC	15.13	6.47	5.20	97.9
		22.87	54.20	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	22.9	
Orientation of Resultant Force	θ_r (degrees)	54.2	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.57	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	22.4	
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)	22.4	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	0.87	
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)	30.14	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	*D/C*	0.74	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	809.3	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	4	
Column depth	d_c (in)	22.5	
Column flange thickness	t_{fc} (in)	1.36	
Cover plate overhang dimension	*C3* (in)	7	
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} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	9.24	
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)	22.3	
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)	10.6	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	24.7	
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*	0.89	**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)*	809.3	
External continiuty plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	2541.5	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	813.3	
External continuity plate thickness (see step 2)	*T5* (in)	1.500	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	1642.5	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}\,T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	22481.7	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.61	**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)*	809.3	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	4.0	
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)	108.1	
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)	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)	2005.5	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	804.5	
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)	2911.4	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	2183.6	
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	*D/C*	0.37	**OK**

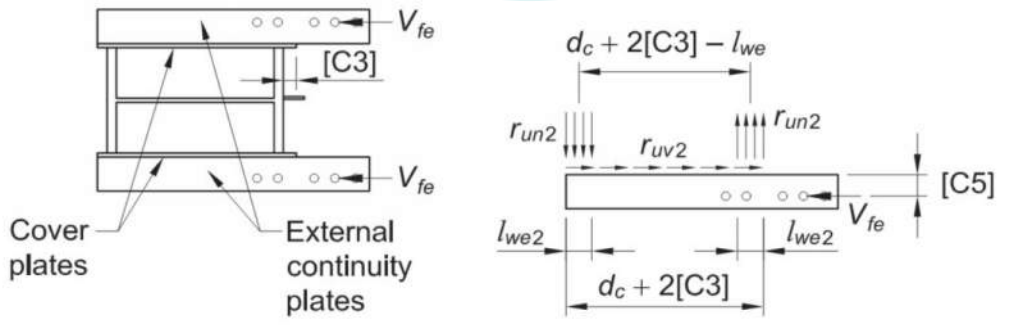


Fig. C-15.5. Weld demands for external continuity plate welds.

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
DESIGNED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	4.0	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	4	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	233.5	
External continuity plate thickness (see step 2)	*T5* (in)	1.5	
External continuity plate width	*C4* (in)	4.75	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, *T5([C4]-($d_{hole,std}$+1/16))*	A_n (in^2)	5.156	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	335.2	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	251.4	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.93	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	5	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	291.9	
External continuity plate dimension	*C2 (in)*	2.5	
Cover plate overhang dimension	*C3* (in)	7	
External continuity plate dimension	*C5* (in)	3.125	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	9.5	
External continuity plate thickness (see step 2)	*T5* (in)	1.5	
External continuity plate width	*C4* (in)	4.75	
Diagonal bonus (AISC 360 B4.3b), *[C9]^2 (T5)/4[C5]*	*Bonus* (in)	10.830	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, T5*([C4]-$d_{hole,std}$-1/16) + Bonus*	A_n (in^2)	15.986	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	1039.1	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	779.3	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.37	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	6	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	350.3	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	2.5	
Cover plate overhang dimension	*C3* (in)	7	
External continuity plate dimension	*C5* (in)	3.125	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	12.875	
External continuity plate thickness (see step 2)	*T5* (in)	1.5	
External continuity plate width	*C4* (in)	4.75	
Diagonal bonus (AISC 360 B4.3b), *[C7]^2 (T5)/(4[C5])*	*Bonus* (in)	19.892	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, *(T5)([C4]-$d_{hole,std}$-1/16) + Bonus*	A_n (in^2)	25.048	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	1628.1	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	1221.1	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.29	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	5	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	291.9	
External continuity plate width	*C4* (in)	4.75	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	9.5	
External continuity plate thickness (see step 2)	*T5* (in)	1.5	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	7.125	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	14.250	
Net tension area, A_{gt} - *((T5)*($d_{hole,std}$+1/16))*	A_{nt} (in^2)	5.156	
Net shear area, A_{gv}	A_{nv} (in^2)	14.250	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	762.7	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	572.0	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.51	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	6	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	350.3	
Bolt spacing (see schedule)	s (in)	3.375	
External continuity plate dimension	*C2 (in)*	2.5	
Cover plate overhang dimension	*C3* (in)	7	
External continuity plate width	*C4* (in)	4.75	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	12.875	
External continuity plate thickness (see step 2)	*T5* (in)	1.5	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	7.125	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	19.313	
Net tension area, A_{gt}-*((T4)(d_{hole}+1/16))*	A_{nt} (in^2)	5.156	
Net shear area, A_{gv}	A_{nv} (in^2)	19.313	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	914.5	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	685.9	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.51	**OK**

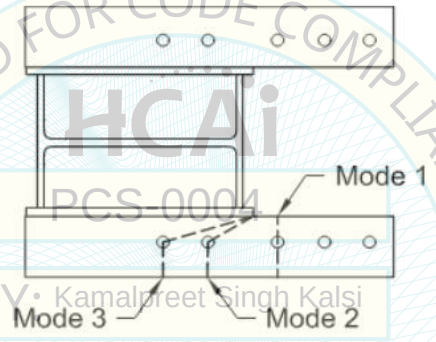


Fig. C-15.6. Tension rupture limit state for external continuity plate.

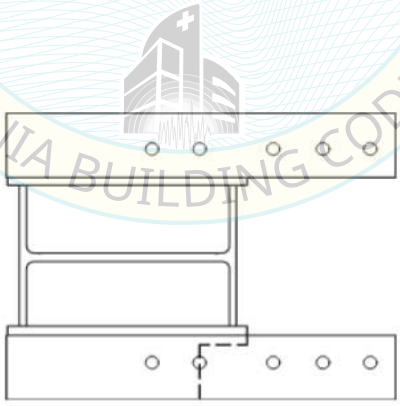


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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)	404.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	4.0	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	817.3	
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	7	
Length of connection on beam	$B4$ (in)	21.875	
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)	47.688	
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)	29.089	
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)	1565.8	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1174.3	
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)	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 provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	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 (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	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1}\ t_{min}\ F_{u,min}\ (n_n - 1)$	R_{nt1} (kips)	597.2	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2}\ t_{min}\ F_{u,min}$	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), $1.5 T_b \mu D_u h_f n_s$	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)	10.125	
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)	1659.7	

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	$T3$ (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 (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, $T3 l_{tab}$	A_{gv} (in^2)	11.88	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) T3$	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), $(0.75) 0.6 F_{uST} A_{nv}$	ϕR_{nv} (kips)	251.37	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75) F_{uST} A_e$	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)	1659.74	
Shear tab gross area, $T3 l_{tab}$	A_{gv} (in^2)	11.88	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	56.41	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0) 0.6 F_{yST} A_{gv}$	ϕR_{nv} (kips)	356.25	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9) F_{yST} A_{gv}$	ϕR_{nn} (kips)	534.38	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9) F_{yST} Z_{tab}$	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	0.79	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	19	
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)	8.6	
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)	10.125	
Size of weld in 1/16^{ths} of an inch	D_{w3}	15	
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)	1659.7	
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)	38.9	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	39.8	
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)	41.8	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.95	**OK**
This check is N/A when bridge plates do not apply			

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 (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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (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)

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	4.0	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	408.6	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	1.125	
Top plate dimension (see schedule)	$P8$ (in)	24	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	27.000	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	810.0	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	810.0	
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	7	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	15.188	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	592.3	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	444.2	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.92	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

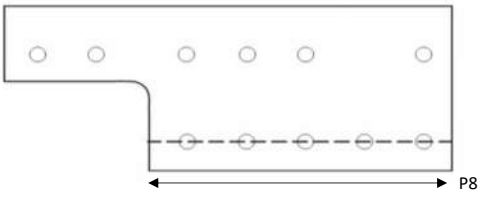


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile 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)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	4.0	
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	408.6	

Yielding check (LS 1)

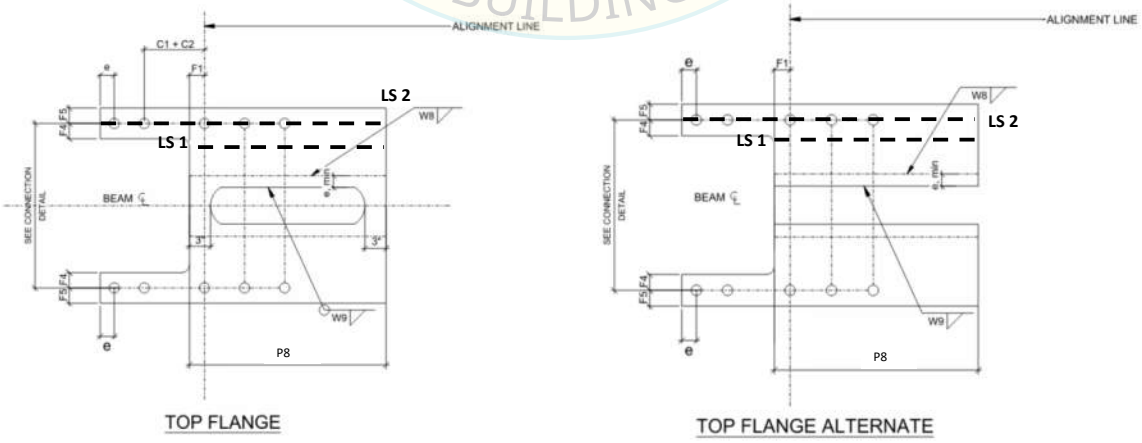
Top plate thickness (see step 2)	*T4* (in)	1.125	
Top plate dimension (see schedule)	*P8* (in)	19.5	
Fuse plate dimension	*F1 (in)*	1.88	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	21.938	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	658.1	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	658.1	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.62	**OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Number of oversized bolts in Zone M (see step 5)	n_m	3	
Number of oversized bolts on top plate (see step 5)	n_b	7	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.5	
Bolt spacing (see schedule)	*s* (in)	3.375	
Fuse plate dimension	*F1 (in)*	1.88	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	38.5	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	43.313	
Number of oversized bolts on top plate (see step 5)	n_b	7	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	31.992	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	1247.7	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	935.8	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.44	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	408.6	
Beam flange width	b_{fb} (in)	12.9	
Weld 8 size	D_{W8} (1/16 in)	10	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.63	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	18.25	
Weld 9 size	D_{W9} (1/16 in)	10	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	13.50	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	12.9	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	13.9	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.92	**OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	4.0	
Total number of bolts (see step 5)	n_b	7	
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)	175.1	

Yielding check

Top plate dimension	*P4* (in)	2	
Top plate dimension	*P5* (in)	2.75	
Top plate thickness (see step 2s)	*T4* (in)	1.125	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	5.344	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	267.2	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	240.5	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.73	**OK**

Rupture check

Top plate dimension	*P4* (in)	2	
Top plate dimension	*P5* (in)	2.75	
Top plate thickness (see step 2)	*T4* (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)(T4)*	A_e (in^2)	3.656	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	237.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	178.2	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.98	**OK**

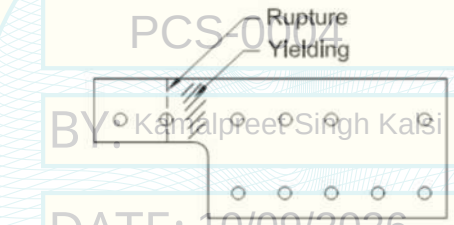


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

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)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	4.0	
Top plate thickness (see step 2)	*T4* (in)	1.125	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	*P10* (in)	10.825	
Inside length of the top plate	*P8* (in)	24.00	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	13.453	
Dimension, *([P8] - m) / 2*	*e* (in)	5.273	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.0	
Provided P2 dimension	*P2* (in)	2.75	**OK**

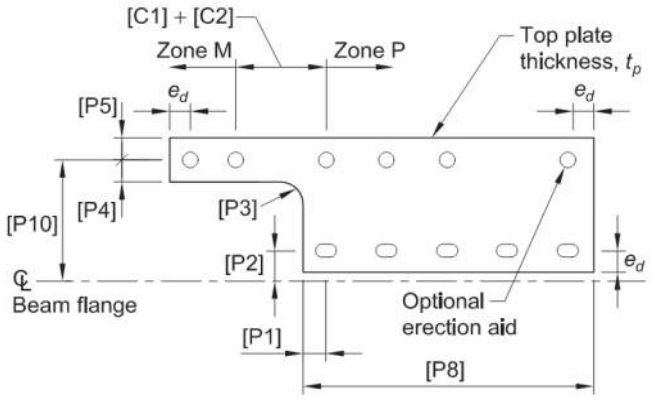


Fig. 15.6. Top plate detail.

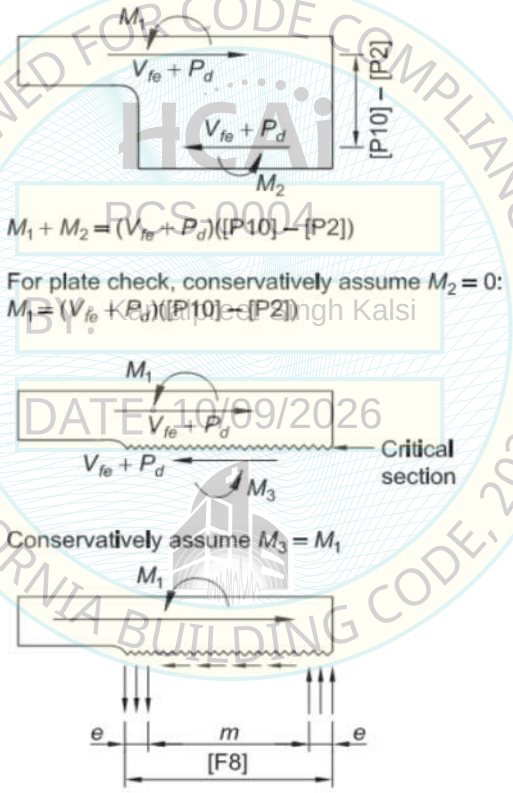


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS 0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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**
Fuse plate thickness (see step 2)	$T2$ (in)	1.125	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	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)	$n_{b,\ Bot}$	7	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	346.8	
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)	$T2$ (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, $2([F7]+[F8])T2$	A_g (in^2)	13.669	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	683.4	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	615.1	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.56	**OK**

Step 18. (continued)

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)	$T2$ (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, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	10.294	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	P_n (kips)	802.9	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	602.2	
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.58	**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

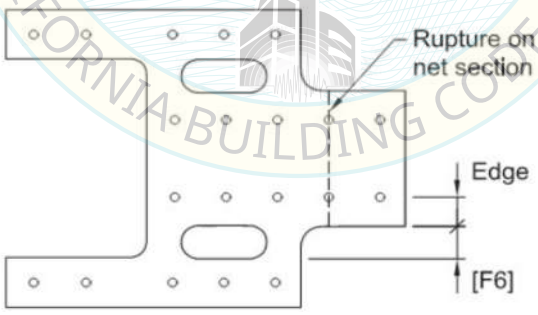


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
UPDATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	2.75	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900	
Fuse plate thickness (see step 2)	*T2* (in)	1.125	
Top plate thickness (see step 2)	*T4* (in)	1.125	
Beam depth	d_b (in)	24.7	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	163.9	
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_n	4	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	108.5	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	113.9	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	45.6	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	*F2* $_{max}$ (in)	3.350	
Provided yielding region depth	*F2* (in)	3.250	**OK**
Fuse yield force (one side), *2(0.6F* $_{yFuse}$ *)[F2]T2*	V_y (kips)	219.4	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	404.6	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.8	**OK**
Fuse plate yielding region geometry			
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	*(F6/F2)* $_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.82	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

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)	$n_{b,\ Bot}$	7
Number of bolts in the M region	$n_{m,\ Bot}$	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)	346.8

Yielding check

Fuse plate dimension	*F4* (in)	2	
Fuse plate dimension	*F5* (in)	2.25	
Fuse plate thickness (see step 2)	*T2* (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])(T2)*	A_g (in^2)	9.5625	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	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*	0.81	**OK**

Rupture check

Fuse plate dimension	*F4* (in)	2	
Fuse plate dimension	*F5* (in)	2.25	
Fuse plate thickness (see step 2)	*T2* (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, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	6.1875	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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*	0.96	**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

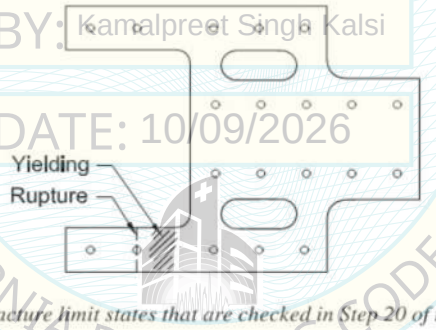


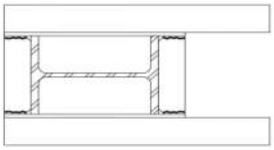
Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

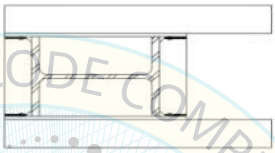
Bridge plate calculations

Bridge plate tension failure check



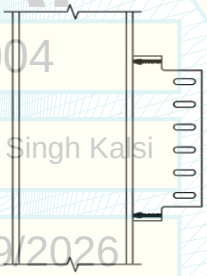
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	404.6	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	8.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	4	
External continuity plate dimension	*C5* (in)	3.125	
Column depth	d_c (in)	22.5	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	7	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}$ *T4([C3]-1")*	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**

Bridge plate to cover plate weld failure check



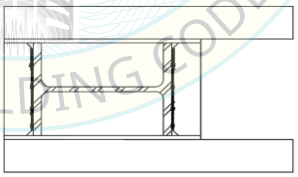
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16$ *([C3]-1")(2 sides)*	ϕR_n (kips)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

Shear tab to bridge plate weld failure check



Minimum 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)	10.125	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	7	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

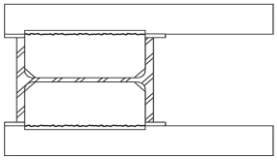
Bridge plate to column flange weld failure check



Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	10.125	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	7	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

*These checks are N/A when bridge plates do not apply

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 *- 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)	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), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	370.9	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.7	**OK**

*This check is N/A when shear plates do not apply

HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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)	4580	
Column flange width	b_{fc} (in)	12.4	
Beam depth	d_b (in)	24.7	
Bay width	B (in)	360.0	
Number of bolts in zone M	M	3	
Number of bolts in zone P	P	4	
Bolt spacing	s (in)	3.375	
Cover plate thickness	$T1$ (in)	1.5	
Fuse plate thickness	T2 (in)	1.125	
Top plate thickness	$T4$ (in)	1.125	
External continuity plate dimension	$C5$ (in)	3.125	
Fuse plate dimension	$F2$ (in)	3.25	
Fuse plate dimension	$F3$ (in)	7.125	
Fuse plate dimension	$F4$ (in)	2.0	
Fuse plate dimension	$F6$ (in)	2.75	
Fuse plate dimension	$F8$ (in)	3.325	
Distance from beam centerline to bolt line	$P2$ (in)	2.75	
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	8.58E+04	
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	2.98E+04	
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	3.99E+04	
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	5.16E+04	
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	2.23E+04	
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	5.27E+04	
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	1.04E+07	
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	6.64E+06	
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.64	**OK**

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CERTIFIED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

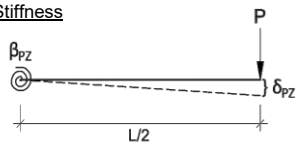
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

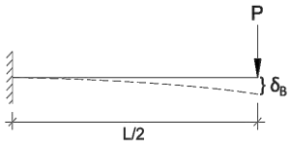


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.7
Beam flange thickness	t_{fb} (in)	1.09
Column depth	d_c (in)	22.5
Column flange width	b_{fc} (in)	12.4
Column web thickness	t_{wc} (in)	0.75
Column flange thickness	t_{fc} (in)	1.36
Cover plate overhang	$C3$ (in)	7.0
Cover plate thickness	$T1$ (in)	1.5
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	3121.5
Column ratio, $(d_c - t_{fc}) / B$	α	0.059
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)	5.05E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	2.89E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	N/A
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)	N/A
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	N/A **N/A**

*This check is N/A when connection stiffness is applied in the model analysis

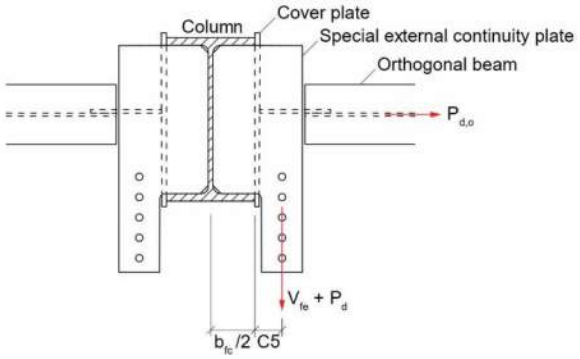
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

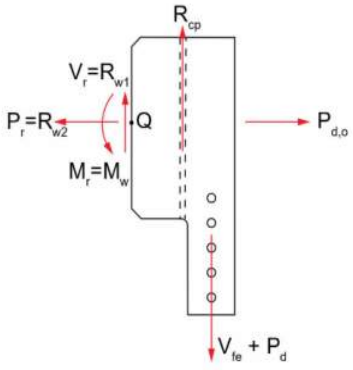
Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	809.3
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	8.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	4.0
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
Column flange width	b_{fc} (in.)	12.4
Column depth	d_c (in.)	22.5
Column flange thickness	t_{fc} (in.)	1.36
Column web thickness	t_{wc} (in.)	0.75
External continuity plate thickness	$T5$ (in)	1.500
External continuity plate gage dimension	$C5$ (in.)	3.13
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	146.72	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	29.67	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
		Special detailing not required	

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	1.86	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	132.3	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	28.17	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply		Special detailing not required	





Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size			
Orthogonal beam depth	$d_{b,o}$ (in)		
In plane moment frame beam 1		W24X146	
In plane moment frame beam 2		W24X146	
Max in-plane moment frame beam depth	d_b (in)	24.7	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0	
New continuity plate required for deeper cantilever beam		N/A	
New continuity plate width past column flange	C4 (in)	4.75	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A	**N/A**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

4 ADDITIONAL EXAMPLE CONNECTIONS

DF150 Detailed Connection Design (in-plane drag load) 4.1
DF151 Detailed Connection Design (orthogonal drag load) 4.2
DF152 Detailed Connection Design (orthogonal cantilever beam) 4.3
DF153 Detailed Connection Design 4.4

stiffness checks: Mpr<Mp) 4.7
DF251 Detailed Connection Design (in-plane cantilever beam)

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
PCSS0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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.)	44	
Beam depth	d_b (in.)	24.7	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	146.0	**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}	5.9	**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.)	44	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	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), Factor based on code and member type, $F(E / (R_{yc} F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	
	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	6.45	**OK**

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

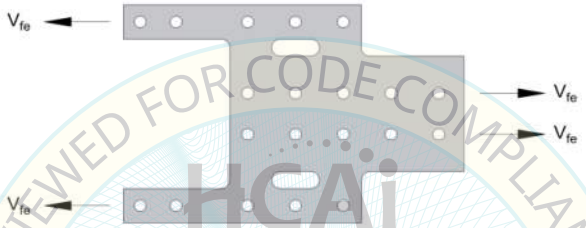
Column plastic section modulus	Z_c (in^3)	307	
Column design axial load	P_u (kips)	271.9	
Column area	A_{gc} (in^2)	35.9	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.7	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c^*(F_{yc}-P_u/A_{gc})^*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	3.23E+04	
Beam plastic section modulus	Z_b (in^3)	418	
Minimum 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)	2.27E+04	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.42	**OK**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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**
Fuse plate thickness	$T2$ (in)	1.125	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	1.25	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	1.125	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	24.7	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	403.7	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	403.7	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	403.7	
Beam area	A_{gb} (in^2)	43.0	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	25.0	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	428.7	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	56	
Fuse plate thickness	$T2$ (in)	1.125	
Top plate thickness	$T4$ (in)	1.25	
External continuity plate thickness	$T5$ (in)	1.125	
Beam depth	d_b (in)	24.7	
Column depth	d_c (in)	21.7	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	524.4	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	0.875	
Cover plate overhange dimension	$C3$ (in)	2.625	
Column depth	d_c (in)	21.7	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	26.95	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	707.4	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	707.4	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.61	**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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	30.70	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (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	0.65	**OK**

Step 3. (continued)

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, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	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 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	25.0	
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)	62.6	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	6.8	
Number of bolts per line (AISC 358 15.2(1))	n_b	7	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Top plate thickness	$T4$ (in)	1.25	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	3.450	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.9705	
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	3	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	428.7	
Beam flange thickness	t_{fb} (in)	1.09	
Top plate thickness	$T4$ (in)	1.25	
Governing material thickness, $\min(t_{fb}, T4)$	t_{min} (in)	1.1	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1339.1	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	988.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1065.4	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	584.5	
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	584.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	438.4	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.98	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3068	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	1.25	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	59.3	
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	7	
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)	189.8	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	161.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.37	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	403.7	
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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	4.8	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	7	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Fuse plate thickness	$T2$ (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	3.834	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.2	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	4	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	3	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	403.7	
Beam flange thickness	t_{fb} (in)	1.09	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1339.1	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	988.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1065.4	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	584.5	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	584.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	438.4	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.92	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kemalpaşa Çağıltalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 5b. (continued)

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, $M_u/(2(d_b+T2/2 + T4/2))$	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 on the bottom flange	$n_{b,Bot}$	7	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	189.8	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	161.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.37	**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 (in)	1.625	
Beam flange width factor, 0 *if* $b_{fb} < b_{fc}$, 1.0 *otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	2.625	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	5.732	
Distance from alignment line to column	*C1* (in)	5.75	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	25	
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	
Fuse plate thickness	$T2$ (in)	1.125	
Top plate thickness	$T4$ (in)	1.25	
External continuity plate thickness	$T5$ (in)	1.125	
Cover plate thickness	$T1$ (in)	0.875	
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2T5$ for cap plate and ortho. cant./drag condition	h_{cp} (in)	30.7	
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)	0.6875	
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	29.33	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $T5 + T1 + [C6]$	l_{We1} (in)	5.000	
Weld 1 size	D_{W1} (1/16 in)	9	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
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)	1.06	
Vertical distance from center of rotation to geometric center	e_y (in)	0.34	

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.

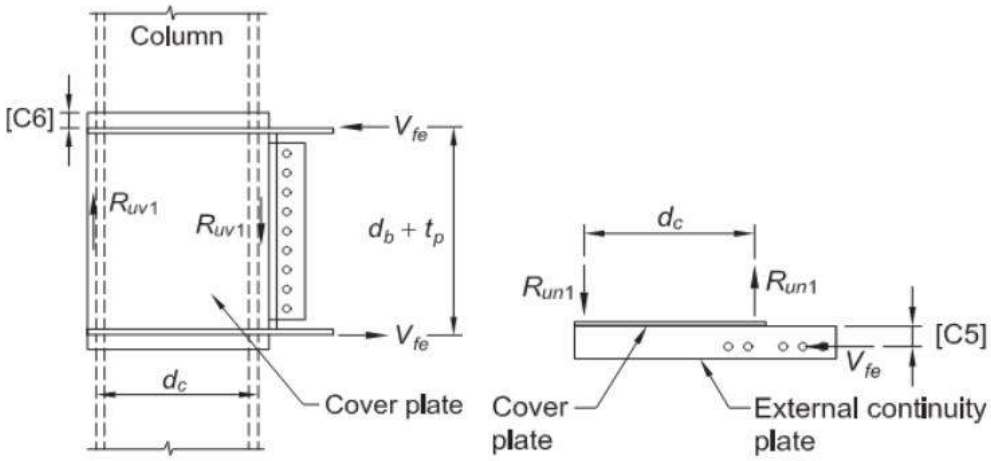
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	24594	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	17.67	13.39	56.35	236.5
W4TL	18.82	5.93	34.84	111.6
W1LB	18.40	12.93	49.67	237.9
W4BR	17.10	5.98	30.33	102.2
W1RC	9.81	8.16	3.71	80.1
W4TC	15.49	5.02	4.65	77.8
W1LC	11.92	8.45	3.05	100.8
W4BC	14.81	5.00	4.86	74.1
		13.39	56.35	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	13.4	
Orientation of Resultant Force	θ_r (degrees)	56.3	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.59	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	10.0	
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)	10.0	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.25	
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)	17.12	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.58	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	25	
Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang dimension	*C3* (in)	2.625	
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} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	3.96	
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)	15.9	
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)	14.8	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	21.7	
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*	0.97	**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)*	403.7	
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	1286.0	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	428.7	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	909.6	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	9192.3	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.61	**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)*	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	25.0	
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)	89.5	
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)	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)	1328.2	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	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)	2100.0	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1575.0	
Weld 2 demand/capacity check for cap plate configuration, $R_u/\phi R_n$	*D/C*	0.27	**OK**

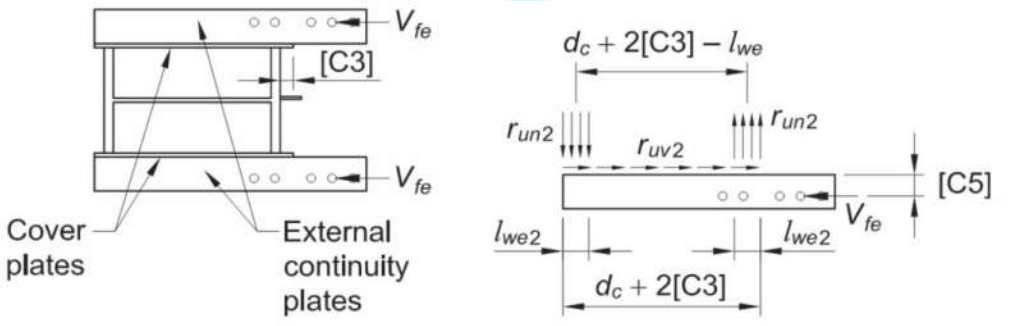


Fig. C-15.5. Weld demands for external continuity plate welds.

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
DESIGNED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	25.0	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	4	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	245.0	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate width	*C4* (in)	6	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, $T5([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	5.273	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	342.8	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	257.1	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.95	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	5	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	306.2	
External continuity plate dimension	*C2 (in)*	0.75	
Cover plate overhang dimension	*C3* (in)	2.625	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	3.375	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate width	*C4* (in)	6	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (T5)/4[C5]$	*Bonus* (in)	1.068	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $T5([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	6.341	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	412.2	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	309.1	
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, $n_{rup1} + 2$	n_{rup}	6	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	367.4	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	0.75	
Cover plate overhang dimension	*C3* (in)	2.625	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	6.75	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate width	*C4* (in)	6	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (T5)/(4[C5])$	*Bonus* (in)	4.271	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, $(T5)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	9.545	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	620.4	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	465.3	
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, $n_{rup1} + 1$	n_{rup}	5	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	306.2	
External continuity plate width	*C4* (in)	6	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	3.375	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	6.750	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	3.797	
Net tension area, $A_{gt} - ((T5)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	5.273	
Net shear area, A_{gv}	A_{nv} (in^2)	3.797	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	456.7	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	342.5	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.89	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	6	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	367.4	
Bolt spacing (see schedule)	s (in)	3.375	
External continuity plate dimension	*C2 (in)*	0.75	
Cover plate overhang dimension	*C3* (in)	2.625	
External continuity plate width	*C4* (in)	6	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	6.75	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	6.750	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	7.594	
Net tension area, A_{gt}-*((T4)(d_{hole}+1/16))*	A_{nt} (in^2)	5.273	
Net shear area, A_{gv}	A_{nv} (in^2)	7.594	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	570.6	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	427.9	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.86	**OK**

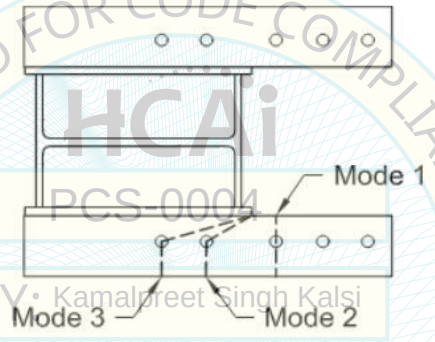


Fig. C-15.6. Tension rupture limit state for external continuity plate.

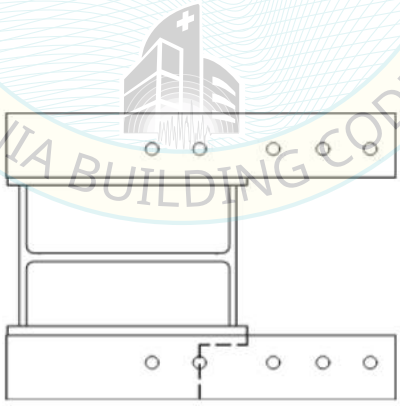


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	25.0	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	857.3	
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	7	
Length of connection on beam	$B4$ (in)	21.875	
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)	47.688	
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)	29.089	
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)	1530.4	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1147.8	
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.75	**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 provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	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 (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	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1}\ t_{min}\ F_{u,min}\ (n_n - 1)$	R_{nt1} (kips)	477.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2}\ t_{min}\ F_{u,min}$	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), $1.5 T_b \mu D_u h_f n_s$	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)	5.75	
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)	930.1	

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	$T3$ (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 (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, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) T3$	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), $(0.75) 0.6 F_{uST} A_{nv}$	ϕR_{nv} (kips)	201.09	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75) F_{uST} A_e$	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)	930.05	
Shear tab gross area, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	45.13	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0) 0.6 F_{yST} A_{gv}$	ϕR_{nv} (kips)	285.00	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9) F_{yST} A_{gv}$	ϕR_{nn} (kips)	427.50	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9) F_{yST} Z_{tab}$	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	0.71	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	19	
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)	8.5	
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)	5.75	
Size of weld in 1/16^{ths} of an inch	D_{w3}	11	
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)	930.1	
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)	26.7	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	28.0	
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)	30.6	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.92	**OK**
This check is N/A when bridge plates do not apply			

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 (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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (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)

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	25.0	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	428.7	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	1.25	
Top plate dimension (see schedule)	$P8$ (in)	23.75	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	29.688	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	890.6	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	890.6	
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	7	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	16.563	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	645.9	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	484.5	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.88	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

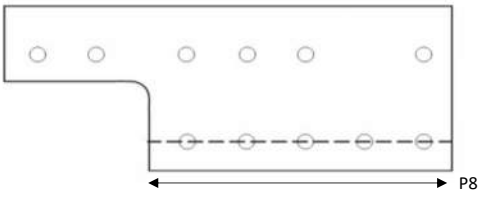


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	25.0	
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	428.7	

Yielding check (LS 1)

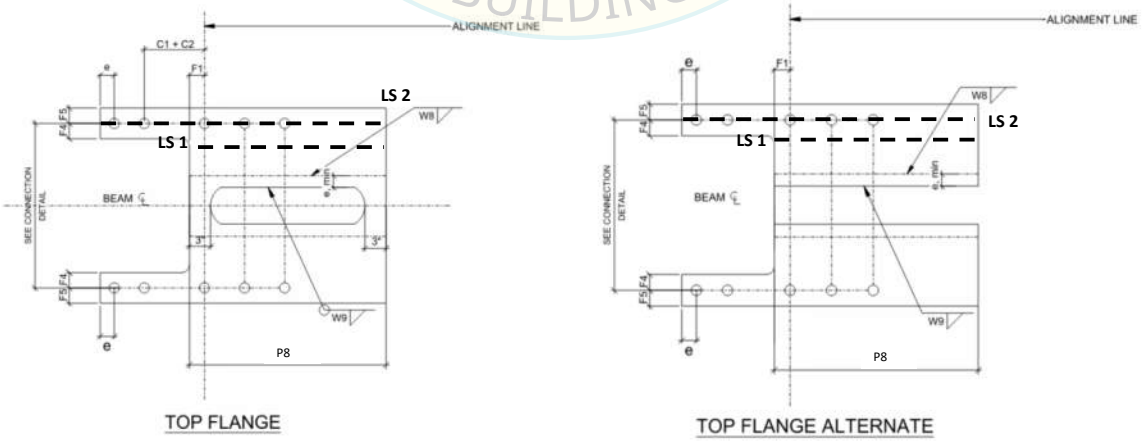
Top plate thickness (see step 2)	*T4* (in)	1.25	
Top plate dimension (see schedule)	*P8* (in)	19.5	
Fuse plate dimension	*F1 (in)*	1.75	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	24.375	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	731.3	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	731.3	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.59	**OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Number of oversized bolts in Zone M (see step 5)	n_m	3	
Number of oversized bolts on top plate (see step 5)	n_b	7	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.5	
Bolt spacing (see schedule)	*s* (in)	3.375	
Fuse plate dimension	*F1 (in)*	1.75	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	32.5	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	40.625	
Number of oversized bolts on top plate (see step 5)	n_b	7	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	28.047	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	1093.8	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	820.4	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.52	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	428.7	
Beam flange width	b_{fb} (in)	12.9	
Weld 8 size	D_{W8} (1/16 in)	10	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.63	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	18.25	
Weld 9 size	D_{W9} (1/16 in)	10	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	13.50	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	13.5	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	13.9	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.97	**OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	25.0	
Total number of bolts (see step 5)	n_b	7	
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)	183.7	

Yielding check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	2.75	
Top plate thickness (see step 2s)	*T4* (in)	1.25	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	5.781	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	289.1	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	260.2	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.71	**OK**

Rupture check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	2.75	
Top plate thickness (see step 2)	*T4* (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)(T4)*	A_e (in^2)	3.906	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	253.9	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	190.4	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.96	**OK**

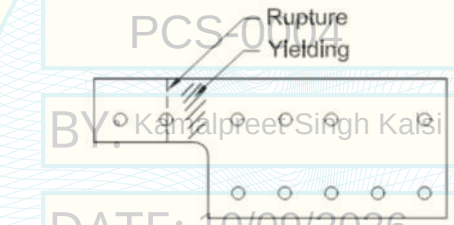


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0024
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

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)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	25.0	
Top plate thickness (see step 2)	*T4* (in)	1.25	
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)	23.75	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	12.701	
Dimension, *([P8] - m) / 2*	*e* (in)	5.524	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.0	
Provided P2 dimension	*P2* (in)	3	**OK**

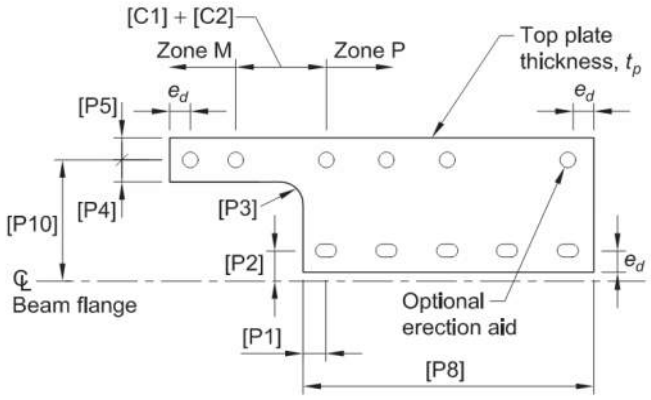


Fig. 15.6. Top plate detail.

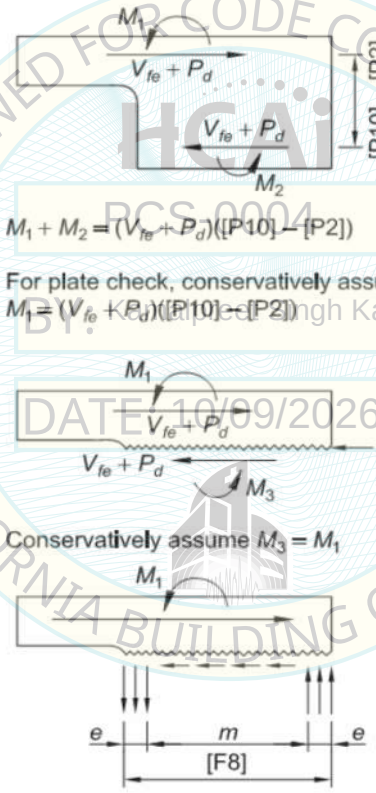


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS 0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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**
Fuse plate thickness (see step 2)	$T2$ (in)	1.125	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	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)	403.7	
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)	$n_{b,\ Bot}$	7	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	346.0	
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)	$T2$ (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, $2([F7]+[F8])T2$	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	0.63	**OK**

Step 18. (continued)

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)	$T2$ (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, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	8.888	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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	0.67	**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

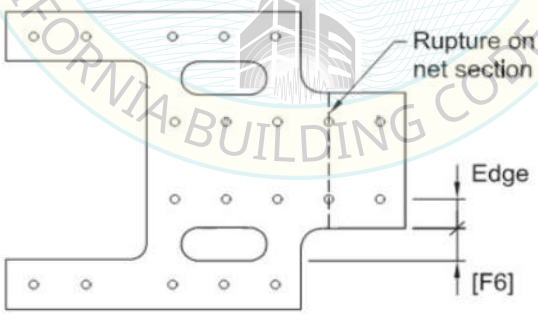


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
UPDATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	2.75	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900	
Fuse plate thickness (see step 2)	*T2* (in)	1.125	
Top plate thickness (see step 2)	*T4* (in)	1.25	
Beam depth	d_b (in)	24.7	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	161.7	
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_n	4	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	108.5	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	113.9	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	45.6	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	*F2* $_{max}$ (in)	3.342	
Provided yielding region depth	*F2* (in)	3.250	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	219.4	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	403.7	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.8	**OK**

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	*(F6/F2)* $_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.82	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7	
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)	$n_{b,\ Bot}$	7	
Number of bolts in the M region	$n_{m,\ Bot}$	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)	346.0	

Yielding check

Fuse plate dimension	*F4* (in)	1.875	
Fuse plate dimension	*F5* (in)	2.375	
Fuse plate thickness (see step 2)	*T2* (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])(T2)*	A_g (in^2)	9.5625	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	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*	0.80	**OK**

Rupture check

Fuse plate dimension	*F4* (in)	1.875	
Fuse plate dimension	*F5* (in)	2.375	
Fuse plate thickness (see step 2)	*T2* (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, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	6.1875	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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*	0.96	**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

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

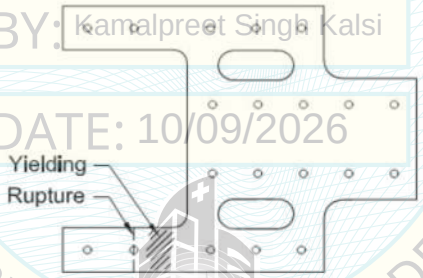


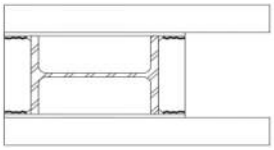
Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

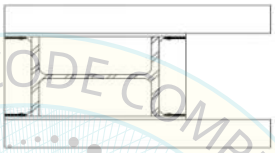
Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	25	
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)	N/A	
Cover plate overhang	*C3* (in)	2.625	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}$ *T4([C3]-1")*	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**



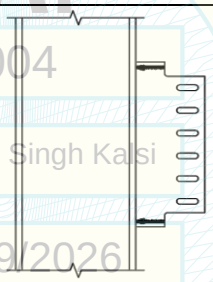
Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



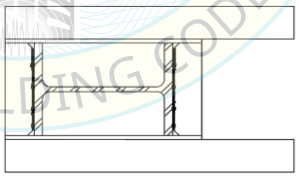
Shear tab to bridge plate weld failure check

Minimum 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)	5.75	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2.625	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



Bridge plate to column flange weld failure check

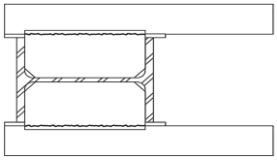
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	5.75	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2.625	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	5.25	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



*These checks are N/A when bridge plates do not apply

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), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	296.7	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**



*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	4580
Column flange width	b_{fc} (in)	12.4
Beam depth	d_b (in)	24.7
Bay width	B (in)	360.0
Number of bolts in zone M	M	3
Number of bolts in zone P	P	4
Bolt spacing	s (in)	3.375
Cover plate thickness	$T1$ (in)	0.875
Fuse plate thickness	T2 (in)	1.125
Top plate thickness	$T4$ (in)	1.25
External continuity plate dimension	$C5$ (in)	3
Fuse plate dimension	$F2$ (in)	3.25
Fuse plate dimension	$F3$ (in)	7
Fuse plate dimension	$F4$ (in)	1.9
Fuse plate dimension	$F6$ (in)	2.75
Fuse plate dimension	$F8$ (in)	2.45
Distance from beam centerline to bolt line	$P2$ (in)	3
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	9.07E+04
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	2.98E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	3.99E+04
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	6.94E+04
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	2.38E+04
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	6.68E+04
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	1.18E+07
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	6.64E+06
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.57 **OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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

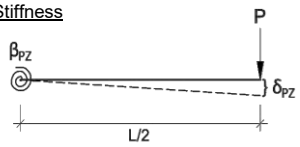
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

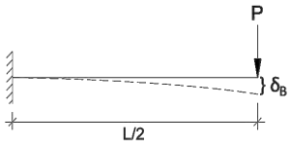


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.7
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)	2.6
Cover plate thickness	$T1$ (in)	0.875
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	1486.5
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)	2.40E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.44E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.41E+07
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)	2.21E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.92 **OK**

*This check is N/A when connection stiffness is applied in the model analysis

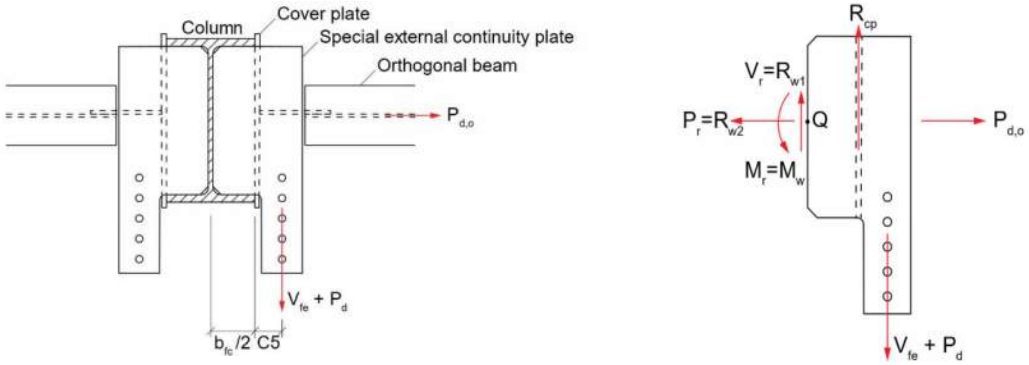
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	403.7
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	50.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	25.0
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
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
External continuity plate thickness	$T5$ (in)	1.125
External continuity plate gage dimension	$C5$ (in.)	3.00
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	110.04	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	22.25	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
		Special detailing not required	

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	1.46	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	99.2	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	21.13	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply		Special detailing not required	



Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size			
Orthogonal beam depth	$d_{b,o}$ (in)		
In plane moment frame beam 1		W24X146	
In plane moment frame beam 2		NA	
Max in-plane moment frame beam depth	d_b (in)	24.7	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0	
New continuity plate required for deeper cantilever beam		N/A	
New continuity plate width past column flange	C4 (in)	6	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A	**N/A**

This check is N/A when OOP forces do not apply

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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.)	44	
Beam depth	d_b (in.)	24.7	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	146.0	**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}	5.9	**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.)	44	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	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), Factor based on code and member type, $F(E/(R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	
	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	6.45	**OK**

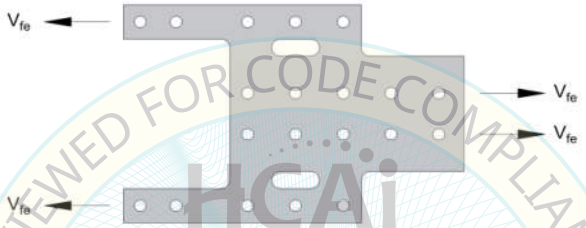
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	307	
Column design axial load	P_u (kips)	271.9	
Column area	A_{gc} (in^2)	35.9	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.7	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c*(F_{yc}-P_u/A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	3.23E+04	
Beam plastic section modulus	Z_b (in^3)	418	
Minimum 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)	2.27E+04	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.42	**OK**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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**
Fuse plate thickness	$T2$ (in)	1.25	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	1.125	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	1.125	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	24.7	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	403.7	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	403.7	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	403.7	
Beam area	A_{gb} (in^2)	43.0	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	0.0	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	403.7	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	56	
Fuse plate thickness	$T2$ (in)	1.25	
Top plate thickness	$T4$ (in)	1.125	
External continuity plate thickness	$T5$ (in)	1.125	
Beam depth	d_b (in)	24.7	
Column depth	d_c (in)	21.7	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	509.5	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	0.875	
Cover plate overhange dimension	$C3$ (in)	2.5	
Column depth	d_c (in)	21.7	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	26.7	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	700.9	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	700.9	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.58	**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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	26.58	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	697.6	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	697.6	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.73	**OK**

Step 3. (continued)

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, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	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 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
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)	62.6	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	6.4	
Number of bolts per line (AISC 358 15.2(1))	n_b	7	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Top plate thickness	$T4$ (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	3.834	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.1976	
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	3	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	403.7	
Beam flange thickness	t_{fb} (in)	1.09	
Top plate thickness	$T4$ (in)	1.125	
Governing material thickness, $\min(t_{fb}, T4)$	t_{min} (in)	1.1	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1339.1	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	988.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1065.4	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	584.5	
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	584.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	438.4	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.92	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3068	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	1.125	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	59.3	
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	7	
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)	189.8	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	161.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.37	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	403.7	
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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	4.8	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	7	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Fuse plate thickness	$T2$ (in)	1.25	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	3.450	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.0	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	4	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	3	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	403.7	
Beam flange thickness	t_{fb} (in)	1.09	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1339.1	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	988.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	77.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1065.4	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	584.5	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	584.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	438.4	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.92	**OK**

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kemalettin Cagatayoglu
DATE: 10/09/2026

Step 5b. (continued)

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, $M_u/(2(d_b+T2/2 + T4/2))$	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 on the bottom flange	$n_{b,Bot}$	7	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	189.8	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	161.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.37	**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 (in)	1.625	
Beam flange width factor, 0 *if* $b_{fb} < b_{fc}$, 1.0 *otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	2.5	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	5.607	
Distance from alignment line to column	*C1* (in)	5.625	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	0	
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	
Fuse plate thickness	$T2$ (in)	1.25	
Top plate thickness	$T4$ (in)	1.125	
External continuity plate thickness	$T5$ (in)	1.125	
Cover plate thickness	$T1$ (in)	0.875	
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2T5$ for cap plate and ortho. cant./drag condition	h_{cp} (in)	26.575	
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)	0.6875	
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	25.20	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $T5 + T1 + [C6]$	l_{We1} (in)	5.000	
Weld 1 size	D_{W1} (1/16 in)	9	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
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)	1.16	
Vertical distance from center of rotation to geometric center	e_y (in)	0.00	

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.

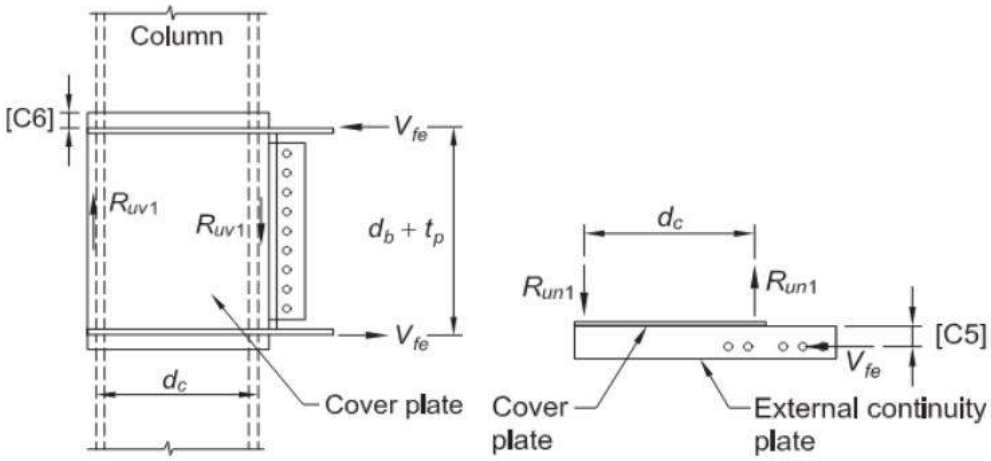
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	21382	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	15.70	11.36	51.88	178.3
W4TL	18.60	5.98	35.70	111.2
W1LB	17.23	10.96	45.79	188.8
W4BR	17.34	5.94	29.47	103.0
W1RC	9.69	6.68	1.49	64.7
W4TC	15.16	5.03	5.14	76.3
W1LC	12.01	6.99	1.20	84.0
W4BC	15.16	5.03	5.14	76.3
		11.36	51.88	

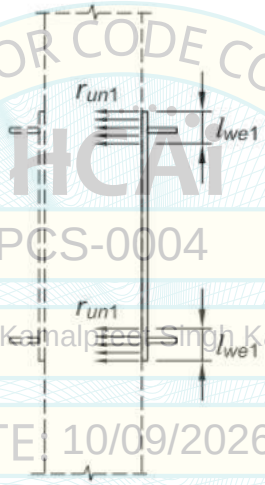
Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	11.4	
Orientation of Resultant Force	θ_r (degrees)	51.9	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.50	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	11.0	
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)	11.0	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.08	
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)	16.91	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.65	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0	
Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang dimension	*C3* (in)	2.5	
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} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	3.84	
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)	14.4	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	20.9	
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*	0.94	**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)*	403.7	
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	1211.0	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	403.7	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}\,(d_c+2[C3])T5$	ϕR_n (kips)	901.1	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}\,T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	9022.5	
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)*	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
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)	89.0	
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)	1320.8	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	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)	2093.7	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1570.2	
Weld 2 demand/capacity check for cap plate configuration, $R_u\,/\,\phi R_n$	*D/C*	0.26	**OK**

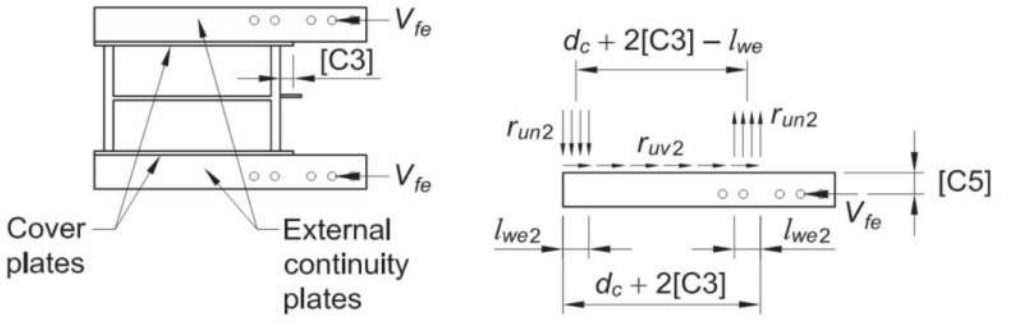


Fig. C-15.5. Weld demands for external continuity plate welds.

DESIGNED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	4	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	230.7	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate width	*C4* (in)	6.375	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Net area of external continuity plate cross section, *T5([C4]-($d_{hole,std}$ +1/16))*	A_n (in^2)	5.695	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	370.2	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	277.6	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.83	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	5	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	288.3	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	2.5	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	1.625	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate width	*C4* (in)	6.375	
Diagonal bonus (AISC 360 B4.3b), *[C9]^2 (T5)/4[C5]*	*Bonus* (in)	0.248	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, T5*([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	5.943	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	386.3	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	289.7	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	1.00	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	6	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	346.0	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	2.5	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
External continuity plate width	*C4* (in)	6.375	
Diagonal bonus (AISC 360 B4.3b), *[C7]^2 (T5)/(4[C5])*	*Bonus* (in)	2.344	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, *(T5)([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	8.039	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	522.5	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	391.9	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.88	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	5	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	288.3	
External continuity plate width	*C4* (in)	6.375	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	1.625	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	7.172	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	1.828	
Net tension area, A_{gt} - *((T5)*($d_{hole,std}$ +1/16))*	A_{nt} (in^2)	5.695	
Net shear area, A_{gv}	A_{nv} (in^2)	1.828	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	425.0	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	318.8	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.90	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	6	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	346.0	
Bolt spacing (see schedule)	s (in)	3.375	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	2.5	
External continuity plate width	*C4* (in)	6.375	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	5	
External continuity plate thickness (see step 2)	*T5* (in)	1.125	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	7.172	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	5.625	
Net tension area, $A_{gt}-((T4)(d_{hole}+1/16))$	A_{nt} (in^2)	5.695	
Net shear area, A_{gv}	A_{nv} (in^2)	5.625	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	538.9	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	404.2	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.86	**OK**

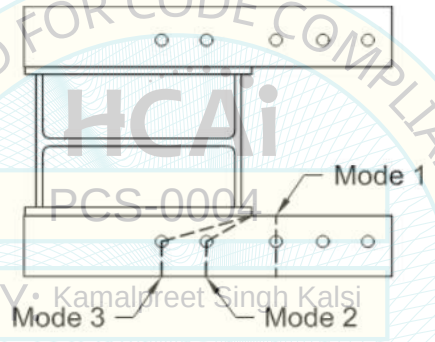


Fig. C-15.6. Tension rupture limit state for external continuity plate.

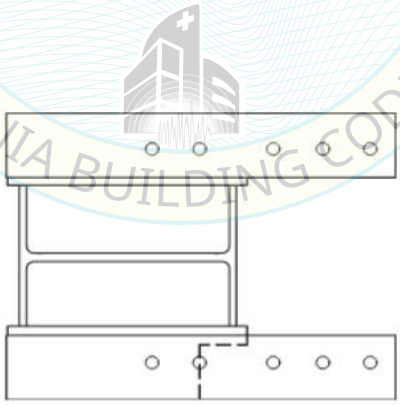


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	807.3	
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	7	
Length of connection on beam	$B4$ (in)	21.875	
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)	47.688	
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)	29.089	
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)	1530.4	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	1147.8	
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)	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 provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	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 (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	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1}\ t_{min}\ F_{u,min}\ (n_n - 1)$	R_{nt1} (kips)	477.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2}\ t_{min}\ F_{u,min}$	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), $1.5 T_b \mu D_u h_f n_s$	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)	5.625	
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)	909.8	

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	$T3$ (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 (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, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) T3$	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), $(0.75) 0.6 F_{uST} A_{nv}$	ϕR_{nv} (kips)	201.09	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75) F_{uST} A_e$	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)	909.84	
Shear tab gross area, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	45.13	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0) 0.6 F_{yST} A_{gv}$	ϕR_{nv} (kips)	285.00	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9) F_{yST} A_{gv}$	ϕR_{nn} (kips)	427.50	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9) F_{yST} Z_{tab}$	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	0.70	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	19	
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)	8.5	
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)	5.625	
Size of weld in 1/16^{ths} of an inch	D_{w3}	11	
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)	909.8	
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)	26.4	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	27.7	
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)	30.6	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.91	**OK**
This check is N/A when bridge plates do not apply			

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 (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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (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)

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	403.7	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	1.125	
Top plate dimension (see schedule)	$P8$ (in)	23.75	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	26.719	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	801.6	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	801.6	
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	7	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	14.906	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	581.3	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	436.0	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.93	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

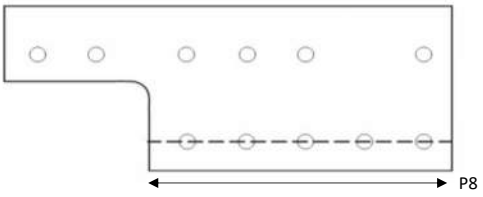


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	403.7	

Yielding check (LS 1)

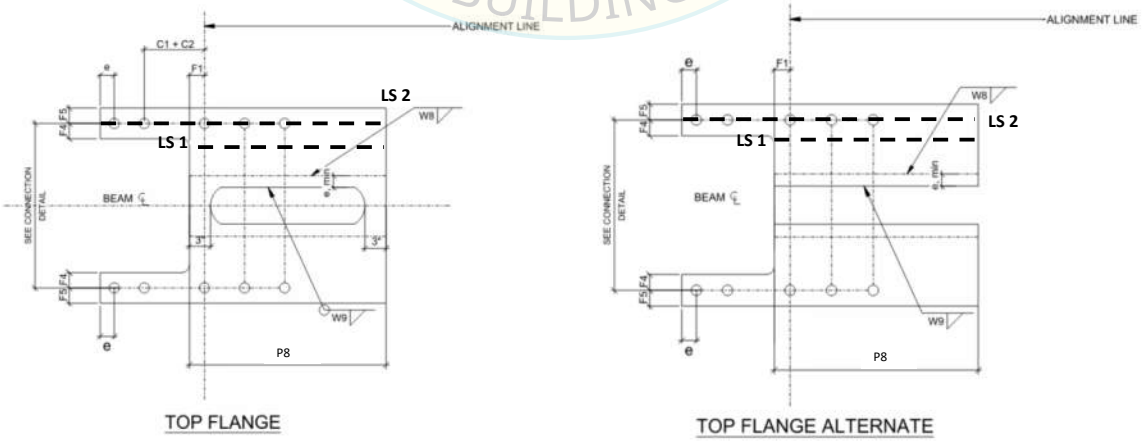
Top plate thickness (see step 2)	*T4* (in)	1.125	
Top plate dimension (see schedule)	*P8* (in)	19.5	
Fuse plate dimension	*F1 (in)*	1.75	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	21.938	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	658.1	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	658.1	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.61	**OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Number of oversized bolts in Zone M (see step 5)	n_m	3	
Number of oversized bolts on top plate (see step 5)	n_b	7	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.5	
Bolt spacing (see schedule)	*s* (in)	3.375	
Fuse plate dimension	*F1 (in)*	1.75	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	30.75	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	34.594	
Number of oversized bolts on top plate (see step 5)	n_b	7	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	23.273	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	907.7	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	680.7	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.59	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	403.7	
Beam flange width	b_{fb} (in)	12.9	
Weld 8 size	D_{W8} (1/16 in)	10	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.63	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	18.25	
Weld 9 size	D_{W9} (1/16 in)	10	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	13.50	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	12.7	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	13.9	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.91	**OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Total number of bolts (see step 5)	n_b	7	
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)	173.0	

Yielding check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	2.875	
Top plate thickness (see step 2s)	*T4* (in)	1.125	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	5.344	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	267.2	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	240.5	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.72	**OK**

Rupture check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	2.875	
Top plate thickness (see step 2)	*T4* (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)(T4)*	A_e (in^2)	3.656	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	237.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	178.2	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.97	**OK**

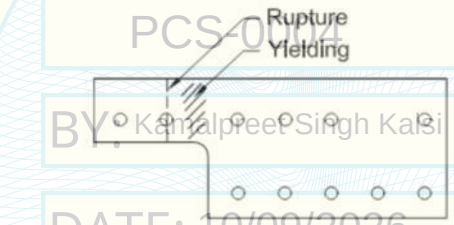


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0024
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

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)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Top plate thickness (see step 2)	*T4* (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)	23.75	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	13.290	
Dimension, *([P8] - m) / 2*	*e* (in)	5.230	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.0	
Provided P2 dimension	*P2* (in)	3	**OK**

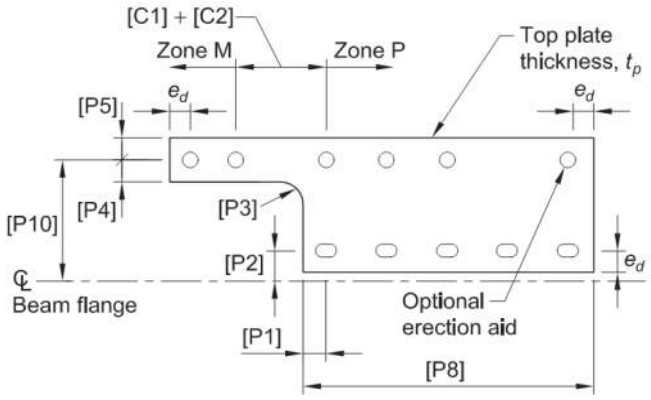


Fig. 15.6. Top plate detail.

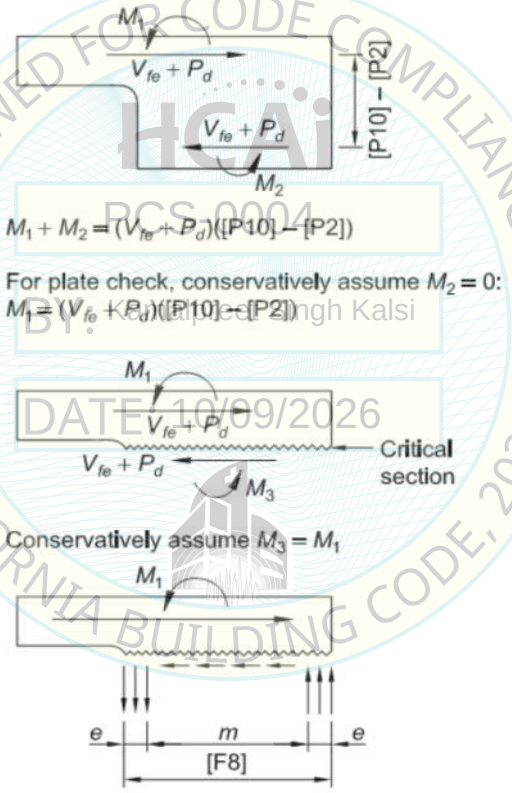


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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**
Fuse plate thickness (see step 2)	$T2$ (in)	1.25	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	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)	403.7	
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)	$n_{b,\ Bot}$	7	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	346.0	
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)	$T2$ (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, $2([F7]+[F8])T2$	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	0.56	**OK**

Step 18. (continued)

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)	$T2$ (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, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	9.875	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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	0.60	**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

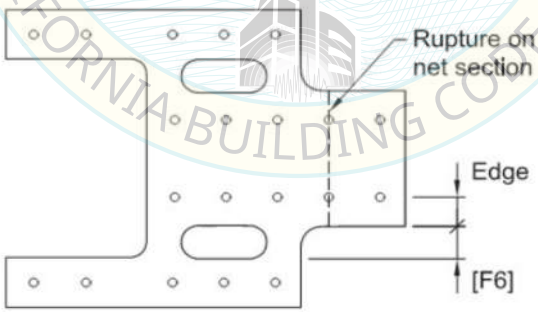


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
UPDATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	2.75	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	20900	
Fuse plate thickness (see step 2)	*T2* (in)	1.25	
Top plate thickness (see step 2)	*T4* (in)	1.125	
Beam depth	d_b (in)	24.7	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	161.7	
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_n	4	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	108.5	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	113.9	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	45.6	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	*F2* $_{max}$ (in)	3.008	
Provided yielding region depth	*F2* (in)	3.000	**OK**
Fuse yield force (one side), *2(0.6F* $_{yFuse}$ *)[F2]T2*	V_y (kips)	225.0	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	403.7	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.8	**OK**

Fuse plate yielding region geometry			
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	*(F6/F2)* $_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.91	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7
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)	$n_{b,\ Bot}$	7
Number of bolts in the M region	$n_{m,\ Bot}$	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)	346.0

Yielding check

Fuse plate dimension	*F4* (in)	1.875	
Fuse plate dimension	*F5* (in)	2	
Fuse plate thickness (see step 2)	*T2* (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])(T2)*	A_g (in^2)	9.6875	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	R_n (kips)	484.38	
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	435.94	
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.79	**OK**

Rupture check

Fuse plate dimension	*F4* (in)	1.875	
Fuse plate dimension	*F5* (in)	2	
Fuse plate thickness (see step 2)	*T2* (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, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	5.9375	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	R_n (kips)	463.1	
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	347.3	
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

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
RETAINED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

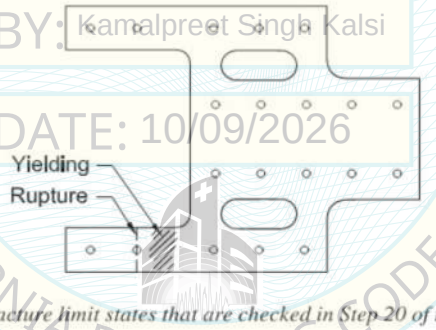
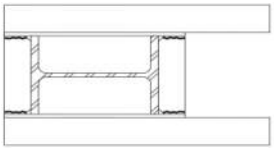


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

MISCELLANEOUS CALCULATIONS

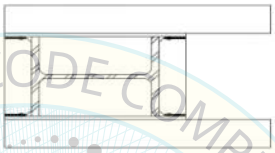
Bridge plate calculations

Bridge plate tension failure check



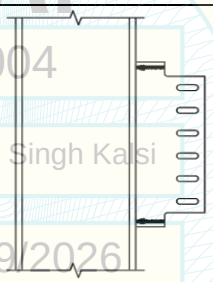
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	403.7	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	0	
External continuity plate dimension	*C5* (in)	3	
Column depth	d_c (in)	21.7	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	2.5	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}$ *T4([C3]-1")*	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**

Bridge plate to cover plate weld failure check



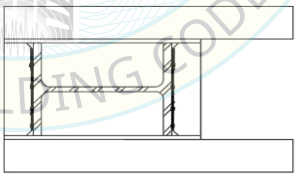
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16$ *([C3]-1")(2 sides)*	ϕR_n (kips)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

Shear tab to bridge plate weld failure check



Minimum 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)	5.625	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2.5	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

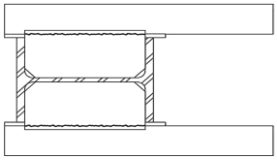
Bridge plate to column flange weld failure check



Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	5.625	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2.5	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

*These checks are N/A when bridge plates do not apply

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), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	296.7	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**

*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	4580
Column flange width	b_{fc} (in)	12.4
Beam depth	d_b (in)	24.7
Bay width	B (in)	360.0
Number of bolts in zone M	M	3
Number of bolts in zone P	P	4
Bolt spacing	s (in)	3.375
Cover plate thickness	$T1$ (in)	0.875
Fuse plate thickness	T2 (in)	1.25
Top plate thickness	$T4$ (in)	1.125
External continuity plate dimension	$C5$ (in)	3
Fuse plate dimension	$F2$ (in)	3
Fuse plate dimension	$F3$ (in)	7.5
Fuse plate dimension	$F4$ (in)	1.9
Fuse plate dimension	$F6$ (in)	2.75
Fuse plate dimension	$F8$ (in)	2.45
Distance from beam centerline to bolt line	$P2$ (in)	3
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	1.01E+05
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	3.05E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	3.48E+04
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	7.71E+04
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	2.37E+04
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	6.01E+04
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	1.14E+07
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	6.64E+06
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.58 **OK**

DETAILED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

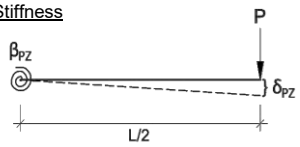
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

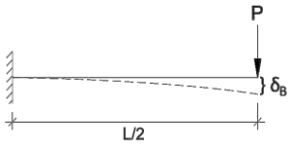


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.7
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)	2.5
Cover plate thickness	$T1$ (in)	0.875
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	1475.7
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)	2.38E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.44E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.40E+07
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)	2.21E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.92 **OK**

*This check is N/A when connection stiffness is applied in the model analysis

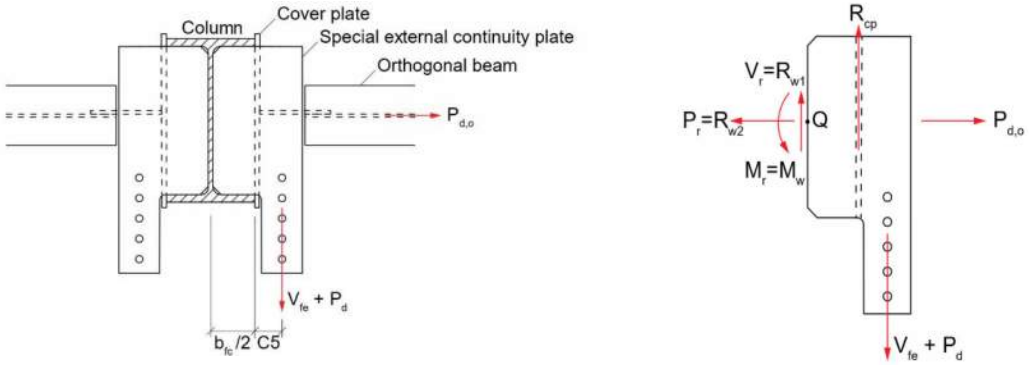
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	403.7
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	0.0
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	50.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	25.0
Moment from orthgonal connection (seismic load combination) (from EOR)	$M_{u,o}$ (k-in)	0
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)	0.0
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	25.0
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
External continuity plate thickness	$T5$ (in)	1.125
External continuity plate gage dimension	$C5$ (in.)	3.00
Plate-to column flange force, $[(\Sigma V_{fe} + P_d) / (1+0.5t_{wc} / T5)]$	R_{cp} (kips)	318.7
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	85.0
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	25.0

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	1211.0	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	25.0	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	403.7	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	110.04	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	22.25	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	4951.7	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	1001.4	
Shear capacity at section, $0.75(0.6F_{uECP})A$	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	0.39	**OK**
	Special detailing not required		

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5]) - R_{cp}(b_{fc}/2)$	M_r (k-in)	1737.9	
Normal demand at section, $=P_{d,o}$	P_r (kips)	25.0	
Shear demand at section, $=R_{w1}$	V_r (kips)	85.0	
Corner clip dimension	K_c (in)	1.46	
Plate plastic section modulus, $(d_c - 2K_c)^2\ T5/4$	Z (in^3)	99.2	
Plate section area, $(d_c - 2K_c)T5$	A (in^2)	21.13	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	4463.7	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	950.7	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	618.0	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.39	**OK**
These checks are N/A when OOP forces do not apply	Special detailing not required		



Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size		W24X55	
Orthogonal beam depth	$d_{b,o}$ (in)	23.6	
In plane moment frame beam 1		W24X146	
In plane moment frame beam 2		NA	
Max in-plane moment frame beam depth	d_b (in)	24.7	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	25.0	
New continuity plate required for deeper cantilever beam		Not Req'd	
New continuity plate width past column flange	C4 (in)	6.375	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	0.6875	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	131.08	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	5.70	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	0.044	**NA**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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.)	44	
Beam depth	d_b (in.)	24.5	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	131.0	**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.)	44	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	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),	$\lambda_{hd,flange}$	7.35	
Factor based on code and member type, $F(E / (R_{yc} F_{yc}))^{0.5}$	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	6.45	**OK**

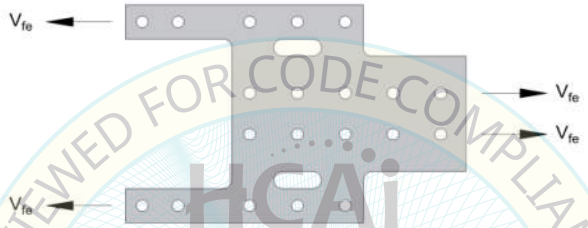
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	307	
Column design axial load	P_u (kips)	53.5	
Column area	A_{gc} (in^2)	35.9	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.5	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c (F_{yc} - P_u/A_{gc})(H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	3.68E+04	
Beam plastic section modulus	Z_b (in^3)	370	
Minimum 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)	1.98E+04	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.86	**OK**

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

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

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**
Fuse plate thickness	$T2$ (in)	1.125	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	1.125	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	1	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	24.5	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	361.0	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	$P_{d,total}$ (kips)	0.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	0.0	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	361.0	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	35	
Fuse plate thickness	$T2$ (in)	1.125	
Top plate thickness	$T4$ (in)	1.125	
External continuity plate thickness	$T5$ (in)	1	
Beam depth	d_b (in)	24.5	
Column depth	d_c (in)	21.7	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	443.8	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	0.875	
Cover plate overhange dimension	$C3$ (in)	2	
Column depth	d_c (in)	21.7	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	25.7	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	674.6	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	674.6	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.54	**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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	22.50	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	590.6	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	590.6	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.75	**OK**

Step 3. (continued)

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, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	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 5a. Determine the number of bolts for each bolt line on the 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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
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)	62.6	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	5.8	
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 (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Top plate thickness	$T4$ (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	3.428	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.9574	
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)	361.0	
Beam flange thickness	t_{fb} (in)	0.96	
Top plate thickness	$T4$ (in)	1.125	
Governing material thickness, $\min(t_{fb}, T4)$	t_{min} (in)	1.0	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1010.9	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	725.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	67.9	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	793.3	
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	0.96	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1451	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	1.125	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	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	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*	0.20	**OK**

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

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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	4.3	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	6	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (see schedule)	s (in)	3.375	
Fuse plate thickness	$T2$ (in)	1.125	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	3.428	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.0	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	3	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	3	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	361.0	
Beam flange thickness	t_{fb} (in)	0.96	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	1010.9	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	725.4	
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, $e - d_O / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	67.9	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	793.3	
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*	0.96	**OK**

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kemalettin Cagatayalsi
DATE: 10/09/2026

Step 5b. (continued)

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, $M_u/(2(d_b+T2/2 + T4/2))$	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 on the bottom flange	$n_{b,Bot}$	6	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	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*	0.20	**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 (in)	1.625	
Beam flange width factor, 0 *if* $b_{fb} < b_{fc}$, 1.0 *otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	2	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	5.095	
Distance from alignment line to column	*C1* (in)	5.125	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

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)	$P_{d,total}$ (kips)	0.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	0	
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	
Fuse plate thickness	*T2* (in)	1.125	
Top plate thickness	*T4* (in)	1.125	
External continuity plate thickness	*T5* (in)	1	
Cover plate thickness	*T1* (in)	0.875	
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3	
Cover plate height, d_b + 2[C6], d_b + [C6] - T5 or just d_b - 2T5 for cap plate and ortho. cant./drag condition	h_{cp} (in)	22.5	
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)*	0.6875	
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), *T5 + T1 + [C6]*	l_{We1} (in)	4.875	
Weld 1 size	D_{W1} (1/16 in)	9	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
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)	0.80	
Vertical distance from center of rotation to geometric center	e_y (in)	0.00	

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.

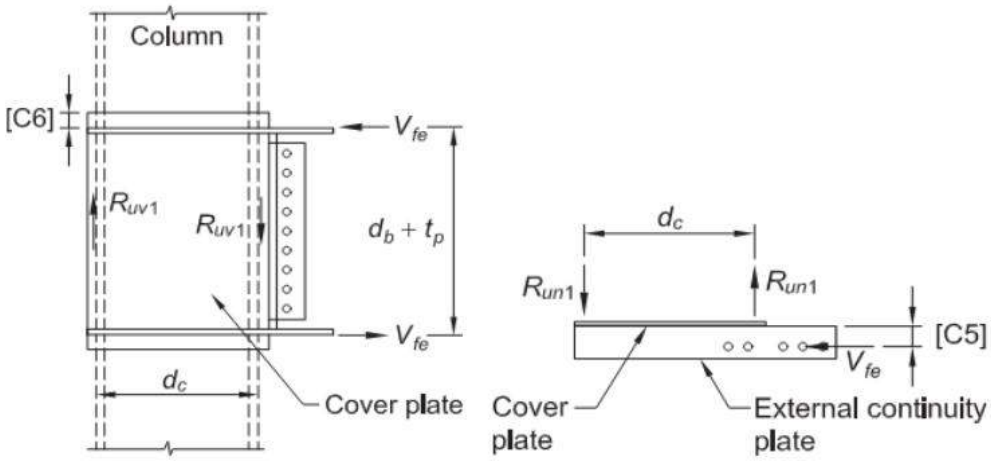
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	18346	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	14.34	9.07	45.52	130.0
W4TL	18.31	5.94	34.98	108.8
W1LB	15.51	8.85	41.27	137.2
W4BR	17.44	5.93	30.65	103.4
W1RC	10.05	5.58	1.19	56.0
W4TC	15.03	4.97	3.82	74.7
W1LC	11.66	5.76	1.03	67.2
W4BC	15.03	4.97	3.82	74.7
		9.07	45.52	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	9.1	
Orientation of Resultant Force	θ_r (degrees)	45.5	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.42	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	10.9	
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)	10.9	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	0.98	
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)	16.29	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	*D/C*	0.67	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0	
Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang dimension	*C3* (in)	2	
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} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	3.34	
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)	14.0	
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)	15.2	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	20.7	
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*	0.93	**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)	1082.9	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	361.0	
External continuity plate thickness (see step 2)	*T5* (in)	1.000	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	771.0	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	7430.5	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.61	**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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
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)	87.0	
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)	1291.1	
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)	2068.4	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1551.3	
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	*D/C*	0.23	**OK**

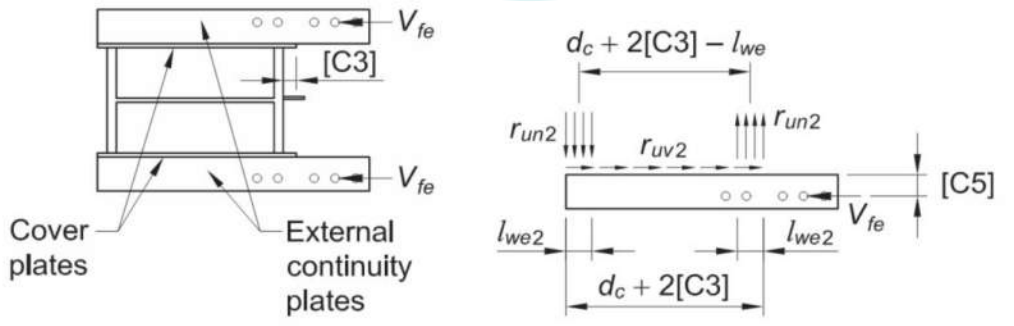


Fig. C-15.5. Weld demands for external continuity plate welds.

DESIGNED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	3	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	180.5	
External continuity plate thickness (see step 2)	*T5* (in)	1	
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, *T5([C4]-($d_{hole,std}$ +1/16))*	A_n (in^2)	4.938	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	320.9	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	240.7	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.75	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	4	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	240.7	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	1.125	
External continuity plate thickness (see step 2)	*T5* (in)	1	
External continuity plate width	*C4* (in)	6.25	
Diagonal bonus (AISC 360 B4.3b), *[C9]^2 (T5)/4[C5]*	*Bonus* (in)	0.105	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, T5*([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	5.043	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	327.8	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	245.8	
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, $n_{rup1} + 2$	n_{rup}	5	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	300.8	
Bolt spacing (see schedule)	*s* (in)	3.375	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate dimension	*C5* (in)	3	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	4.5	
External continuity plate thickness (see step 2)	*T5* (in)	1	
External continuity plate width	*C4* (in)	6.25	
Diagonal bonus (AISC 360 B4.3b), *[C7]^2 (T5)/(4[C5])*	*Bonus* (in)	1.688	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Area of external continuity plate cross section with bonus, *(T5)([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	6.625	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	430.6	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	323.0	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.93	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	4	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	240.7	
External continuity plate width	*C4* (in)	6.25	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	1.125	
External continuity plate thickness (see step 2)	*T5* (in)	1	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	6.250	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	1.125	
Net tension area, A_{gt} - *((T5)*($d_{hole,std}$ +1/16))*	A_{nt} (in^2)	4.938	
Net shear area, A_{gv}	A_{nv} (in^2)	1.125	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	354.7	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	266.0	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.90	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	5	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	300.8	
Bolt spacing (see schedule)	s (in)	3.375	
External continuity plate dimension	*C2 (in)*	-0.875	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate width	*C4* (in)	6.25	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	4.5	
External continuity plate thickness (see step 2)	*T5* (in)	1	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.25	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	6.250	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	4.500	
Net tension area, $A_{gt}-((T4)(d_{hole}+1/16))$	A_{nt} (in^2)	4.938	
Net shear area, A_{gv}	A_{nv} (in^2)	4.500	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	455.9	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	342.0	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.88	**OK**

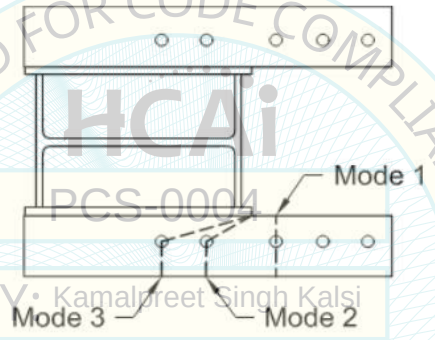


Fig. C-15.6. Tension rupture limit state for external continuity plate.

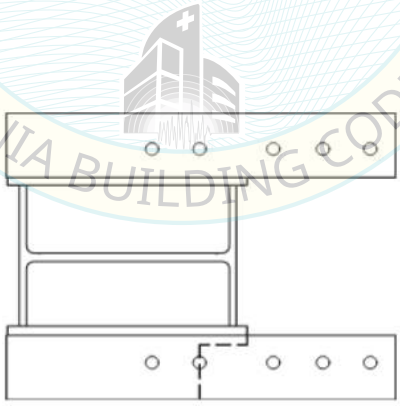


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	722.0	
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	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)	7.104	
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	35.520	
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)	21.660	
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)	1224.6	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	918.5	
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.79	**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 provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	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 (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	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1}\ t_{min}\ F_{u,min}\ (n_n - 1)$	R_{nt1} (kips)	477.8	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2}\ t_{min}\ F_{u,min}$	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), $1.5 T_b \mu D_u h_f n_s$	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)	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)	596.4	

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	$T3$ (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 (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, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) T3$	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), $(0.75) 0.6 F_{uST} A_{nv}$	ϕR_{nv} (kips)	201.09	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75) F_{uST} A_e$	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)	596.40	
Shear tab gross area, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	45.13	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0) 0.6 F_{yST} A_{gv}$	ϕR_{nv} (kips)	285.00	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9) F_{yST} A_{gv}$	ϕR_{nn} (kips)	427.50	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9) F_{yST} Z_{tab}$	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	0.47	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	19	
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)	6.1	
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)	5.125	
Size of weld in 1/16^{ths} of an inch	D_{W3}	9	
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)	596.4	
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)	21.2	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	22.0	
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)	25.1	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.88	**OK**
This check is N/A when bridge plates do not apply			

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 (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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (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)

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	361.0	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	1.125	
Top plate dimension (see schedule)	$P8$ (in)	20.375	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	22.922	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	687.7	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	687.7	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.52	**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, $A_g - (T4 l_s n_b)$	A_n (in^2)	12.797	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	499.1	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	374.3	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.96	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

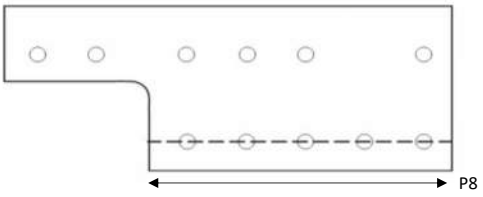


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile 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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	361.0	

Yielding check (LS 1)

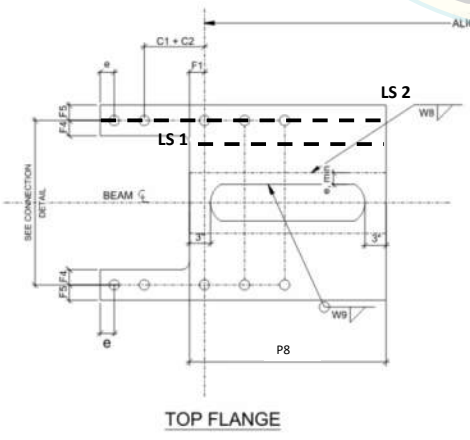
Top plate thickness (see step 2)	*T4* (in)	1.125	
Top plate dimension (see schedule)	*P8* (in)	16.125	
Fuse plate dimension	*F1 (in)*	1.75	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	18.141	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	544.2	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	544.2	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.66	**OK**

Rupture check (LS 2)

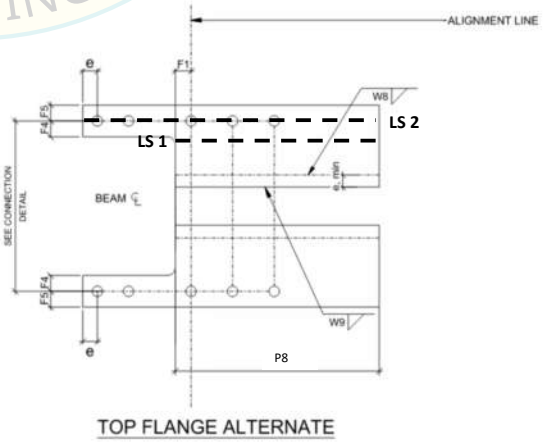
Bolt diameter (see step 4)	d_{bolt} (in)	1.125	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.4375	
Number of oversized bolts in Zone M (see step 5)	n_m	3	
Number of oversized bolts on top plate (see step 5)	n_b	6	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.5	
Bolt spacing (see schedule)	*s* (in)	3.375	
Fuse plate dimension	*F1 (in)*	1.75	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	26.875	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	30.234	
Number of oversized bolts on top plate (see step 5)	n_b	6	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	20.531	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	800.7	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	600.5	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.60	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	361.0	
Beam flange width	b_{fb} (in)	12.9	
Weld 8 size	D_{W8} (1/16 in)	11	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.69	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	14.75	
Weld 9 size	D_{W9} (1/16 in)	11	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	10.13	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	14.5	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	15.3	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.95	**OK**



TOP FLANGE



TOP FLANGE ALTERNATE

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
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)	180.5	

Yielding check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	3	
Top plate thickness (see step 2s)	*T4* (in)	1.125	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	5.484	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	274.2	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	246.8	
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)	3	
Top plate thickness (see step 2)	*T4* (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)(T4)*	A_e (in^2)	3.797	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	246.8	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	185.1	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.98	**OK**

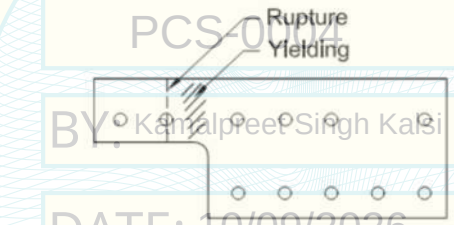


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0024
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Top plate thickness (see step 2)	*T4* (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)	20.38	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	11.884	
Dimension, *([P8] - m) / 2*	*e* (in)	4.246	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.5	
Provided P2 dimension	*P2* (in)	2.75	**OK**

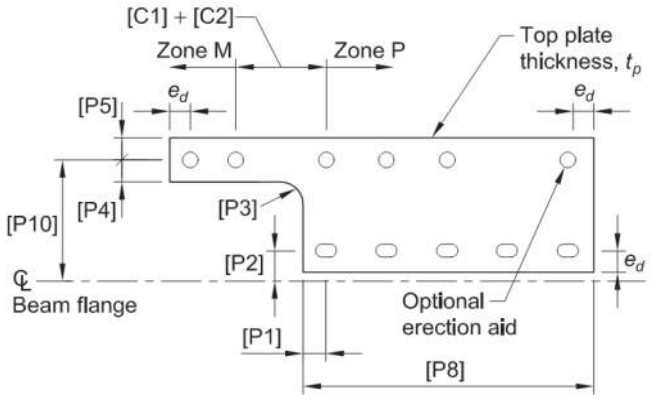


Fig. 15.6. Top plate detail.

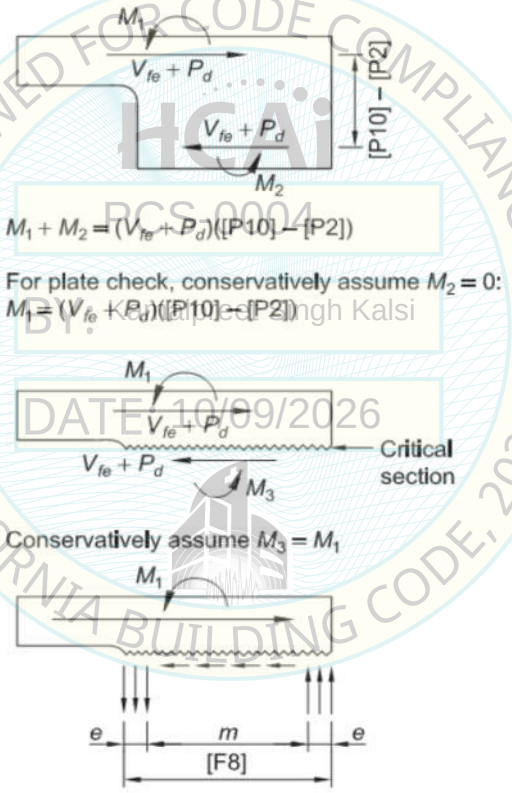


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS 0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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**
Fuse plate thickness (see step 2)	$T2$ (in)	1.125	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	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)	$n_{b,\ Bot}$	6	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	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)	$T2$ (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, $2([F7]+[F8])T2$	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**

Step 18. (continued)

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)	$T2$ (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, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	9.450	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse}R_{tp}A_e$ [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

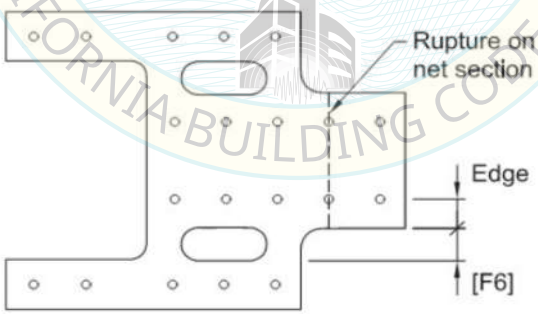


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaljeet Singh Kalsi
UPDATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	2.5	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	18500	
Fuse plate thickness (see step 2)	*T2* (in)	1.125	
Top plate thickness (see step 2)	*T4* (in)	1.125	
Beam depth	d_b (in)	24.5	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	116.4	
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_n	4	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	108.5	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	113.9	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	45.6	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	2.977	
Provided yielding region depth	*F2* (in)	2.875	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	194.1	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	361.0	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.9	**OK**

Fuse plate yielding region geometry			
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.84	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

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)	$n_{b,\ Bot}$	6
Number of bolts in the M region	$n_{m,\ Bot}$	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)	361.0

Yielding check

Fuse plate dimension	*F4* (in)	1.875	
Fuse plate dimension	*F5* (in)	2.375	
Fuse plate thickness (see step 2)	*T2* (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])(T2)*	A_g (in^2)	9.5625	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	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)	*T2* (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, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	6.1875	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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

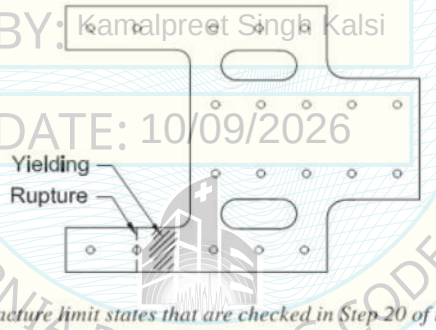


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

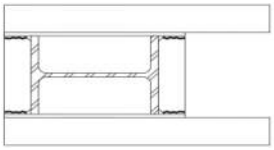
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

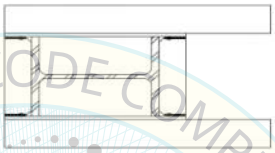
Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	361.0	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	0	
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)	N/A	
Cover plate overhang	*C3* (in)	2	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}\ T4([C3]-1")$	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**



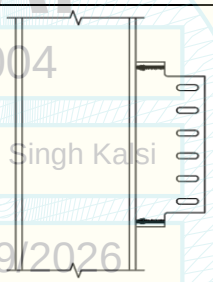
Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



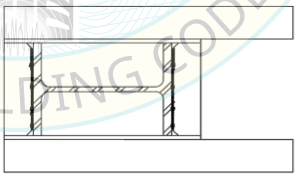
Shear tab to bridge plate weld failure check

Minimum 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)	5.125	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



Bridge plate to column flange weld failure check

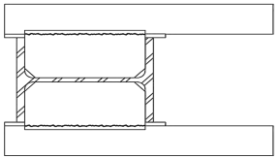
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	5.125	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



*These checks are N/A when bridge plates do not apply

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), $1.0(0.6)F_{ySP}\ t_{sp}\ (d_c - 2t_{fc})$	ϕR_n (kips)	296.7	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**



*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	4020
Column flange width	b_{fc} (in)	12.4
Beam depth	d_b (in)	24.5
Bay width	B (in)	360.0
Number of bolts in zone M	M	3
Number of bolts in zone P	P	3
Bolt spacing	s (in)	3.375
Cover plate thickness	$T1$ (in)	0.875
Fuse plate thickness	T2 (in)	1.125
Top plate thickness	$T4$ (in)	1.125
External continuity plate dimension	$C5$ (in)	3
Fuse plate dimension	$F2$ (in)	2.875
Fuse plate dimension	$F3$ (in)	4.375
Fuse plate dimension	$F4$ (in)	1.9
Fuse plate dimension	$F6$ (in)	2.5
Fuse plate dimension	$F8$ (in)	2.95
Distance from beam centerline to bolt line	$P2$ (in)	2.75
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	6.80E+04
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	2.90E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	3.67E+04
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	4.32E+04
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	2.01E+04
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	4.64E+04
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	9.21E+06
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	5.83E+06
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.63 **OK**

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CERTIFIED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

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

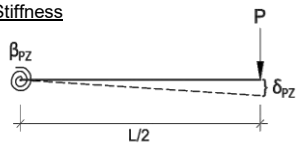
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

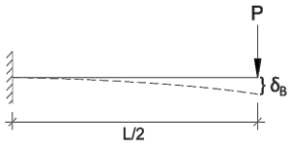


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	24.5
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)	2.0
Cover plate thickness	$T1$ (in)	0.875
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	1420.9
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)	2.29E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.44E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.31E+07
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)	1.94E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.84 **OK**

*This check is N/A when connection stiffness is applied in the model analysis

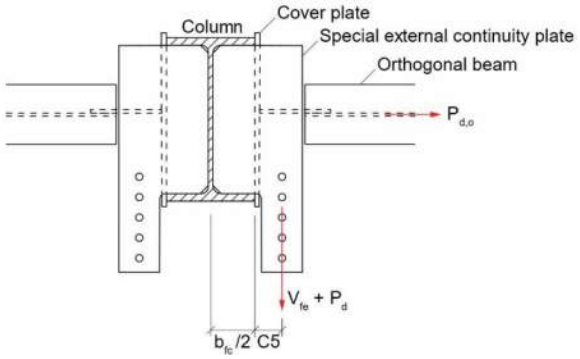
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

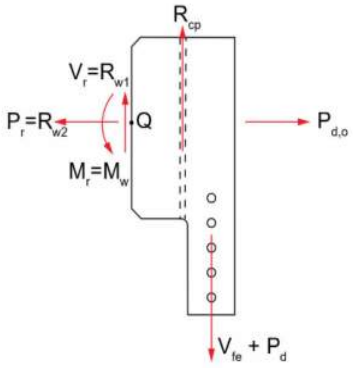
Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

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)	$P_{d,total}$ (kips)	0.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	0.0
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
Moment from orthgonal connection (seismic load combination) (from EOR)	$M_{u,o}$ (k-in)	4690
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)	203.1
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	203.1
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
External continuity plate thickness	$T5$ (in)	1.000
External continuity plate gage dimension	$C5$ (in.)	3.00
Plate-to column flange force, $[(\Sigma V_{fe} + P_d) / (1+0.5t_{wc} / T5)]$	R_{cp} (kips)	277.7
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	83.3
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	203.1

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	1082.9	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	203.1	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	361.0	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	97.81	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	19.78	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	4401.5	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	890.1	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	578.6	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.45	**OK**
	Special detailing not required		

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5]) - R_{cp}(b_{fc}/2)$	M_r (k-in)	1599.4	
Normal demand at section, $=P_{d,o}$	P_r (kips)	203.1	
Shear demand at section, $=R_{w1}$	V_r (kips)	83.3	
Corner clip dimension	K_c (in)	1.46	
Plate plastic section modulus, $(d_c - 2K_c)^2\ T5/4$	Z (in^3)	88.2	
Plate section area, $(d_c - 2K_c)T5$	A (in^2)	18.78	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	3967.7	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	845.1	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	549.3	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	0.46	**OK**
These checks are N/A when OOP forces do not apply	Special detailing not required		





Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size		W24X55	
Orthogonal beam depth	$d_{b,o}$ (in)	23.6	
In plane moment frame beam 1		W24X131	
In plane moment frame beam 2		NA	
Max in-plane moment frame beam depth	d_b (in)	24.5	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	203.1	
New continuity plate required for deeper cantilever beam		Not Req'd	
New continuity plate width past column flange	C4 (in)	6.25	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	0.6875	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	136.37	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	47.26	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	0.347	**NA**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

Connection ID DF153

W40X324 Column and W40X431 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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Beam limitation checks

Beam size		W40X431	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	44	
Beam depth	d_b (in.)	41.3	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	431.0	**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}	25.5	**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}	3.4	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks

Column size		W40X324	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	44	
Column depth	d_c (in.)	40.2	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	324	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	1.81	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	270.9	
Gross area of the column section	A_g (in^2)	95.3	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	55.5	
Column web width-to-thickness ratio	λ_{web}	34.2	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1), Factor based on code and member type, $F(E / (R_{yc} F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	
	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	4.4	**OK**

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

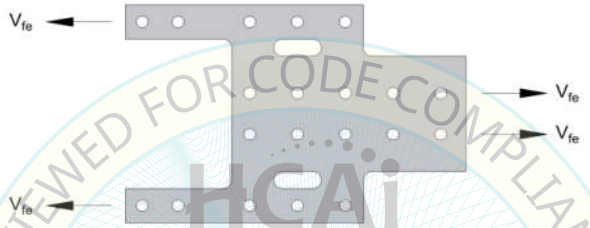
Column plastic section modulus	Z_c (in^3)	1460	
Column design axial load	P_u (kips)	270.9	
Column area	A_{gc} (in^2)	95.3	
Story Height	H (in)	216.0	
Column depth	d_c (in)	40.2	
Beam depth	d_b (in)	41.3	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c (F_{yc} - P_u / A_{gc}) (H/2)/(H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	2.00E+05	
Beam plastic section modulus	Z_b (in^3)	1960	
Minimum bay width	B (in)	360.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	98000	
Clear distance between column faces, $B - d_c$	L_h (in)	319.8	
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)	43.8	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	656.7	
Increase in moment from beam shear, $V_b (d_c/2)$	M_{uv} (k-in)	13200	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	1.11E+05	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.80	**OK**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)	1960	
Beam plastic bending moment , $F_{yb}Z_b$	M_p (k-in)	98000	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	98000	**OK**
Fuse plate thickness	$T2$ (in)	2.25	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	2.5	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	2.5	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	41.3	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	1121.9	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	1121.9	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1121.9	
Beam area	A_{gb} (in^2)	127.0	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	0.0	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	1121.9	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	91	
Fuse plate thickness	$T2$ (in)	2.25	
Top plate thickness	$T4$ (in)	2.5	
External continuity plate thickness	$T5$ (in)	2.5	
Beam depth	d_b (in)	41.3	
Column depth	d_c (in)	40.2	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	1264.3	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	1.75	
Cover plate overhange dimension	$C3$ (in)	2.75	
Column depth	d_c (in)	40.2	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	45.7	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	2399.3	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	2399.3	
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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	47.30	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	2483.3	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	2483.3	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.51	**OK**

Step 3. (continued)

Cover plate thickness check (AISC 341 E3.6e.2)			
Beam depth	d_b (in)	41.3	
Beam flange thickness	t_{fb} (in)	2.36	
Column flange thickness	t_{fc} (in)	1.81	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	36.58	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	36.58	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.813	
Thickness check, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	2.15	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.25	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Beam plastic section modulus	Z_b (in^3)	1960	
Beam flange thickness	t_{fb} (in)	2.36	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	3.540	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	275.7	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	1684.3	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	107800	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	120428	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.90	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	77.3	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	14.5	
Number of bolts per line (AISC 358 15.2(1))	n_b	15	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	4	
Top plate thickness	$T4$ (in)	2.5	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	4.795	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.2723	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	8	**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	7	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	1121.9	
Beam flange thickness	t_{fb} (in)	2.36	
Top plate thickness	$T4$ (in)	2.5	
Governing material thickness, min$(t_{fb}, T4)$	t_{min} (in)	2.4	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	6903.0	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.4	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	6281.7	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	178.3	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	6460.1	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1546.3	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1546.3	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	1159.7	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.97	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	3068	
Fuse plate thickness	$T2$ (in)	2	
Top plate thickness	$T4$ (in)	2.5	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	35.1	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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	15	
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)	518.7	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	440.9	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.08	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1121.9	
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, $P_{d,total}/2$	P_d (kips)	0.0	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	103.1	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	10.9	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	15	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	4	
Fuse plate thickness	$T2$ (in)	2.25	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	5.327	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.5	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	8	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	7	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	1121.9	
Beam flange thickness	t_{fb} (in)	2.36	
Fuse plate thickness	$T2$ (in)	2.25	
Governing material thickness, min(t_{fb}, $T2$)	t_{min} (in)	2.3	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	6903.0	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.4	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	6281.7	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	170.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	6451.7	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1546.3	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1546.3	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	1159.7	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.97	**OK**

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kemalettin Cagatayoglu
DATE: 10/09/2026

Step 5b. (continued)

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, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	35.0	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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 on the bottom flange	$n_{b,Bot}$	15	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	518.7	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	440.9	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.08	**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)	41.3	
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$, *1.0 otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	2.75	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	6.978	
Distance from alignment line to column	*C1* (in)	10.125	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	0	
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	91	
Column depth	d_c (in)	40.2	
Column flange thickness	t_{fc} (in)	1.81	
Beam depth	d_b (in)	41.3	
Fuse plate thickness	*T2* (in)	2.25	
Top plate thickness	*T4* (in)	2.5	
External continuity plate thickness	*T5* (in)	2.5	
Cover plate thickness	*T1* (in)	1.75	
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3	
Cover plate height, d_b + 2[C6], d_b + [C6] - T5 or just d_b - 2T5 for cap plate and ortho. cant./drag condition	h_{cp} (in)	47.3	
Gage dimension from the cover plate to the line of bolts on the external continuity plates	*C5* (in)	3.5	
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)	45.68	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), *T5 + T1 + [C6]*	l_{We1} (in)	7.250	
Weld 1 size	D_{W1} (1/16 in)	11	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	11	
Weld 4 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
Weld 4 length (0 indicates shear plates are not used), d_c - 2t_{fc}	l_{W4} (in)	36.58	
Horizontal distance from center of rotation to geometric center	e_x (in)	1.18	
Vertical distance from center of rotation to geometric center	e_y (in)	0.00	

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.

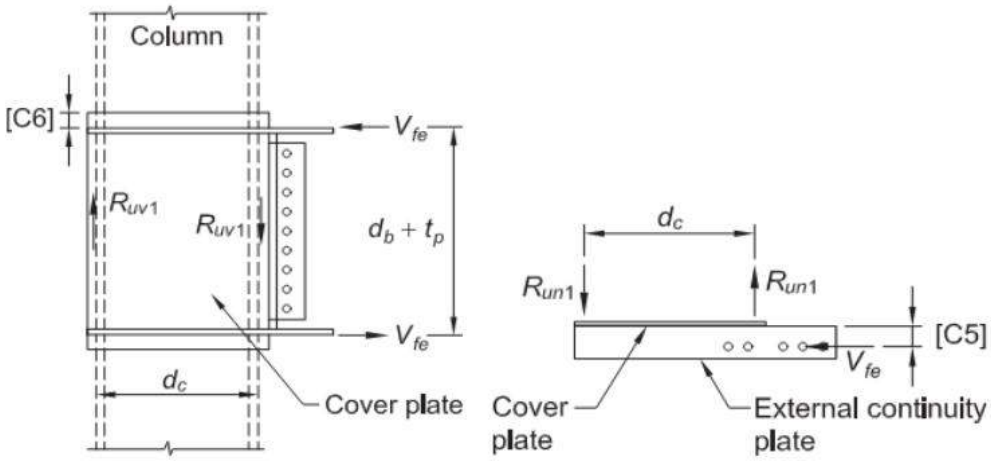
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	93101	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	29.30	24.37	49.79	714.2
W4TL	30.21	18.46	39.23	557.7
W1LB	30.88	23.74	46.44	733.1
W4BR	28.77	18.19	35.58	523.4
W1RC	18.92	15.75	1.38	298.0
W4TC	23.45	13.83	3.79	324.3
W1LC	21.29	16.13	1.23	343.4
W4BC	23.45	13.83	3.79	324.3
		24.37	49.79	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	24.4	
Orientation of Resultant Force	θ_r (degrees)	49.8	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.91	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	14.0	
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)	14.0	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.33	
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)	20.01	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	*D/C*	0.70	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0	
Column depth	d_c (in)	40.2	
Column flange thickness	t_{fc} (in)	1.81	
Cover plate overhang dimension	*C3* (in)	2.75	
External continuity plate dimension	*C5* (in)	3.5	
Weld 2 size	D_{W2} (1/16 in)	14	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.88	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), t_{fc} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	5.44	
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)	24.5	
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)	18.8	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	30.9	
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)	39.0	
External continuity plate-to-cover plate weld tearing check, $r_u\,/\,\phi r_n$	*D/C*	0.79	**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)*	1121.9	
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	3926.7	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	1121.9	
External continuity plate thickness (see step 2)	*T5* (in)	2.500	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}\,(d_c+2[C3])T5$	ϕR_n (kips)	3427.5	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}\,T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	58738.8	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.39	**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)*	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Column fillet dimension	*k* (in)	3.0	
Column fillet dimension	*k1* (in)	1.7	
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	156.3	
Column flange width	b_{fc} (in)	15.9	
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	56.85	
Weld 2 size	D_{W2} (1/16 in)	14	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	4062.1	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	1477.1	
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)	5668.4	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	4251.3	
Weld 2 demand/capacity check for cap plate configuration, $R_u\,/\,\phi R_n$	*D/C*	0.26	**OK**

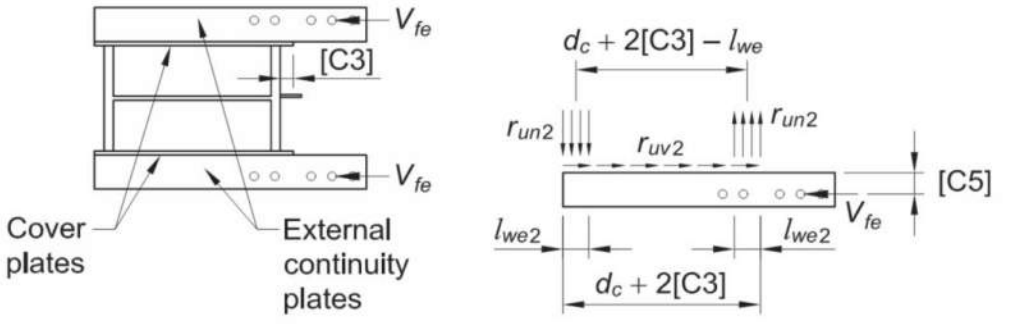


Fig. C-15.5. Weld demands for external continuity plate welds.

DESIGNED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	9	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	673.2	
External continuity plate thickness (see step 2)	*T5* (in)	2.5	
External continuity plate width	*C4* (in)	8.25	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Net area of external continuity plate cross section, *T5([C4]-($d_{hole,std}$+1/16))*	A_n (in^2)	17.031	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	1107.0	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	830.3	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.81	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	10	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	747.9	
External continuity plate dimension	*C2 (in)*	-6.125	
Cover plate overhang dimension	*C3* (in)	2.75	
External continuity plate dimension	*C5* (in)	3.5	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	-3.375	
External continuity plate thickness (see step 2)	*T5* (in)	2.5	
External continuity plate width	*C4* (in)	8.25	
Diagonal bonus (AISC 360 B4.3b), *[C9]^2 (T5)/4[C5]*	*Bonus* (in)	2.034	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Area of external continuity plate cross section with bonus, T5*([C4]-$d_{hole,std}$-1/16) + Bonus*	A_n (in^2)	19.065	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	1239.2	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	929.4	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.80	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	11	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	822.7	
Bolt spacing (see schedule)	*s* (in)	4	
External continuity plate dimension	*C2 (in)*	-6.125	
Cover plate overhang dimension	*C3* (in)	2.75	
External continuity plate dimension	*C5* (in)	3.5	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	0.625	
External continuity plate thickness (see step 2)	*T5* (in)	2.5	
External continuity plate width	*C4* (in)	8.25	
Diagonal bonus (AISC 360 B4.3b), *[C7]^2 (T5)/(4[C5])*	*Bonus* (in)	0.070	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Area of external continuity plate cross section with bonus, *(T5)([C4]-$d_{hole,std}$-1/16) + Bonus*	A_n (in^2)	17.101	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	1111.6	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	833.7	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.99	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	10	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	747.9	
External continuity plate width	*C4* (in)	8.25	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	-3.375	
External continuity plate thickness (see step 2)	*T5* (in)	2.5	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	20.625	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	8.438	
Net tension area, A_{gt} - *((T5)*($d_{hole,std}$+1/16))*	A_{nt} (in^2)	17.031	
Net shear area, A_{gv}	A_{nv} (in^2)	8.438	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	1360.2	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	1020.1	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.73	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	11	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	822.7	
Bolt spacing (see schedule)	s (in)	4	
External continuity plate dimension	*C2 (in)*	-6.125	
Cover plate overhang dimension	*C3* (in)	2.75	
External continuity plate width	*C4* (in)	8.25	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	0.625	
External continuity plate thickness (see step 2)	*T5* (in)	2.5	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	20.625	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	1.563	
Net tension area, $A_{gt}-((T4)(d_{hole}+1/16))$	A_{nt} (in^2)	17.031	
Net shear area, A_{gv}	A_{nv} (in^2)	1.563	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	1153.9	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	865.4	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.95	**OK**

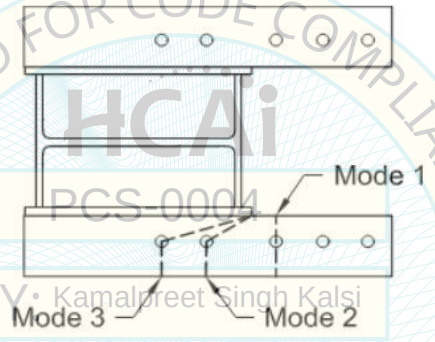


Fig. C-15.6. Tension rupture limit state for external continuity plate.

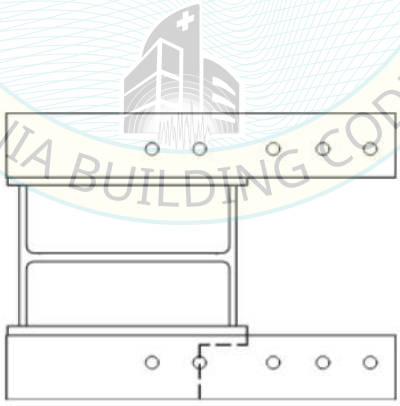


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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)	98000	
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	656.7	
Beam depth	d_b (in)	41.3	
Beam web thickness	t_{wb} (in)	1.34	
Beam web area, $d_b t_{wb}$	A_w (in)	55.342	
Beam web width-to-thickness ratio, h / t_w	l_{web}	25.5	
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)	1660.3	
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.40	**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)	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	2243.8	
Beam flange thickness	t_{fb} (in)	2.36	
Beam flange width	b_{fb} (in)	16.2	
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Number of holes along each bolt line (see step 5)	n_b	15	
Length of connection on beam	$B4$ (in)	57.75	
Distance from beam centerline to bolt line	$P2$ (in)	3.75	
Gross tension area, $(b_{fb}-2[P2])t_{fb}$	A_{gt} (in^2)	20.532	
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	272.580	
Net tension area, $A_{gt}-(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	17.140	
Net shear area, $A_{gv}-(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	174.198	
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)	7907.8	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	5930.8	
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.38	**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)	656.7	
Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	1.227	
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	103.1	
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	77.3	
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	8	
Provided number of shear tab bolts	n_n	9	**OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	34	
Required length of shear tab (AISC 358 Eq. 15.6-22), $T_{beam} - 1in$	$l_{tab,req}$ (in)	33	
Length of shear tab (see drawings), $T_{beam} - 1in$	l_{tab} (in)	33	**OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	1.34	
Shear tab provided thickness	$T3$ (in)	1.25	
Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	1.3	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}t_{min}F_{u,min}n_n$	R_{nb} (kips)	2193.8	
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Edge distance on shear tab (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	3.719	
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)	2.3	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{u,min}\ (n_n\ -1)$	R_{nt1} (kips)	1828.1	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{u,min}$	R_{nt2} (kips)	91.4	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1919.5	
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	927.8	
Controlling strength, $min(R_{nb}, R_{nt,} R_{nv})$	R_n (kips)	927.8	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	695.8	
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.94	**OK**

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

Minimum fastener tension (AISC 360 Table J3.1)	T_b	102	
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)	51.9	
Provided number of shear tab bolts (see step 12)	n_n	9	
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	466.8	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	656.7	
Distance from alignment line to column	$C1$ (in)	10.125	
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)	6649.2	

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Slotted bolt hole width (standard slot width)	w_b (in)	1.375	
Shear tab provided thickness	$T3$ (in)	1.25	
Shear tab length (see step 12)	l_{tab} (in)	33	
Beam T dimension limit	T_{beam} (in)	34	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	3.6875	
Gross area of shear tab, $T3l_{tab}$	A_{gv} (in^2)	41.25	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16)T3$	A_{nv} (in^2)	25.08	
Effective net area, $A_e = A_{nv}$	A_e (in^2)	25.08	
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{uST}A_{nv}$	ϕR_{nv} (kips)	733.54	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{uST}A_e$	ϕR_{nn} (kips)	1222.56	
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	0.79	**OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	656.71	
Required axial strength of the shear tab (see above)	P_u (kips)	466.80	
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)	6649.22	
Shear tab gross area, $T3l_{tab}$	A_{gv} (in^2)	41.25	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	340.31	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yST}A_{gv}$	ϕR_{nv} (kips)	1237.50	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yST}A_{gv}$	ϕR_{nn} (kips)	1856.25	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_{yST}Z_{tab}$	ϕM_n (kips)	15314.06	
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	0.58	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	33	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	656.7	
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	19.9	
Number of slotted bolt holes in shear tab	n_{vb}	9	
Slotted bolt hole spacing	s_{slot} (in)	3.6875	
Distance from line of bolts to shear tab weld	$C1$ (in)	10.125	
Size of weld in 1/16^{ths} of an inch	D_{W3}	20	
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)	6649.2	
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)	51.0	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	54.8	
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)	55.7	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.98	**OK**
This check is N/A when bridge plates do not apply			

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e (in)	1.75	
Beam depth	d_b (in)	41.3	
Length of the shear tab (see step 12)	l_{tab} (in)	33	
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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (in)	2.749	
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	2.75	**OK**
Provided outside shear tab slot dimension, [S2] = [S1]	$S2$ (in)	2.75	

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

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1121.9	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	2.5	
Top plate dimension (see schedule)	$P8$ (in)	60	
Gross shear area of top plate, (P8)(T4)	A_g (in^2)	150.000	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	4500.0	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	4500.0	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.25	**OK**

Rupture check			
Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.625	
Number of short slotted bolts on top plate (see step 5)	n_b	15	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	89.063	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	3473.4	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	2605.1	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.43	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

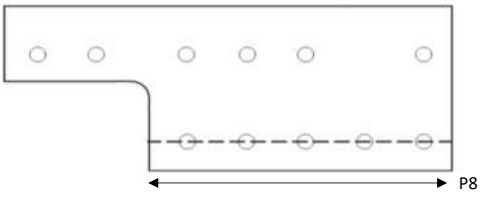


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1121.9
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1121.9

Yielding check (LS 1)

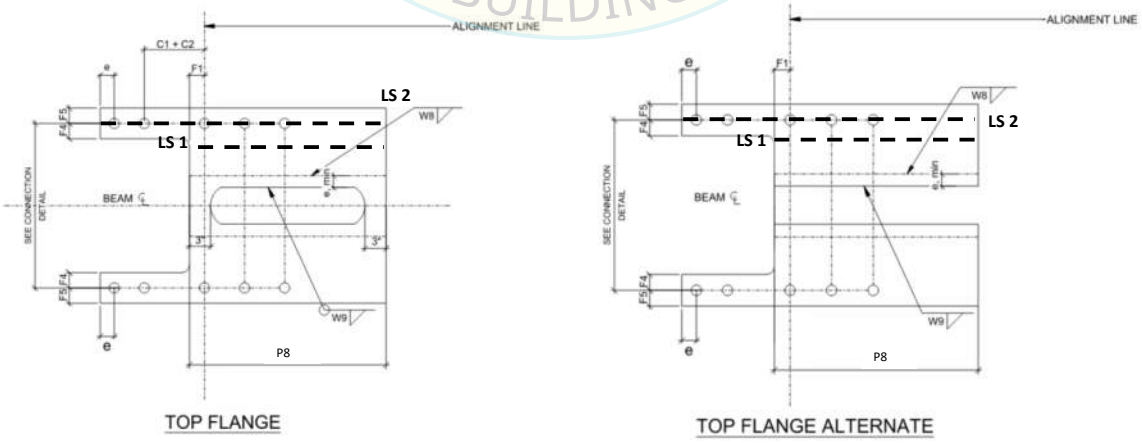
Top plate thickness (see step 2)	*T4* (in)	2.5	
Top plate dimension (see schedule)	*P8* (in)	55.5	
Fuse plate dimension	*F1 (in)*	2.00	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	138.750	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	4162.5	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	4162.5	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.27	**OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
Number of oversized bolts in Zone M (see step 5)	n_m	7	
Number of oversized bolts on top plate (see step 5)	n_b	15	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.625	
Bolt spacing (see schedule)	*s* (in)	4	
Fuse plate dimension	*F1 (in)*	2.00	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	83.125	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	207.813	
Number of oversized bolts on top plate (see step 5)	n_b	15	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	149.219	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	5819.5	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	4364.6	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.26	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	1121.9	
Beam flange width	b_{fb} (in)	16.2	
Weld 8 size	D_{W8} (1/16 in)	8	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.50	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	54.50	
Weld 9 size	D_{W9} (1/16 in)	8	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	49.50	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	10.8	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	11.1	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.97	**OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Total number of bolts (see step 5)	n_b	15	
Number of bolts in the M region (see step 5)	n_m	7	
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)	523.6	

Yielding check

Top plate dimension	*P4* (in)	2.25	
Top plate dimension	*P5* (in)	3.75	
Top plate thickness (see step 2s)	*T4* (in)	2.5	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	15.000	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	750.0	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	675.0	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.78	**OK**

Rupture check

Top plate dimension	*P4* (in)	2.25	
Top plate dimension	*P5* (in)	3.75	
Top plate thickness (see step 2)	*T4* (in)	2.5	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
Effective area of rupture, *([P4]+[P5]-d_O-1/16)(T4)*	A_e (in^2)	10.938	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	710.9	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	533.2	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.98	**OK**

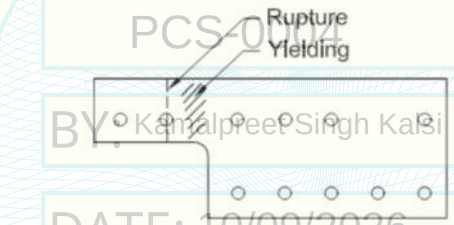


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

Entering and tightening check

Beam flange fillet dimension	k_1	1.9	
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.5625	
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3.4375	
Provided P2 dimension	*P2* (in)	3.75	**OK**

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Top plate thickness (see step 2)	*T4* (in)	2.5	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	*P10* (in)	13.2	
Inside length of the top plate	*P8* (in)	60.00	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	16.621	
Dimension, *([P8] - m) / 2*	*e* (in)	21.689	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.0	
Provided P2 dimension	*P2* (in)	3.75	**OK**

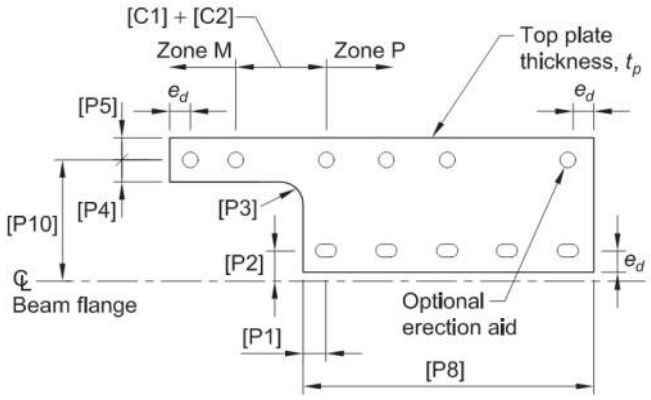


Fig. 15.6. Top plate detail.

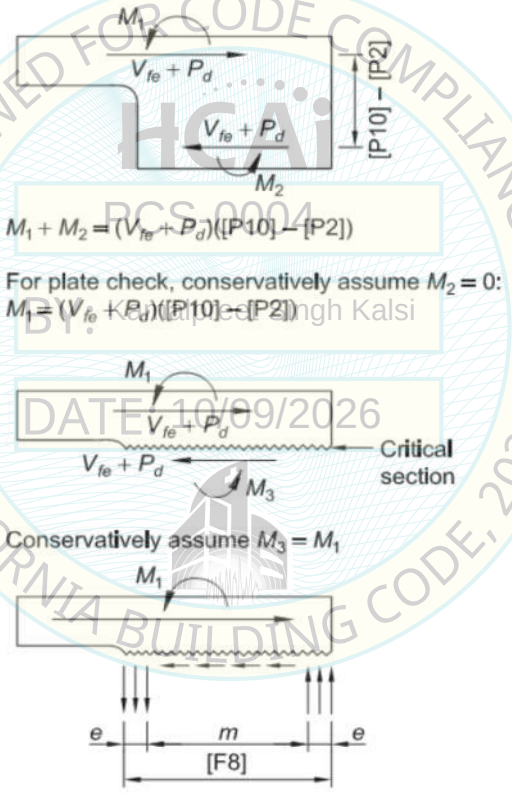


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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)	4	**OK**
Fuse plate thickness (see step 2)	$T2$ (in)	2.25	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	1.8	**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)	1121.9	
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)	$n_{b,\ Bot}$	15	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	7	
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)	1047.1	
Fuse plate dimension, centerline to gage line	$F7$ (in)	3.75	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	3.2	
Fuse plate thickness (see step 2)	$T2$ (in)	2.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
Gross area for yielding, $2([F7]+[F8])T2$	A_g (in^2)	31.275	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	1563.8	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	1407.4	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.74	**OK**

Step 18. (continued)

Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Fuse plate dimension	$F7$ (in)	3.75	
Fuse plate dimension	$F8$ (in)	3.2	
Fuse plate thickness (see step 2)	$T2$ (in)	2.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.563	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	23.963	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	P_n (kips)	1869.1	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	1401.8	
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.75	**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

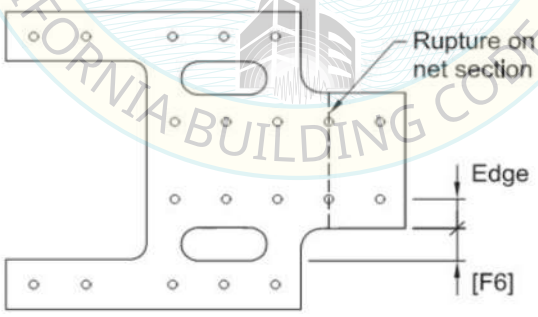


Fig. C-15.11. Limit states related to dimension [F6].

HCAi
PCS-0004
BY Kamaldeep Singh Kalsi
UPDATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	4	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	98000	
Fuse plate thickness (see step 2)	*T2* (in)	2.25	
Top plate thickness (see step 2)	*T4* (in)	2.5	
Beam depth	d_b (in)	41.3	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	656.7	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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_n	9	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	311.2	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	326.8	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	130.7	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	4.639	
Provided yielding region depth	*F2* (in)	4.625	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	624.4	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	1121.9	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.8	**OK**

Fuse plate yielding region geometry			
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.86	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1121.9	
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)	$n_{b,\ Bot}$	15	
Number of bolts in the M region	$n_{m,\ Bot}$	7	
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)	1047.1	

Yielding check

Fuse plate dimension	*F4* (in)	2.25	
Fuse plate dimension	*F5* (in)	3.375	
Fuse plate thickness (see step 2)	*T2* (in)	2.25	
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
Gross area of yielding, *2([F4]+[F5])(T2)*	A_g (in^2)	25.3125	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	R_n (kips)	1265.63	
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	1139.06	
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.92	**OK**

Rupture check

Fuse plate dimension	*F4* (in)	2.25	
Fuse plate dimension	*F5* (in)	3.375	
Fuse plate thickness (see step 2)	*T2* (in)	2.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
Effective net area of rupture, *2([F4]+[F5]-d_O-1/16)(T2)*	A_e (in^2)	18	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	R_n (kips)	1404.0	
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	1053.0	
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

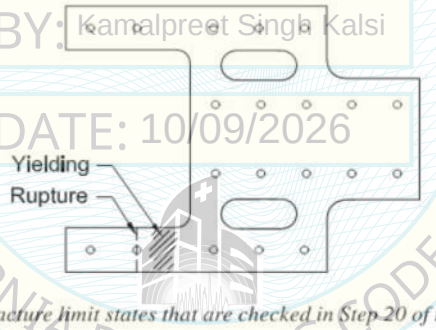


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

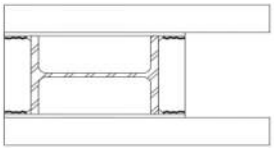
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

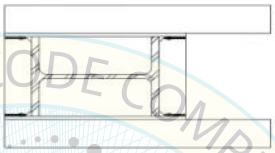
Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1121.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	0	
External continuity plate dimension	*C5* (in)	3.5	
Column depth	d_c (in)	40.2	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	2.75	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}$ *T4([C3]-1")*	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**



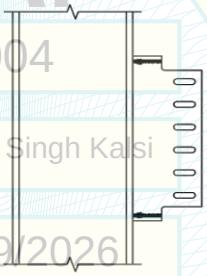
Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



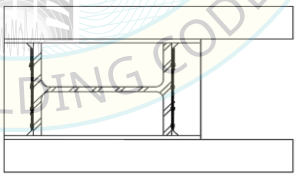
Shear tab to bridge plate weld failure check

Minimum bay width	*B* (in)	360.0	
Column depth	d_c (in)	40.2	
Beam depth	d_b (in)	41.3	
Shear force from gravity	$V_{gravity}$ (kips)	43.8	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	98000	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	656.7	
Distance from alignment line to column	*C1* (in)	10.125	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2.75	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



Bridge plate to column flange weld failure check

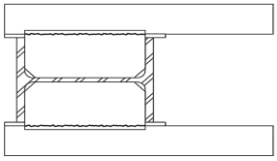
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	10.125	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2.75	
Column flange width	b_{fc} (in)	15.9	
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	5.5	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



*These checks are N/A when bridge plates do not apply

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

Column depth	d_c (in)	40.2	
Column flange thickness	t_{fc} (in)	1.81	
Weld 4 size	D_{W4} (1/16 in)	11	
Weld 4 effective length, d_c - 2 t_{fc}	l_{eff} (in)	36.58	
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)	560.1	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.875	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	960.2	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**



*This check is N/A when shear plates do not apply

HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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)	34800
Column flange width	b_{fc} (in)	15.9
Beam depth	d_b (in)	41.3
Bay width	B (in)	360.0
Number of bolts in zone M	M	7
Number of bolts in zone P	P	8
Bolt spacing	s (in)	4
Cover plate thickness	$T1$ (in)	1.75
Fuse plate thickness	T2 (in)	2.25
Top plate thickness	$T4$ (in)	2.5
External continuity plate dimension	$C5$ (in)	3.5
Fuse plate dimension	$F2$ (in)	4.625
Fuse plate dimension	$F3$ (in)	22.5
Fuse plate dimension	$F4$ (in)	2.3
Fuse plate dimension	$F6$ (in)	4
Fuse plate dimension	$F8$ (in)	3.2
Distance from beam centerline to bolt line	$P2$ (in)	3.75
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	3.56E+05
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	5.83E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	7.46E+04
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	2.50E+05
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	5.35E+04
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	2.61E+05
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	8.47E+07
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	5.05E+07
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.60 **OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

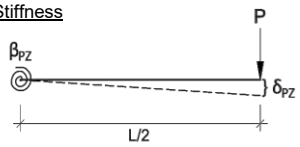
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

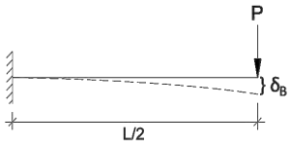


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	41.3
Beam flange thickness	t_{fb} (in)	2.36
Column depth	d_c (in)	40.2
Column flange width	b_{fc} (in)	15.9
Column web thickness	t_{wc} (in)	1
Column flange thickness	t_{fc} (in)	1.81
Cover plate overhang	$C3$ (in)	2.8
Cover plate thickness	$T1$ (in)	1.75
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	8266.2
Column ratio, $(d_c - t_{fc}) / B$	α	0.107
Beam ratio, $(d_b - t_{fb}) / H$	β	0.180
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	1.82E+08
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	8.95E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	1.83E+08
Beam moment of inertia	I_{xb} (in^3)	34800
Required panel zone spring stiffness, *$60EI_{xb} / B$ for one-sided, $120EI_{xb} / B$ for two-sided*	K_{req} (k-in)	1.68E+08
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.92 **OK**

*This check is N/A when connection stiffness is applied in the model analysis

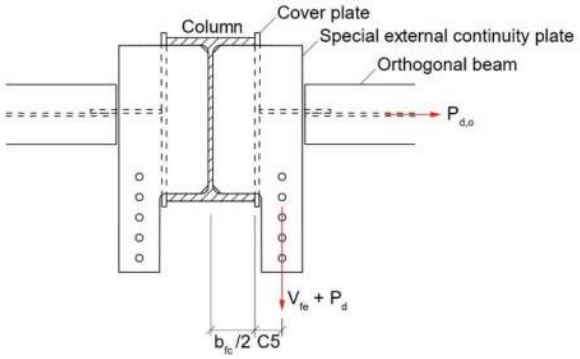
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

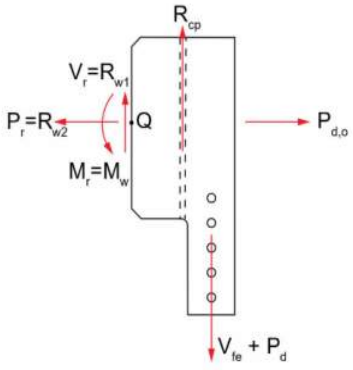
Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	1121.9
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	0.0
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
Column flange width	b_{fc} (in.)	15.9
Column depth	d_c (in.)	40.2
Column flange thickness	t_{fc} (in.)	1.81
Column web thickness	t_{wc} (in.)	1.00
External continuity plate thickness	$T5$ (in)	2.500
External continuity plate gage dimension	$C5$ (in.)	3.50
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	836.31	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	91.45	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
		Special detailing not required	

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	2.99	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	731.9	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	85.55	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply		Special detailing not required	





Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size			
Orthogonal beam depth	$d_{b,o}$ (in)		
In plane moment frame beam 1		W40X431	
In plane moment frame beam 2		NA	
Max in-plane moment frame beam depth	d_b (in)	41.3	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0	
New continuity plate required for deeper cantilever beam		N/A	
New continuity plate width past column flange	C4 (in)	8.25	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A	**N/A**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

Connection ID DFH101

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.*	M 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)	65
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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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.)	44	
Beam depth	d_b (in.)	14.1	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	38.0	**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}	6.6	**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.)	44	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	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), Factor based on code and member type, $F(E / (R_{yc} F_{yc}))^{0.5}$	$\lambda_{hd,flange}$ / F	13.73 / 0.65	
Column flange width-to-thickness ratio	λ_{flange}	11.3	**OK**

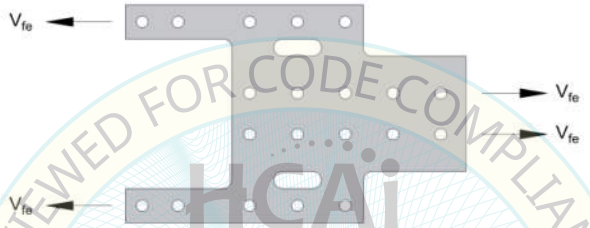
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	84.7	
Column design axial load	P_u (kips)	123.5	
Column area	A_{gc} (in^2)	24.7	
Story Height	H (in)	180.0	
Column depth	d_c (in)	10	
Beam depth	d_b (in)	14.1	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c * (F_{yc} - P_u / A_{gc}) * (H/2) / (H/2 - d_b/2 - d_c/4 - [C6])$	ΣM^*_{pc} (k-in)	8.86E+03	
Beam plastic section modulus	Z_b (in^3)	61.5	
Minimum 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)	3.26E+03	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	2.71	**OK**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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**
Fuse plate thickness	$T2$	(in)	0.625
Fuse plate specified maximum thickness	$T2_{max}$	(in)	2.25 **OK**
Top plate thickness	$T4$	(in)	0.625
Top plate specified maximum thickness	$T4_{max}$	(in)	3 **OK**
External continuity plate thickness	$T5$	(in)	0.625
External continuity plate specified maximum thickness	$T5_{max}$	(in)	3 **OK**
Beam depth	d_b	(in)	14.1
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2}	(kips)	0.0
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe}	(kips)	104.4



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	$P_{d,total}$	(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)	$V_{grav-ortho}$	(kips)	0
Fuse plate thickness	$T2$	(in)	0.625
Top plate thickness	$T4$	(in)	0.625
External continuity plate thickness	$T5$ (in)		0.625
Beam depth	d_b	(in)	14.1
Column depth	d_c	(in)	10
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$	(kips)	185.2
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$	(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), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$	(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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp}	(in)	20.10
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$	(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		0.41 **OK**

Step 3. (continued)

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, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	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 5a. Determine the number of bolts for each bolt line on the 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)	$P_{d,total}$ (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)	27.8	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	5.3	
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 (in)	1.0625	
Bolt spacing (see schedule)	s (in)	2.75	
Top plate thickness	$T4$ (in)	0.625	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	1.785	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.6163	
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	2	

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	
Top plate thickness	$T4$ (in)	0.625	
Governing material thickness, min$(t_{fb}, T4)$	t_{min} (in)	0.5	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	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, $e - d_O / 2$	l_{c2} (in)	0.6	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	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, $R_u / \phi R_n$	D/C	0.88	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	600	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	0.625	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	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 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	2.8	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	5	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.0625	
Bolt spacing (see schedule)	s (in)	2.75	
Fuse plate thickness	$T2$ (in)	0.625	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	1.785	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.6	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	3	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	2	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	104.4	
Beam flange thickness	t_{fb} (in)	0.515	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	291.2	
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, $e - d_O / 2$	l_{c2} (in)	0.6	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	23.9	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	315.1	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	185.6	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	185.6	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	139.2	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.75	**OK**

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kemalcan Cagatayalsi
DATE: 10/09/2026

Step 5b. (continued)

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, $M_u/(2(d_b+T2/2 + T4/2))$	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 on the bottom flange	$n_{b,Bot}$	5	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	59.3	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	50.4	
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)	14.1	
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	e (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), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	1.9085	
Distance from alignment line to column	*C1* (in)	5.125	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

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)	$P_{d,total}$ (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)	0	
Column depth	d_c (in)	10	
Column flange thickness	t_{fc} (in)	0.698	
Beam depth	d_b (in)	14.1	
Fuse plate thickness	*T2* (in)	0.625	
Top plate thickness	*T4* (in)	0.625	
External continuity plate thickness	*T5* (in)	0.625	
Cover plate thickness	*T1* (in)	0.75	
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3	
Cover plate height, d_b + 2[C6], d_b + [C6] - T5 or just d_b - 2T5 for cap plate and ortho. cant./drag condition	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), *T5 + T1 + [C6]*	l_{We1} (in)	4.375	
Weld 1 size	D_{W1} (1/16 in)	6	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
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)	0.00	
Vertical distance from center of rotation to geometric center	e_y (in)	0.77	

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.

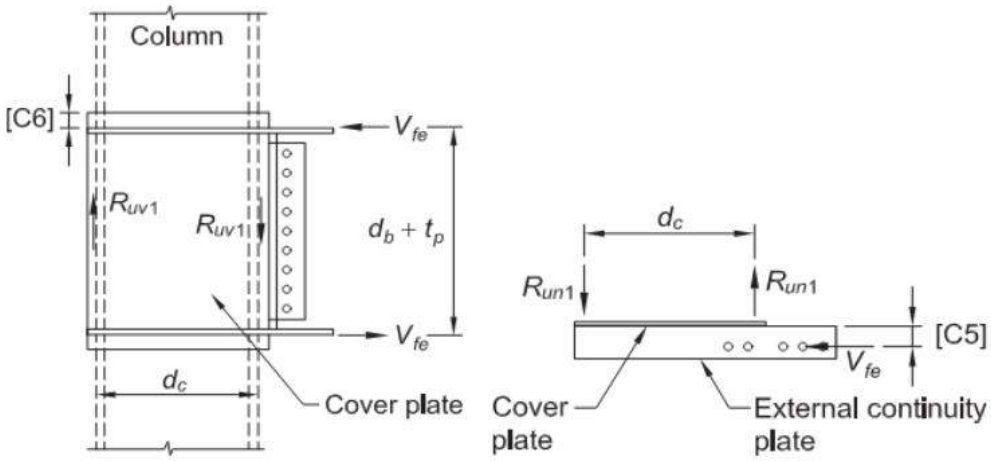
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	3791	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	11.29	6.00	63.73	67.7
W4TL	11.36	0.00	21.51	0.0
W1LB	9.94	5.97	59.80	59.3
W4BR	9.95	0.00	24.76	0.0
W1RC	5.09	3.67	10.87	18.7
W4TC	10.57	0.00	0.46	0.0
W1LC	5.09	3.67	10.87	18.7
W4BC	9.03	0.00	0.54	0.0
		6.00	63.73	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	6.0	
Orientation of Resultant Force	θ_r (degrees)	63.7	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.38	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	7.7	
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)	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)	1.26	
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)	11.77	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	*D/C*	0.94	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

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)	$P_{d,total}$ (kips)	85.5	
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} + *T1* + *[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)	*T5* (in)	0.625	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	262.5	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}T5((d_c+2*[C3])/2)^2$	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)	$P_{d,total}$ (kips)	85.5	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.8	
Column fillet dimension	*k* (in)	0.0	
Column fillet dimension	*k1* (in)	0.0	
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	46.5	
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)	40	
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)	517.8	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	445.4	
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)	1108.2	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	831.2	
Weld 2 demand/capacity check for cap plate configuration, $R_u/\phi R_n$	*D/C*	0.18	**OK**

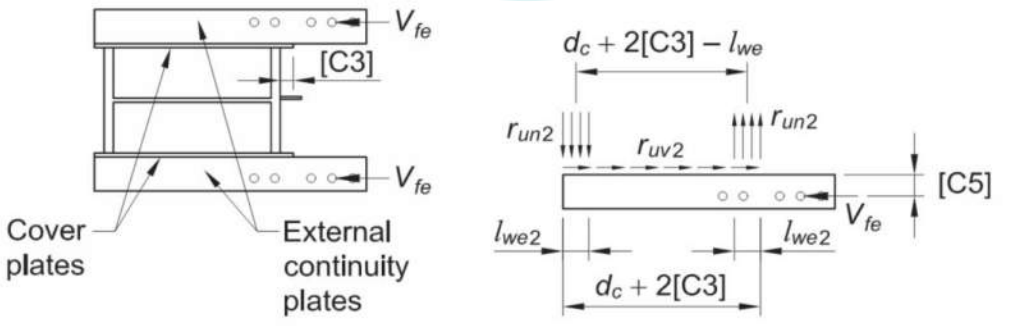


Fig. C-15.5. Weld demands for external continuity plate welds.

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
DESIGNED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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)	$P_{d,total}$ (kips)	85.5	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	42.8	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	4	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	98.1	
External continuity plate thickness (see step 2)	*T5* (in)	0.625	
External continuity plate width	*C4* (in)	4.875	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Net area of external continuity plate cross section, *T5([C4]-($d_{hole,std}$ +1/16))*	A_n (in^2)	2.500	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	162.5	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	121.9	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.81	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	5	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	122.6	
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)	*T5* (in)	0.625	
External continuity plate width	*C4* (in)	4.875	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2 (T5)/4[C5]$	*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, T5*([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	2.516	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	163.6	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	122.7	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	1.00	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	6	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	147.2	
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)	*T5* (in)	0.625	
External continuity plate width	*C4* (in)	4.875	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2 (T5)/(4[C5])$	*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, *(T5)([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	3.195	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	207.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	155.8	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.94	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	5	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	122.6	
External continuity plate width	*C4* (in)	4.875	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	0.5	
External continuity plate thickness (see step 2)	*T5* (in)	0.625	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	3.047	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	0.313	
Net tension area, A_{gt} - *((T5)*($d_{hole,std}$ +1/16))*	A_{nt} (in^2)	2.500	
Net shear area, A_{gv}	A_{nv} (in^2)	0.313	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	171.9	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	128.9	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.95	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	6	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	147.2	
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)	4.875	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	3.25	
External continuity plate thickness (see step 2)	*T5* (in)	0.625	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	0.8125	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	3.047	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	2.031	
Net tension area, A_{gt}-*((T4)(d_{hole}+1/16))*	A_{nt} (in^2)	2.500	
Net shear area, A_{gv}	A_{nv} (in^2)	2.031	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	223.4	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	167.6	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.88	**OK**

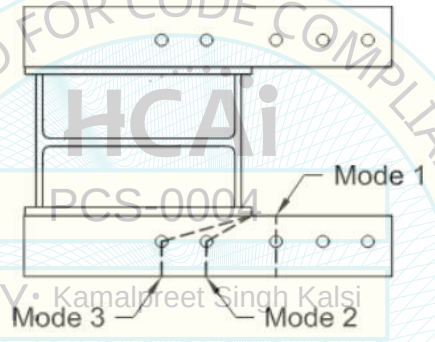


Fig. C-15.6. Tension rupture limit state for external continuity plate.

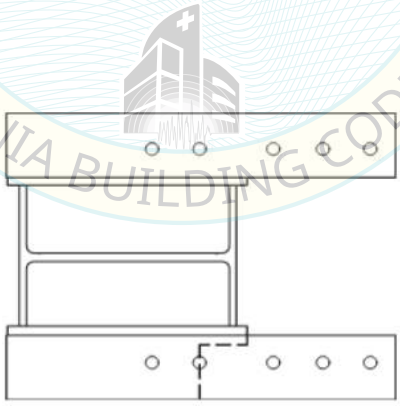


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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)	$P_{d,total}$ (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)	11	
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 provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	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 (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	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{u,min}\ (n_n\ -1)$	R_{nt1} (kips)	169.3	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.6	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{u,min}$	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	$T3$ (in)	0.375	
Shear tab length (see step 12)	l_{tab} (in)	11	
Beam T dimension limit	T_{beam} (in)	11.625	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1	
Slotted bolt hole spacing, $(T_{beam}\ - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	4.1875	
Gross area of shear tab, $T3l_{tab}$	A_{gv} (in^2)	3.98	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16)T3$	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), $(0.75)0.6F_{uST}A_{nv}$	ϕR_{nv} (kips)	87.75	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{uST}A_e$	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, $T3l_{tab}$	A_{gv} (in^2)	3.98	
Shear tab plastic section modulus, $(l_{tab}/2)^2\ T3$	Z_{tab} (in^3)	10.58	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yST}A_{gv}$	ϕR_{nv} (kips)	119.53	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yST}A_{gv}$	ϕR_{nn} (kips)	179.30	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_{yST}Z_{tab}$	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**

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (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**
This check is N/A when bridge plates do not apply			

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 (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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (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)

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (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)	T4 (in)	0.625	
Top plate dimension (see schedule)	P8 (in)	17.25	
Gross shear area of top plate, (P8)(T4)	A_g (in^2)	10.781	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	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, $A_g - (T4 l_s n_b)$	A_n (in^2)	7.031	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	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**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

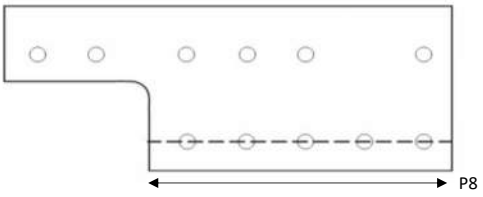


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile 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)	$P_{d,total}$ (kips)	85.5	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	42.8	
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	147.2	

Yielding check (LS 1)

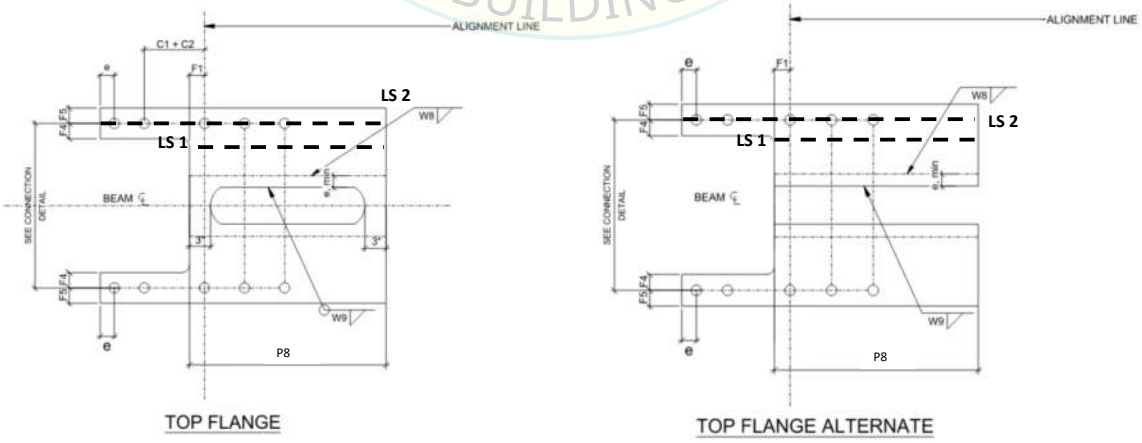
Top plate thickness (see step 2)	*T4* (in)	0.625	
Top plate dimension (see schedule)	*P8* (in)	9.125	
Fuse plate dimension	*F1 (in)*	1.75	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	5.703	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	171.1	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	171.1	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.86	**OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	0.75	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	0.9375	
Number of oversized bolts in Zone M (see step 5)	n_m	2	
Number of oversized bolts on top plate (see step 5)	n_b	6	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1	
Bolt spacing (see schedule)	*s* (in)	2.75	
Fuse plate dimension	*F1 (in)*	1.75	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	14.75	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	9.219	
Number of oversized bolts on top plate (see step 5)	n_b	6	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	5.703	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	222.4	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	166.8	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.88	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	147.2	
Beam flange width	b_{fb} (in)	6.77	
Weld 8 size	D_{W8} (1/16 in)	11	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.69	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	7.75	
Weld 9 size	D_{W9} (1/16 in)	11	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	3.13	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	13.5	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	15.3	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.88	**OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (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	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)	49.1	

Yielding check

Top plate dimension	*P4* (in)	1.375	
Top plate dimension	*P5* (in)	1.25	
Top plate thickness (see step 2s)	*T4* (in)	0.625	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	1.641	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	82.0	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	73.8	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.66	**OK**

Rupture check

Top plate dimension	*P4* (in)	1.375	
Top plate dimension	*P5* (in)	1.25	
Top plate thickness (see step 2)	*T4* (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)(T4)*	A_e (in^2)	1.016	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	66.0	
Factored tensile rupture capacity, *0.75P_n*	ϕP_n (kips)	49.5	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.99	**OK**

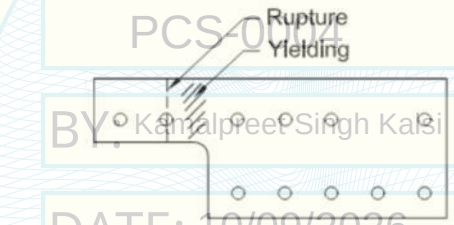


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0024
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

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)	$P_{d,total}$ (kips)	85.5	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	42.8	
Top plate thickness (see step 2)	*T4* (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, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*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), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.0	
Provided P2 dimension	*P2* (in)	2.375	**OK**

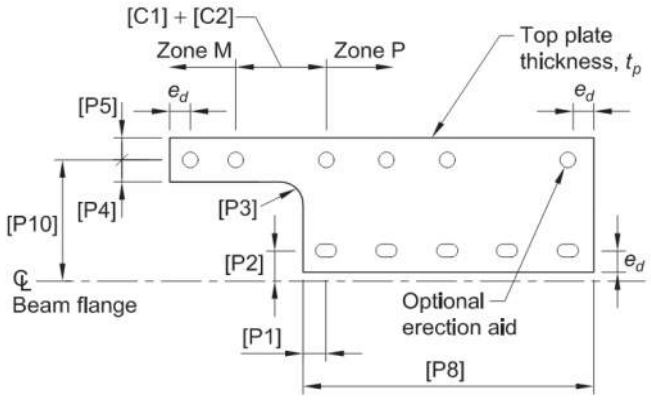


Fig. 15.6. Top plate detail.

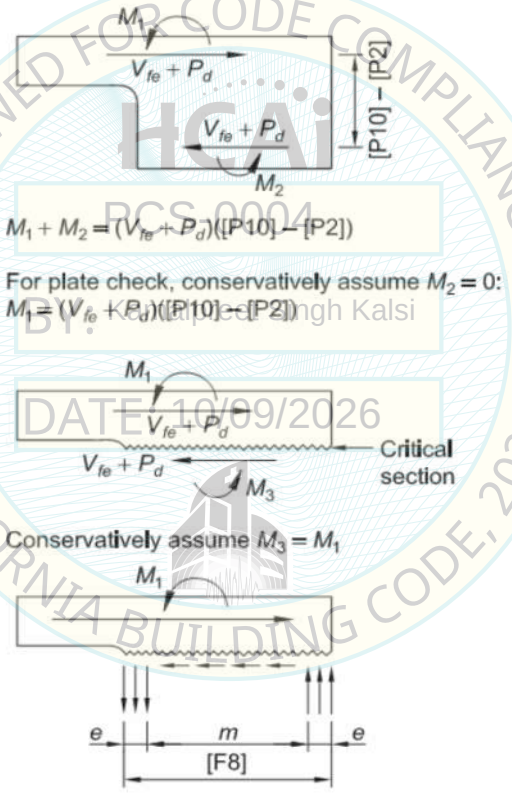


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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**
Fuse plate thickness (see step 2)	$T2$ (in)	0.625	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	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)	$n_{b,\ Bot}$	5	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	83.5	
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)	$T2$ (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, $2([F7]+[F8])T2$	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	0.29	**OK**

Step 18. (continued)

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)	$T2$ (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, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	5.156	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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	0.28	**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

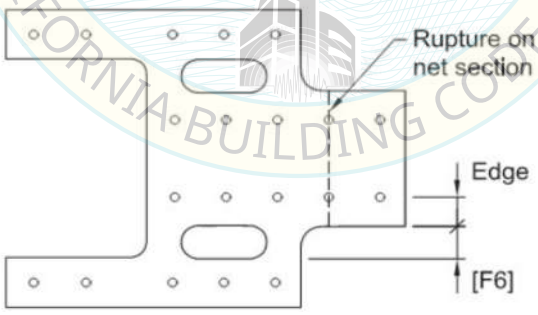


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	1.625	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	3075	
Fuse plate thickness (see step 2)	*T2* (in)	0.625	
Top plate thickness (see step 2)	*T4* (in)	0.625	
Beam depth	d_b (in)	14.1	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	37.7	
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_n	3	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	35.6	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	37.4	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	14.9	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	*F2* $_{max}$ (in)	1.542	
Provided yielding region depth	*F2* (in)	1.500	**OK**
Fuse yield force (one side), *2(0.6F* $_{yFuse}$ *)[F2]T2*	V_y (kips)	56.3	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	104.4	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.9	**OK**

Fuse plate yielding region geometry			
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	*(F6/F2)* $_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	1.05	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

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)	$n_{b,\ Bot}$	5
Number of bolts in the M region	$n_{m,\ Bot}$	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)	83.5

Yielding check

Fuse plate dimension	*F4* (in)	1.375	
Fuse plate dimension	*F5* (in)	1.125	
Fuse plate thickness (see step 2)	*T2* (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])(T2)*	A_g (in^2)	3.125	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	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*	0.59	**OK**

Rupture check

Fuse plate dimension	*F4* (in)	1.375	
Fuse plate dimension	*F5* (in)	1.125	
Fuse plate thickness (see step 2)	*T2* (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, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	1.875	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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*	0.76	**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

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
RETAINED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

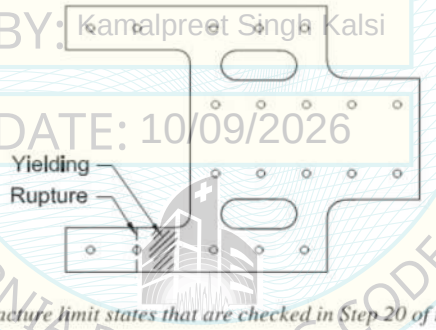


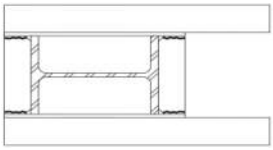
Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

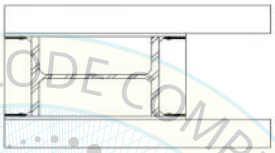
Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	104.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	85.5	
Drag force per line of bolts, P_d / 2	P_d (kips)	42.75	
External continuity plate dimension	*C5* (in)	2.375	
Column depth	d_c (in)	10	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	2	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}$ *T4([C3]-1")*	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**



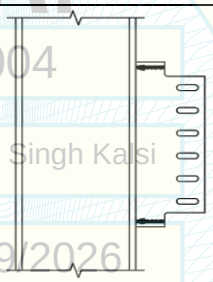
Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



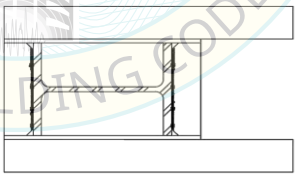
Shear tab to bridge plate weld failure check

Minimum bay width	*B* (in)	240.0	
Column depth	d_c (in)	10	
Beam depth	d_b (in)	14.1	
Shear force from gravity	$V_{gravity}$ (kips)	11.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	3075	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	37.7	
Distance from alignment line to column	*C1* (in)	5.125	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



Bridge plate to column flange weld failure check

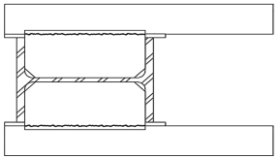
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	5.125	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2	
Column flange width	b_{fc} (in)	10	
Weld 6 effective length, $MIN(b_{fc}$ *- 2", 2[C3])*	l_{eff} (in)	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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



*These checks are N/A when bridge plates do not apply

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

Column depth	d_c (in)	10	
Column flange thickness	t_{fc} (in)	0.698	
Weld 4 size	D_{W4} (1/16 in)	0	
Weld 4 effective length, $d_c - 2\,t_{fc}$	l_{eff} (in)	8.604	
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)	N/A	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.000	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{ySP}\,t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	N/A	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	385
Column flange width	b_{fc} (in)	10
Beam depth	d_b (in)	14.1
Bay width	B (in)	240.0
Number of bolts in zone M	M	2
Number of bolts in zone P	P	4
Bolt spacing	s (in)	2.75
Cover plate thickness	$T1$ (in)	0.75
Fuse plate thickness	T2 (in)	0.625
Top plate thickness	$T4$ (in)	0.625
External continuity plate dimension	$C5$ (in)	2.375
Fuse plate dimension	$F2$ (in)	1.5
Fuse plate dimension	$F3$ (in)	5.375
Fuse plate dimension	$F4$ (in)	1.4
Fuse plate dimension	$F6$ (in)	1.625
Fuse plate dimension	$F8$ (in)	2.75
Distance from beam centerline to bolt line	$P2$ (in)	2.375
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	4.26E+04
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	1.29E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	1.05E+04
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	2.13E+04
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	8.25E+03
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	3.01E+04
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	1.40E+06
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	8.37E+05
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.60 **OK**

DETAILED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

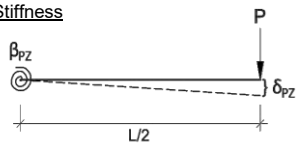
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

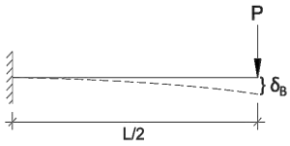


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	14.1
Beam flange thickness	t_{fb} (in)	0.515
Column depth	d_c (in)	10.0
Column flange width	b_{fc} (in)	10
Column web thickness	t_{wc} (in)	0.698
Column flange thickness	t_{fc} (in)	0.698
Cover plate overhang	$C3$ (in)	2.0
Cover plate thickness	$T1$ (in)	0.75
Bay width	B (in)	240.0
Story height	H (in)	180.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	394.5
Column ratio, $(d_c - t_{fc}) / B$	α	0.039
Beam ratio, $(d_b - t_{fb}) / H$	β	0.075
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	5.63E+06
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	5.42E+04
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	5.69E+06
Beam moment of inertia	I_{xb} (in^3)	385
Required panel zone spring stiffness, *$60EI_{xb} / B$ for one-sided, $120EI_{xb} / B$ for two-sided*	K_{req} (k-in)	2.79E+06
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.49 **OK**

*This check is N/A when connection stiffness is applied in the model analysis

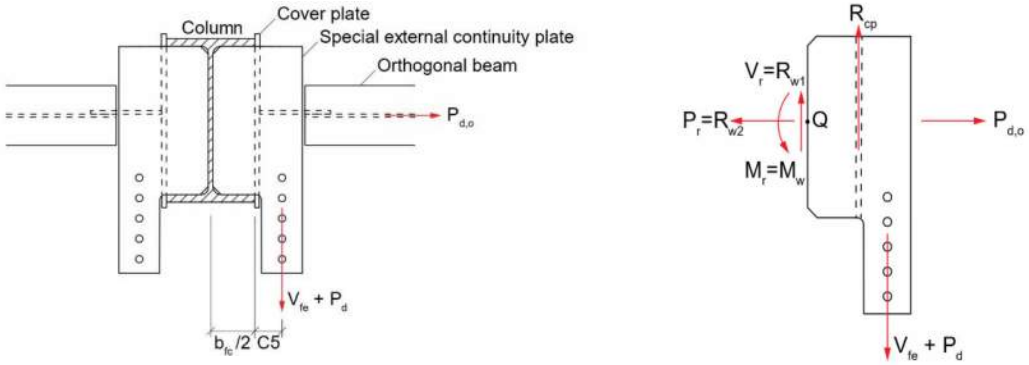
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	104.4
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	85.5
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	42.8
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
Column flange width	b_{fc} (in.)	10.0
Column depth	d_c (in.)	10.0
Column flange thickness	t_{fc} (in.)	0.70
Column web thickness	t_{wc} (in.)	0.70
External continuity plate thickness	$T5$ (in)	0.625
External continuity plate gage dimension	$C5$ (in.)	2.38
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	11.57	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	5.38	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
	Special detailing not required		

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	1.045	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	9.8	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	4.94	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply	Special detailing not required		



Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Ikampal Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size			
Orthogonal beam depth	$d_{b,o}$ (in)		
In plane moment frame beam 1		W14X38	
In plane moment frame beam 2		NA	
Max in-plane moment frame beam depth	d_b (in)	14.1	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0	
New continuity plate required for deeper cantilever beam		N/A	
New continuity plate width past column flange	C4 (in)	4.875	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A	**N/A**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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.)	44	
Beam depth	d_b (in.)	24.3	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	94.0	**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}	5.2	**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.)	44	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	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), Factor based on code and member type, $F(E/(R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	
	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	6.45	**OK**

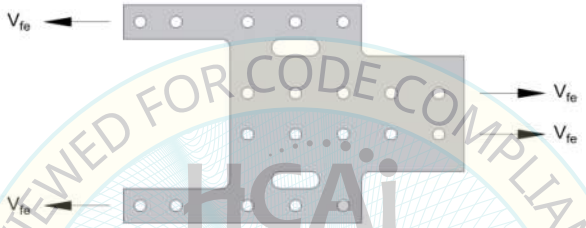
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	307	
Column design axial load	P_u (kips)	53.5	
Column area	A_{gc} (in^2)	35.9	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	24.3	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c*(F_{yc}-P_u/A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	N/A	
Beam plastic section modulus	Z_b (in^3)	254	
Minimum 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)	N/A	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	N/A	**N/A**

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

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

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**
Fuse plate thickness	$T2$ (in)	0.875	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	0.875	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	0.875	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	24.3	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	189.4	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	378.7	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	$P_{d,total}$ (kips)	0.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	0.0	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	378.7	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	35	
Fuse plate thickness	$T2$ (in)	0.875	
Top plate thickness	$T4$ (in)	0.875	
External continuity plate thickness	$T5$ (in)	0.875	
Beam depth	d_b (in)	24.3	
Column depth	d_c (in)	21.7	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	456.9	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	1.125	
Cover plate overhange dimension	$C3$ (in)	2	
Column depth	d_c (in)	21.7	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	25.7	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	867.4	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	867.4	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.44	**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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	26.43	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	891.8	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	891.8	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.51	**OK**

Step 3. (continued)

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, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	2.39	**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 5a. Determine the number of bolts for each bolt line on the 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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
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)	49.5	
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	5	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.375	
Bolt spacing (see schedule)	s (in)	3.25	
Top plate thickness	$T4$ (in)	0.875	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	2.312	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.4999	
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)	189.4	
Beam flange thickness	t_{fb} (in)	0.875	
Top plate thickness	$T4$ (in)	0.875	
Governing material thickness, min$(t_{fb}, T4)$	t_{min} (in)	0.9	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	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, $e - d_O / 2$	l_{c2} (in)	0.8	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	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	0.77	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1451	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	0.875	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	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 5b. Determine the number of bolts for each bolt line on the fuse plate and check limit states

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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	2.9	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	5	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.375	
Bolt spacing (see schedule)	s (in)	3.25	
Fuse plate thickness	$T2$ (in)	0.875	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	2.312	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.5	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	3	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	2	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	189.4	
Beam flange thickness	t_{fb} (in)	0.875	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	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, $e - d_O / 2$	l_{c2} (in)	0.8	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	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	0.77	**OK**

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kemalettin Cagatayhalsi
DATE: 10/09/2026

Step 5b. (continued)

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, $M_u/(2(d_b+T2/2 + T4/2))$	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 on the bottom flange	$n_{b,Bot}$	5	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	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 (in)	1.375	
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), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	2.833	
Distance from alignment line to column	*C1* (in)	6.875	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

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)	$P_{d,total}$ (kips)	0.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	0	
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	
Fuse plate thickness	*T2* (in)	0.875	
Top plate thickness	*T4* (in)	0.875	
External continuity plate thickness	*T5* (in)	0.875	
Cover plate thickness	*T1* (in)	1.125	
Cover plate dimension, vertical extension outside panel zone	*C6* (in)	3	
Cover plate height, d_b + 2[C6], d_b + [C6] - T5 or just d_b - 2T5 for cap plate and ortho. cant./drag condition	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)*	0.6250	
Weld 1 length, h_{cp} - 2l_{hb1}	l_{W1} (in)	25.18	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), *T5 + T1 + [C6]*	l_{We1} (in)	5.000	
Weld 1 size	D_{W1} (1/16 in)	8	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
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)	0.77	
Vertical distance from center of rotation to geometric center	e_y (in)	0.00	

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.

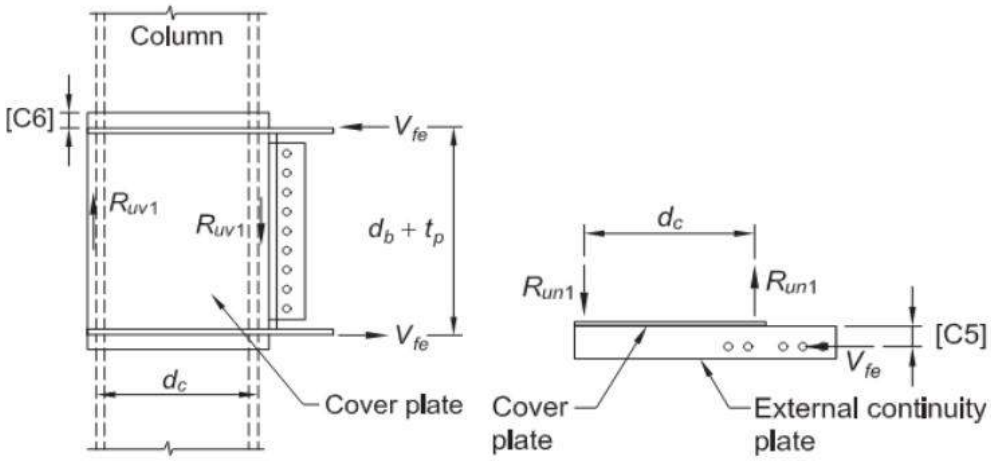
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	19994	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	15.93	9.91	50.76	157.9
W4TL	18.21	5.95	35.08	108.3
W1LB	16.95	9.63	46.70	163.2
W4BR	17.37	5.94	30.90	103.1
W1RC	10.08	6.21	1.43	62.6
W4TC	14.93	4.96	3.73	74.1
W1LC	11.63	6.39	1.24	74.3
W4BC	14.93	4.96	3.73	74.1
		9.91	50.76	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	9.9	
Orientation of Resultant Force	θ_r (degrees)	50.8	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.50	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	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)	1.27	
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)	14.76	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	*D/C*	0.64	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0	
Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate dimension	*C5* (in)	2.75	
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} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	3.59	
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)	14.7	
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)	13.8	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	20.2	
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*	0.91	**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)*	378.7	
External continiuty plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	1041.6	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	378.7	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	674.6	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}\,T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	6501.7	
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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
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)	87.0	
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)	1291.1	
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)	2068.4	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1551.3	
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	*D/C*	0.24	**OK**

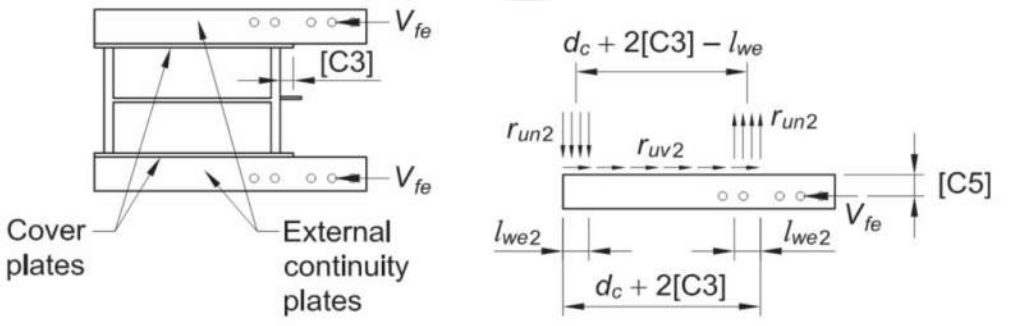


Fig. C-15.5. Weld demands for external continuity plate welds.

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
DESIGNED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	3	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	113.6	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
External continuity plate width	*C4* (in)	4.125	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Net area of external continuity plate cross section, *T5([C4]-($d_{hole,std}$ +1/16))*	A_n (in^2)	2.570	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	167.1	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	125.3	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.91	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	4	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	151.5	
External continuity plate dimension	*C2 (in)*	0.75	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate dimension	*C5* (in)	2.75	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	2.75	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
External continuity plate width	*C4* (in)	4.125	
Diagonal bonus (AISC 360 B4.3b), *[C9]^2 (T5)/4[C5]*	*Bonus* (in)	0.602	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Area of external continuity plate cross section with bonus, *T5([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	3.172	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	206.2	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	154.6	
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, $n_{rup1} + 2$	n_{rup}	5	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	189.4	
Bolt spacing (see schedule)	*s* (in)	3.25	
External continuity plate dimension	*C2 (in)*	0.75	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate dimension	*C5* (in)	2.75	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	6	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
External continuity plate width	*C4* (in)	4.125	
Diagonal bonus (AISC 360 B4.3b), *[C7]^2 (T5)/(4[C5])*	*Bonus* (in)	2.864	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Area of external continuity plate cross section with bonus, *(T5)([C4]-$d_{hole,std}$ -1/16) + Bonus*	A_n (in^2)	5.434	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	353.2	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	264.9	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.71	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	4	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	151.5	
External continuity plate width	*C4* (in)	4.125	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	2.75	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	3.609	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	2.406	
Net tension area, *A_{gt} - ((T5)*($d_{hole,std}$ +1/16))*	A_{nt} (in^2)	2.570	
Net shear area, A_{gv}	A_{nv} (in^2)	2.406	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	239.3	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	179.4	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.84	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	5	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	189.4	
Bolt spacing (see schedule)	s (in)	3.25	
External continuity plate dimension	*C2 (in)*	0.75	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate width	*C4* (in)	4.125	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	6	
External continuity plate thickness (see step 2)	*T5* (in)	0.875	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	3.609	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	5.250	
Net tension area, A_{gt}-*((T4)(d_{hole}+1/16))*	A_{nt} (in^2)	2.570	
Net shear area, A_{gv}	A_{nv} (in^2)	5.250	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	324.6	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	243.4	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.78	**OK**

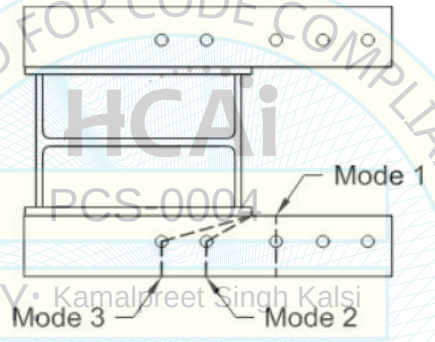


Fig. C-15.6. Tension rupture limit state for external continuity plate.

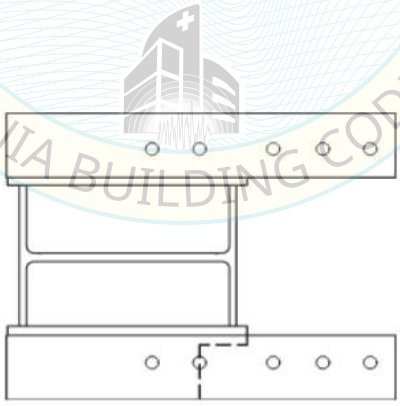


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	378.7	
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	0.67	**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 provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	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 (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	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1}\ t_{min}\ F_{u,min}\ (n_n - 1)$	R_{nt1} (kips)	511.9	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.7	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2}\ t_{min}\ F_{u,min}$	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), $1.5 T_b \mu D_u h_f n_s$	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	$T3$ (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 (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, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) T3$	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), $(0.75) 0.6 F_{uST} A_{nv}$	ϕR_{nv} (kips)	208.41	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75) F_{uST} A_e$	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, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	45.13	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0) 0.6 F_{yST} A_{gv}$	ϕR_{nv} (kips)	285.00	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9) F_{yST} A_{gv}$	ϕR_{nn} (kips)	427.50	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9) F_{yST} Z_{tab}$	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**

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	19	
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)	3.3	
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}	8	
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)	17.9	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	18.2	
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)	22.3	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.82	**OK**

This check is N/A when bridge plates do not apply

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 (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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (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)

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	189.4	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	0.875	
Top plate dimension (see schedule)	$P8$ (in)	17	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	14.875	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	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	0.42	**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, $A_g - (T4 l_s n_b)$	A_n (in^2)	9.133	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	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	0.71	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

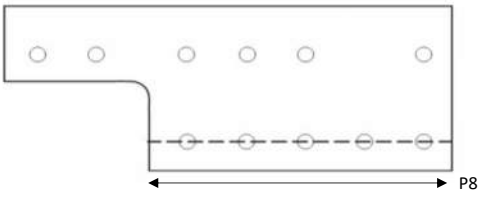


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile 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)	$P_{d,total}$ (kips)	0.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	189.4

Yielding check (LS 1)

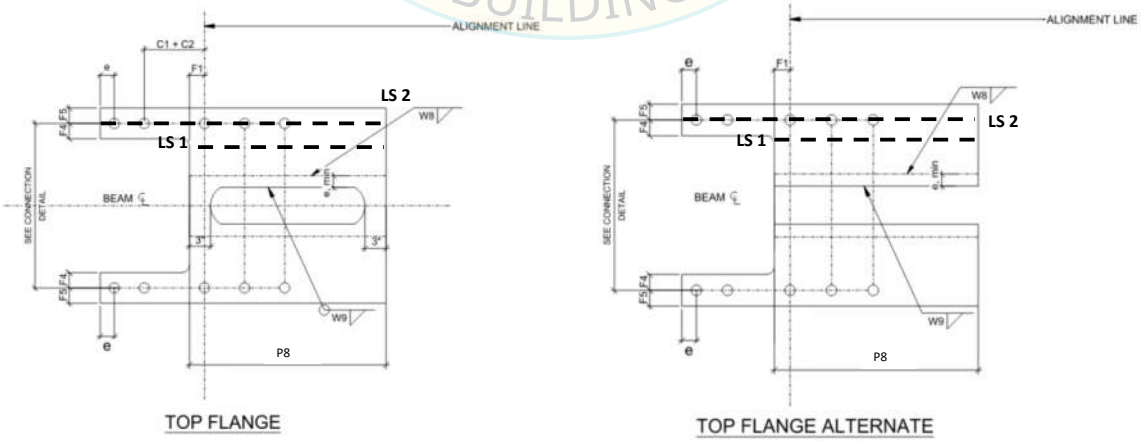
Top plate thickness (see step 2)	*T4* (in)	0.875	
Top plate dimension (see schedule)	*P8* (in)	11.75	
Fuse plate dimension	*F1 (in)*	2.00	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	10.281	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	308.4	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	308.4	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.61	**OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25	
Number of oversized bolts in Zone M (see step 5)	n_m	2	
Number of oversized bolts on top plate (see step 5)	n_b	5	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.25	
Bolt spacing (see schedule)	*s* (in)	3.25	
Fuse plate dimension	*F1 (in)*	2.00	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	21.875	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	19.141	
Number of oversized bolts on top plate (see step 5)	n_b	5	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	13.672	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	533.2	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	399.9	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.47	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	189.4	
Beam flange width	b_{fb} (in)	9.07	
Weld 8 size	D_{W8} (1/16 in)	9	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.56	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	10.63	
Weld 9 size	D_{W9} (1/16 in)	9	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	5.75	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	11.6	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	12.5	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.92	**OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
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)	75.7	

Yielding check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	1.375	
Top plate thickness (see step 2s)	*T4* (in)	0.875	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	2.844	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	142.2	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	128.0	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.59	**OK**

Rupture check

Top plate dimension	*P4* (in)	1.875	
Top plate dimension	*P5* (in)	1.375	
Top plate thickness (see step 2)	*T4* (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)(T4)*	A_e (in^2)	1.695	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	110.2	
Factored tensile rupture capacity, *0.75P_n*	ϕP_n (kips)	82.6	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.92	**OK**

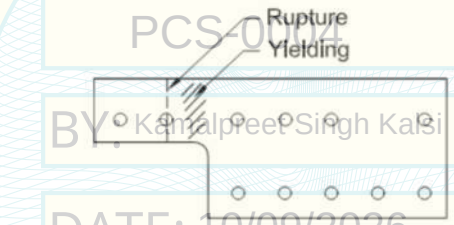


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-00024
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

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)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Top plate thickness (see step 2)	*T4* (in)	0.875	
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)	17.00	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	8.016	
Dimension, *([P8] - m) / 2*	*e* (in)	4.492	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	0.0	
Provided P2 dimension	*P2* (in)	2.75	**OK**

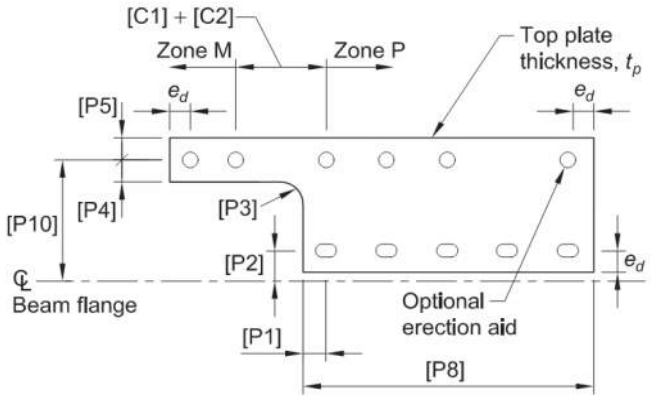


Fig. 15.6. Top plate detail.

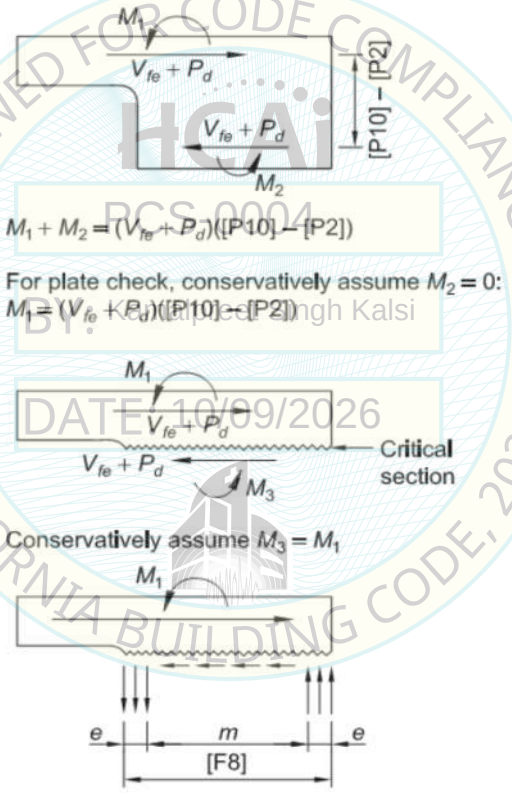


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS 0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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)	2.25	**OK**
Fuse plate thickness (see step 2)	$T2$ (in)	0.875	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	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)	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)	$n_{b,\ Bot}$	5	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	3.2	
Fuse plate thickness (see step 2)	$T2$ (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, $2([F7]+[F8])T2$	A_g (in^2)	10.413	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	520.6	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	468.6	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.32	**OK**

Step 18. (continued)

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)	3.2	
Fuse plate thickness (see step 2)	$T2$ (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, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	8.116	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	P_n (kips)	633.0	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	474.8	
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.32	**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

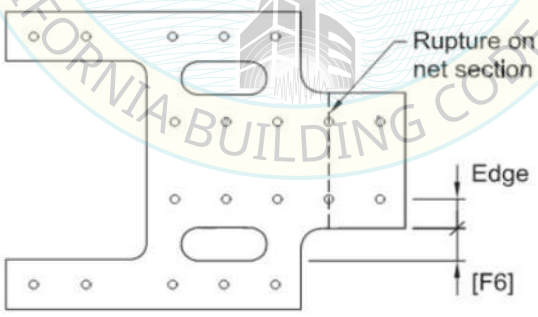


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaljeet Singh Kalsi
UPDATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	2.25	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	9535	
Fuse plate thickness (see step 2)	*T2* (in)	0.875	
Top plate thickness (see step 2)	*T4* (in)	0.875	
Beam depth	d_b (in)	24.3	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	63.4	
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_n	4	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	86.8	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	91.1	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	36.4	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}-R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	1.970	
Provided yielding region depth	*F2* (in)	1.875	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	98.4	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	189.4	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.9	**OK**

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	1.14	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

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)	$n_{b,\ Bot}$	5	
Number of bolts in the M region	$n_{m,\ Bot}$	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)	*T2* (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])(T2)*	A_g (in^2)	5.6875	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	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)	*T2* (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)(T2)*	A_e (in^2)	3.390625	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
RETAINED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

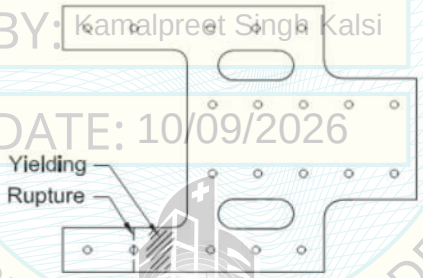
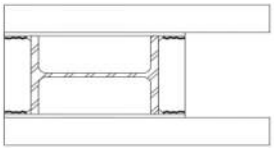


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

MISCELLANEOUS CALCULATIONS

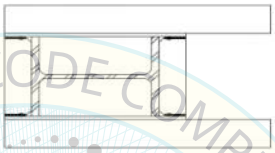
Bridge plate calculations

Bridge plate tension failure check



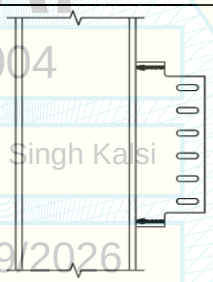
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	189.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	0	
External continuity plate dimension	*C5* (in)	2.75	
Column depth	d_c (in)	21.7	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	2	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}$ *T4([C3]-1")*	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**

Bridge plate to cover plate weld failure check



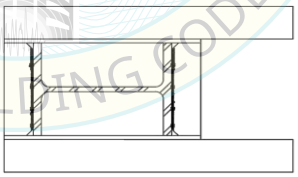
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16$ *([C3]-1")(2 sides)*	ϕR_n (kips)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

Shear tab to bridge plate weld failure check



Minimum 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)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

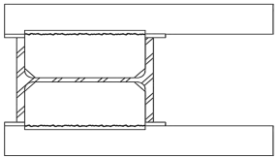
Bridge plate to column flange weld failure check



Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	6.875	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc}$ *- 2", 2[C3])*	l_{eff} (in)	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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

*These checks are N/A when bridge plates do not apply

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), $1.0(0.6)F_{ySP}\,t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	296.7	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**

*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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	$T1$ (in)	1.125	
Fuse plate thickness	T2 (in)	0.875	
Top plate thickness	$T4$ (in)	0.875	
External continuity plate dimension	$C5$ (in)	2.75	
Fuse plate dimension	$F2$ (in)	1.875	
Fuse plate dimension	$F3$ (in)	6.125	
Fuse plate dimension	$F4$ (in)	1.9	
Fuse plate dimension	$F6$ (in)	2.25	
Fuse plate dimension	$F8$ (in)	3.2	
Distance from beam centerline to bolt line	$P2$ (in)	2.75	
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	5.16E+04	
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	1.63E+04	
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	1.09E+04	
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	3.02E+04	
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	9.72E+03	
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	3.04E+04	
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	4.67E+06	
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	3.92E+06	
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.84	**OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

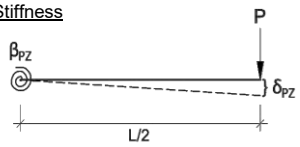
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

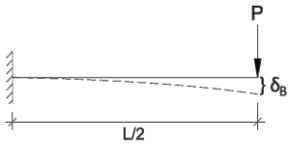


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (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)	2.0
Cover plate thickness	$T1$ (in)	1.125
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	1721.5
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)	2.77E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.44E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.79E+07
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)	2.61E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.94 **OK**

*This check is N/A when connection stiffness is applied in the model analysis

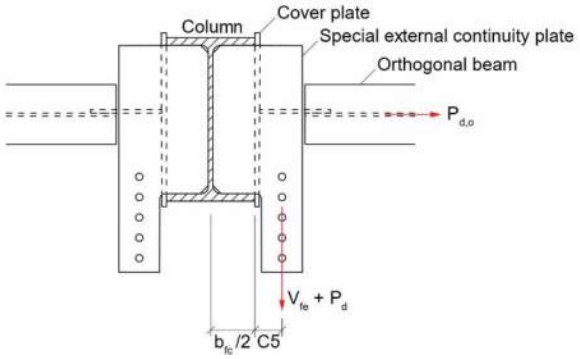
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

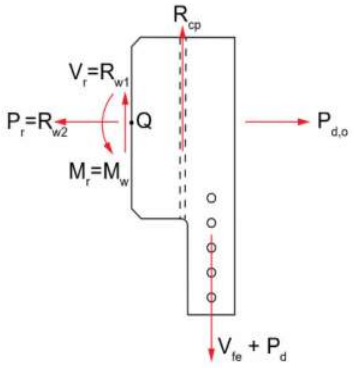
Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	378.7
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	0.0
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
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
External continuity plate thickness	$T5$ (in)	0.875
External continuity plate gage dimension	$C5$ (in.)	2.75
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	85.59	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	17.31	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
	Special detailing not required		

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	1.46	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	77.2	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	16.43	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply	Special detailing not required		





Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Ikampal Singh Kalsi
DATE: 10/09/2026

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size			
Orthogonal beam depth	$d_{b,o}$ (in)		
In plane moment frame beam 1		W24X94	
In plane moment frame beam 2		W24X94	
Max in-plane moment frame beam depth	d_b (in)	24.3	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0	
New continuity plate required for deeper cantilever beam		N/A	
New continuity plate width past column flange	C4 (in)	4.125	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A	**N/A**

This check is N/A when OOP forces do not apply

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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.)	44	
Beam depth	d_b (in.)	23.9	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	76.0	**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}	6.6	**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.)	44	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	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), Factor based on code and member type, $F(E/(R_{yc}F_{yc}))^{0.5}$	$\lambda_{hd,flange}$	7.35	
	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	6.45	**OK**

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

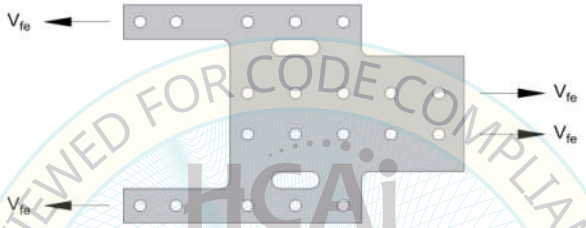
Column plastic section modulus	Z_c (in^3)	307	
Column design axial load	P_u (kips)	53.5	
Column area	A_{gc} (in^2)	35.9	
Story Height	H (in)	216.0	
Column depth	d_c (in)	21.7	
Beam depth	d_b (in)	23.9	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c*(F_{yc}-P_u/A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	3.67E+04	
Beam plastic section modulus	Z_b (in^3)	200	
Minimum 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)	2.14E+04	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.71	**OK**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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**
Fuse plate thickness	$T2$ (in)	1	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	0.75	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	0.75	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	23.9	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	201.8	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	141.3	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	343.1	



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	343.1	
Beam area	A_{gb} (in^2)	22.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	0.0	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	343.1	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav-ortho}$ (kips)	35	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	0.75	
External continuity plate thickness	$T5$ (in)	0.75	
Beam depth	d_b (in)	23.9	
Column depth	d_c (in)	21.7	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav-ortho}/2$	$R_{u,vert}$ (kips)	409.2	
Cover plate shear yielding limit state, horizontal shear (AISC 360 Chapter J)			
Cover plate thickness	$T1$ (in)	0.875	
Cover plate overhange dimension	$C3$ (in)	2	
Column depth	d_c (in)	21.7	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	25.7	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	674.6	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	674.6	
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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag conditic	h_{cp} (in)	29.90	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	784.9	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	784.9	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.52	**OK**

Step 3. (continued)

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, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	1.86	**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 5a. Determine the number of bolts for each bolt line on the top plates and check limit states

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	201.8	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
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)	49.5	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	4.1	
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 (in)	1.375	
Bolt spacing (see schedule)	s (in)	3.25	
Top plate thickness	$T4$ (in)	0.75	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	2.875	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.8461	
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)	201.8	
Beam flange thickness	t_{fb} (in)	0.68	
Top plate thickness	$T4$ (in)	0.75	
Governing material thickness, $\min(t_{fb}, T4)$	t_{min} (in)	0.7	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	530.4	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	424.3	
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, $e - d_O / 2$	l_{c2} (in)	0.8	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	39.8	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	464.1	
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	0.82	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	1451	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	0.75	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	29.3	
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*	0.32	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	201.8	
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, $P_{d,total}/2$	P_d (kips)	0.0	
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, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	3.1	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	5	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.375	
Bolt spacing (see schedule)	s (in)	3.25	
Fuse plate thickness	$T2$ (in)	1	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	2.156	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	2.4	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	3	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	2	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	201.8	
Beam flange thickness	t_{fb} (in)	0.68	
Fuse 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), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	530.4	
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), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	424.3	
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, $e - d_O / 2$	l_{c2} (in)	0.8	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	39.8	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	464.1	
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*	0.82	**OK**

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kemalettin Cagatayalsi
DATE: 10/09/2026

Step 5b. (continued)

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, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	29.4	
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 on the bottom flange	$n_{b,Bot}$	5	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	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*	0.32	**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 (in)	1.375	
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), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	2.809	
Distance from alignment line to column	*C1* (in)	5.125	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	343.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	0	
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	
Fuse plate thickness	$T2$ (in)	1	
Top plate thickness	$T4$ (in)	0.75	
External continuity plate thickness	$T5$ (in)	0.75	
Cover plate thickness	$T1$ (in)	0.875	
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2T5$ for cap plate and ortho. cant./drag condition	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)	0.6875	
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	28.53	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $T5 + T1 + [C6]$	l_{We1} (in)	4.625	
Weld 1 size	D_{W1} (1/16 in)	9	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
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 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
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)	1.91	
Vertical distance from center of rotation to geometric center	e_y (in)	0.00	

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.

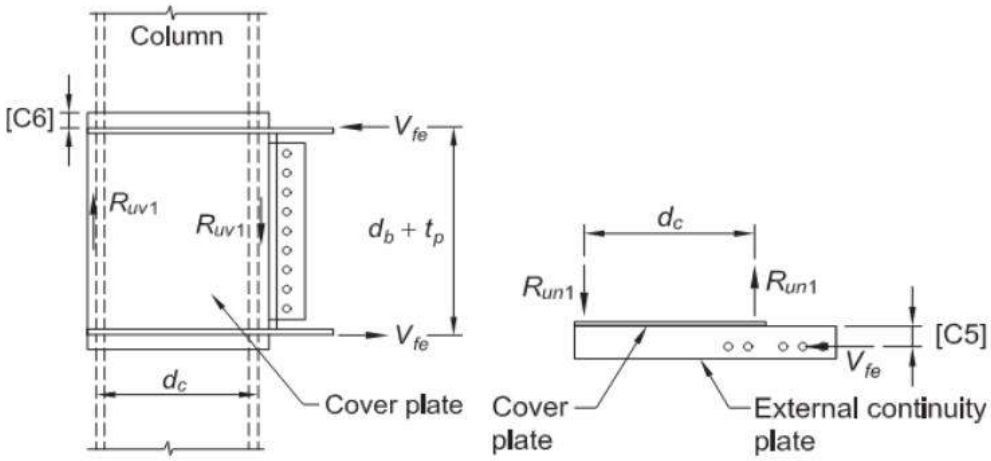
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	15085	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	16.59	13.06	57.39	216.8
W4TL	18.72	0.00	38.27	0.0
W1LB	18.92	12.23	47.61	231.5
W4BR	16.63	0.00	27.91	0.0
W1RC	8.95	7.60	1.83	68.0
W4TC	14.85	0.00	8.15	0.0
W1LC	12.76	8.18	1.28	104.4
W4BC	14.85	0.00	8.15	0.0
		13.06	57.39	

Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	13.1	
Orientation of Resultant Force	θ_r (degrees)	57.4	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.57	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	12.9	
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)	9.4	
Resultant Weld Demand at Critical Location $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	16.0	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.27	
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)	17.17	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.93	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/06/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	343.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0	
Column depth	d_c (in)	21.7	
Column flange thickness	t_{fc} (in)	0.96	
Cover plate overhang dimension	*C3* (in)	2	
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} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	3.40	
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)	13.3	
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)	13.0	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	18.6	
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*	0.95	**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)*	343.1	
External continiuty plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	943.5	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	343.1	
External continuity plate thickness (see step 2)	*T5* (in)	0.750	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	578.3	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}\,T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	5572.9	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.76	**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)*	343.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
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)	87.2	
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)	1133.0	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,t}$	R_{nwt} (kips)	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)	1812.6	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	1359.5	
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	*D/C*	0.25	**OK**

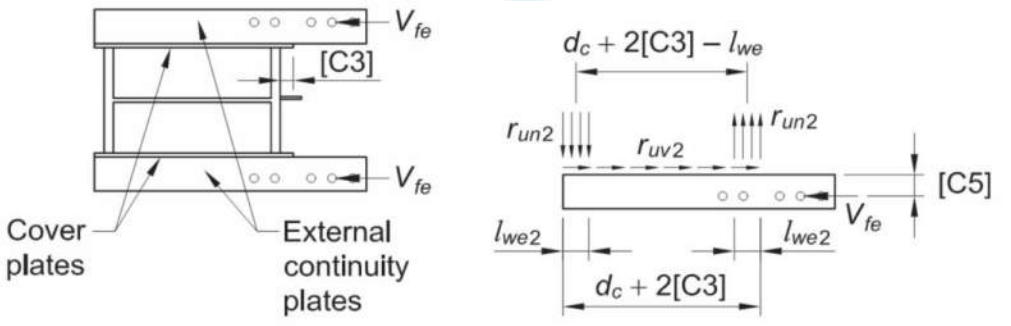


Fig. C-15.5. Weld demands for external continuity plate welds.

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
DESIGNED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	201.8	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	3	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	121.1	
External continuity plate thickness (see step 2)	*T5* (in)	0.75	
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, *T5([C4]-($d_{hole,std}$+1/16))*	A_n (in^2)	2.484	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	161.5	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	121.1	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	1.00	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	4	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	161.5	
External continuity plate dimension	*C2 (in)*	1.875	
Cover plate overhang dimension	*C3* (in)	2	
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)	*T5* (in)	0.75	
External continuity plate width	*C4* (in)	4.5	
Diagonal bonus (AISC 360 B4.3b), *[C9]^2 (T5)/4[C5]*	*Bonus* (in)	1.024	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Area of external continuity plate cross section with bonus, T5*([C4]-$d_{hole,std}$-1/16) + Bonus*	A_n (in^2)	3.508	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	228.0	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	171.0	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.94	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	5	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	201.8	
Bolt spacing (see schedule)	*s* (in)	3.25	
External continuity plate dimension	*C2 (in)*	1.875	
Cover plate overhang dimension	*C3* (in)	2	
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)	*T5* (in)	0.75	
External continuity plate width	*C4* (in)	4.5	
Diagonal bonus (AISC 360 B4.3b), *[C7]^2 (T5)/(4[C5])*	*Bonus* (in)	3.461	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Area of external continuity plate cross section with bonus, *(T5)([C4]-$d_{hole,std}$-1/16) + Bonus*	A_n (in^2)	5.946	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	386.5	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	289.9	
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, $n_{rup1} + 1$	n_{rup}	4	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	161.5	
External continuity plate width	*C4* (in)	4.5	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	3.875	
External continuity plate thickness (see step 2)	*T5* (in)	0.75	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	3.375	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	2.906	
Net tension area, A_{gt} - *((T5)*($d_{hole,std}$+1/16))*	A_{nt} (in^2)	2.484	
Net shear area, A_{gv}	A_{nv} (in^2)	2.906	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	248.7	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	186.5	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.87	**OK**

APPROVED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column			
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	5	
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	201.8	
Bolt spacing (see schedule)	s (in)	3.25	
External continuity plate dimension	*C2 (in)*	1.875	
Cover plate overhang dimension	*C3* (in)	2	
External continuity plate width	*C4* (in)	4.5	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	7.125	
External continuity plate thickness (see step 2)	*T5* (in)	0.75	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.125	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	3.375	
Gross shear area, *T5[C7]*	A_{gv} (in^2)	5.344	
Net tension area, $A_{gt}-((T4)(d_{hole}+1/16))$	A_{nt} (in^2)	2.484	
Net shear area, A_{gv}	A_{nv} (in^2)	5.344	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_n$	R_n (kips)	321.8	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	241.3	
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.84	**OK**

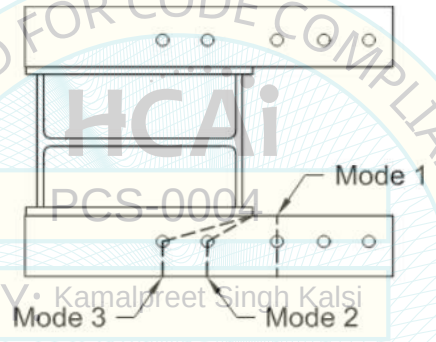


Fig. C-15.6. Tension rupture limit state for external continuity plate.

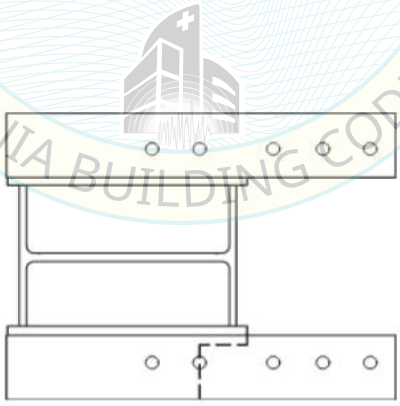


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum 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)	201.8	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	0.0	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	403.6	
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	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)	2.373	
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	19.550	
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)	12.283	
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)	580.8	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	435.6	
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)	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 provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	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 (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	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1}\ t_{min}\ F_{u,min}\ (n_n - 1)$	R_{nt1} (kips)	450.5	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.7	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2}\ t_{min}\ F_{u,min}$	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), $1.5 T_b \mu D_u h_f n_s$	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)	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)	338.9	

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	$T3$ (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 (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, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) T3$	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), $(0.75) 0.6 F_{uST} A_{nv}$	ϕR_{nv} (kips)	208.41	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75) F_{uST} A_e$	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)	338.86	
Shear tab gross area, $T3 l_{tab}$	A_{gv} (in^2)	9.50	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	45.13	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0) 0.6 F_{yST} A_{gv}$	ϕR_{nv} (kips)	285.00	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9) F_{yST} A_{gv}$	ϕR_{nn} (kips)	427.50	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9) F_{yST} Z_{tab}$	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	0.26	**OK**

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	19	
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)	3.5	
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)	5.125	
Size of weld in 1/16^{ths} of an inch	D_{w3}	6	
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)	338.9	
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)	15.2	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	15.6	
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)	16.7	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	*D/C*	0.93	**OK**

This check is N/A when bridge plates do not apply

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 (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), *$\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$*	$S1_{req}$ (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)

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	201.8	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	201.8	

Yielding check			
Top plate thickness (see step 2)	*T4* (in)	0.75	
Top plate dimension (see schedule)	*P8* (in)	16.5	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	12.375	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), *$0.6F_{yTop}A_g$*	R_n (kips)	371.3	
Factored shear yielding capacity, *$1.0R_n$*	ϕR_n (kips)	371.3	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.54	**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, *$A_g - (T4 l_s n_b)$*	A_n (in^2)	7.453	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), *$0.6A_n F_{uTop}$*	R_n (kips)	290.7	
Factored shear rupture capacity, *$0.75R_n$*	ϕR_n (kips)	218.0	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.93	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

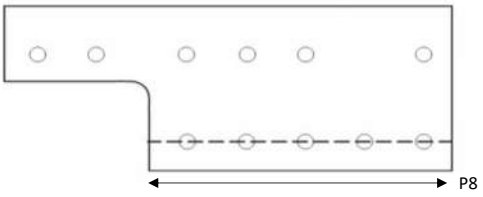


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	201.8	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	201.8	

Yielding check (LS 1)

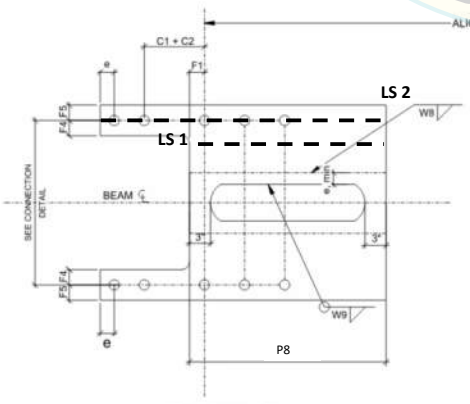
Top plate thickness (see step 2)	*T4* (in)	0.75	
Top plate dimension (see schedule)	*P8* (in)	11.75	
Fuse plate dimension	*F1 (in)*	1.75	
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	8.813	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	264.4	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	264.4	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.76	**OK**

Rupture check (LS 2)

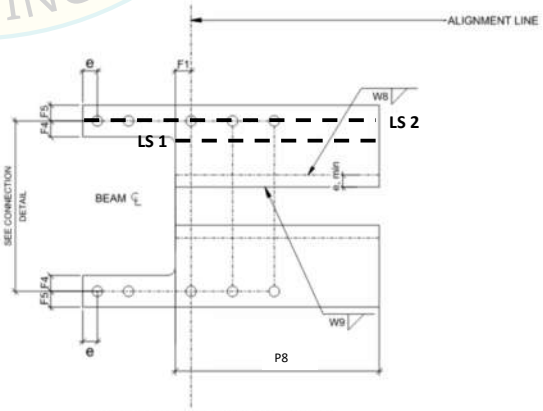
Bolt diameter (see step 4)	d_{bolt} (in)	1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.25	
Number of oversized bolts in Zone M (see step 5)	n_m	2	
Number of oversized bolts on top plate (see step 5)	n_b	5	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.25	
Bolt spacing (see schedule)	*s* (in)	3.25	
Fuse plate dimension	*F1 (in)*	1.75	
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	21.5	
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	16.125	
Number of oversized bolts on top plate (see step 5)	n_b	5	
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	11.438	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	446.1	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	334.5	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.60	**OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	201.8	
Beam flange width	b_{fb} (in)	8.99	
Weld 8 size	D_{W8} (1/16 in)	10	
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.63	
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	10.50	
Weld 9 size	D_{W9} (1/16 in)	10	
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	5.75	
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	12.4	
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	13.9	
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.89	**OK**



TOP FLANGE



TOP FLANGE ALTERNATE

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	201.8	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
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)	80.7	

Yielding check

Top plate dimension	*P4* (in)	1.625	
Top plate dimension	*P5* (in)	2	
Top plate thickness (see step 2s)	*T4* (in)	0.75	
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	2.719	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	135.9	
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	122.3	
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.66	**OK**

Rupture check

Top plate dimension	*P4* (in)	1.625	
Top plate dimension	*P5* (in)	2	
Top plate thickness (see step 2)	*T4* (in)	0.75	
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)(T4)*	A_e (in^2)	1.734	
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	112.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	84.6	
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.95	**OK**

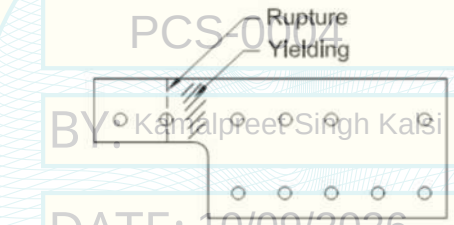


Fig. C-15.9. Tension rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025
HCAi
PCS-00024
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

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)	201.8	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	0.0	
Top plate thickness (see step 2)	*T4* (in)	0.75	
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	*P10* (in)	9.825	
Inside length of the top plate	*P8* (in)	16.50	
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop}\,T4)$	*m* (in)	9.966	
Dimension, *([P8] - m) / 2*	*e* (in)	3.267	
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), $[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)$	$P2_{req}$ (in)	2.6	
Provided P2 dimension	*P2* (in)	2.75	**OK**

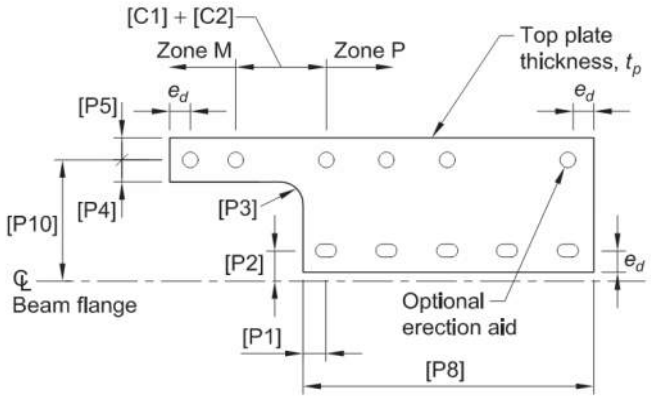


Fig. 15.6. Top plate detail.

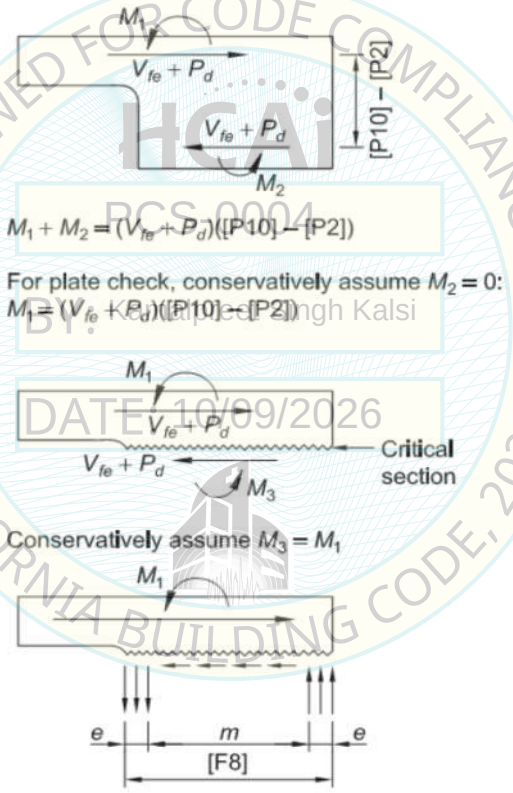


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS 0004
BY: Kulpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.11

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**
Fuse plate thickness (see step 2)	$T2$ (in)	1	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	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)	201.8	
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)	$n_{b,\ Bot}$	5	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	161.5	
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)	$T2$ (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, $2([F7]+[F8])T2$	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	0.30	**OK**

Step 18. (continued)

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)	$T2$ (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, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	9.525	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [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	0.29	**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

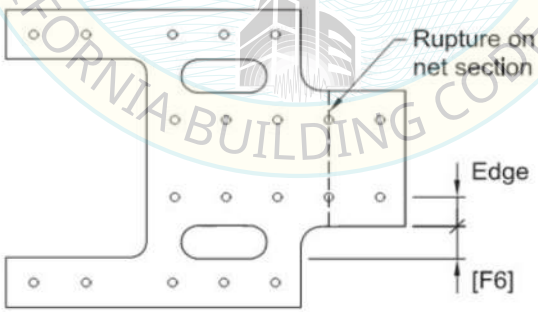


Fig. C-15.11. Limit states related to dimension [F6].

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamaldeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	2.125	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	10000	
Fuse plate thickness (see step 2)	*T2* (in)	1	
Top plate thickness (see step 2)	*T4* (in)	0.75	
Beam depth	d_b (in)	23.9	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	66.1	
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_n	4	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	86.8	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	91.1	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	36.4	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	*F2* $_{max}$ (in)	1.843	
Provided yielding region depth	*F2* (in)	1.750	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	105.0	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	201.8	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.9	**OK**

Fuse plate yielding region geometry			
Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	*(F6/F2)* $_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	1.15	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	201.8	
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)	$n_{b,\ Bot}$	5	
Number of bolts in the M region	$n_{m,\ Bot}$	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)	161.5	

Yielding check

Fuse plate dimension	*F4* (in)	1.625	
Fuse plate dimension	*F5* (in)	1.375	
Fuse plate thickness (see step 2)	*T2* (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])(T2)*	A_g (in^2)	6	
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	R_n (kips)	300.00	
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	270.00	
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.60	**OK**

Rupture check

Fuse plate dimension	*F4* (in)	1.625	
Fuse plate dimension	*F5* (in)	1.375	
Fuse plate thickness (see step 2)	*T2* (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)(T2)*	A_e (in^2)	3.375	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	R_n (kips)	263.3	
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	197.4	
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.82	**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

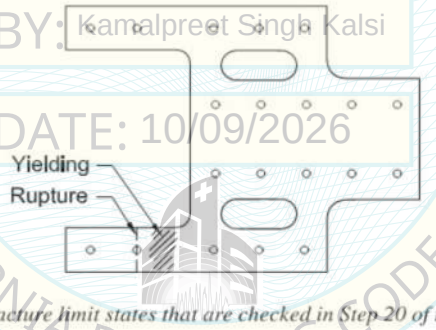


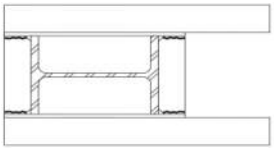
Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

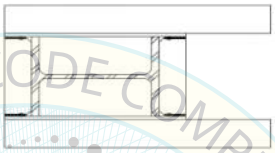
Bridge plate calculations

Bridge plate tension failure check



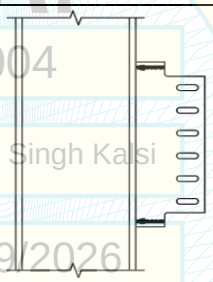
Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	201.8	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	0	
External continuity plate dimension	*C5* (in)	2.75	
Column depth	d_c (in)	21.7	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	2	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}$ *T4([C3]-1")*	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**

Bridge plate to cover plate weld failure check



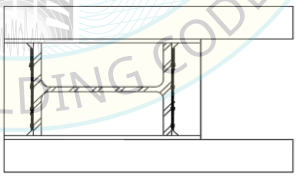
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
Weld 5 factored capacity (AISC 360 Eq. J2-4), $(0.75)(0.6)(F_{EXX})(1.5)(0.707)D_{w5}/16$ *([C3]-1")(2 sides)*	ϕR_n (kips)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

Shear tab to bridge plate weld failure check



Minimum 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, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	66.1	
Distance from alignment line to column	*C1* (in)	5.125	
Weld 7 demand, $V_u C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

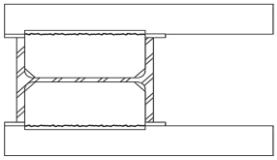
Bridge plate to column flange weld failure check



Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	5.125	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	2	
Column flange width	b_{fc} (in)	12.4	
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

*These checks are N/A when bridge plates do not apply

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)	0	
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)	N/A	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.000	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	N/A	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**

*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Amalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

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)	2100	
Column flange width	b_{fc} (in)	12.4	
Beam depth	d_b (in)	23.9	
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	$T1$ (in)	0.875	
Fuse plate thickness	T2 (in)	1	
Top plate thickness	$T4$ (in)	0.75	
External continuity plate dimension	$C5$ (in)	2.75	
Fuse plate dimension	$F2$ (in)	1.75	
Fuse plate dimension	$F3$ (in)	6.125	
Fuse plate dimension	$F4$ (in)	1.6	
Fuse plate dimension	$F6$ (in)	2.125	
Fuse plate dimension	$F8$ (in)	3.325	
Distance from beam centerline to bolt line	$P2$ (in)	2.75	
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	6.63E+04	
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	1.84E+04	
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	1.20E+04	
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	3.24E+04	
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	1.09E+04	
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	2.70E+04	
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	4.77E+06	
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	3.04E+06	
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.64	**OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

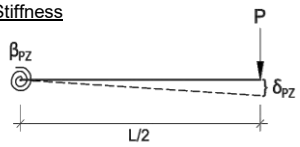
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

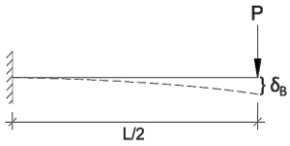


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	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)	2.0
Cover plate thickness	$T1$ (in)	0.875
Bay width	B (in)	360.0
Story height	H (in)	216.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b (d_c + 2[C3])T1$	V_p (in^3)	1386.1
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)	2.23E+07
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.43E+05
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	2.24E+07
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)	2.03E+07
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	0.91 **OK**

*This check is N/A when connection stiffness is applied in the model analysis

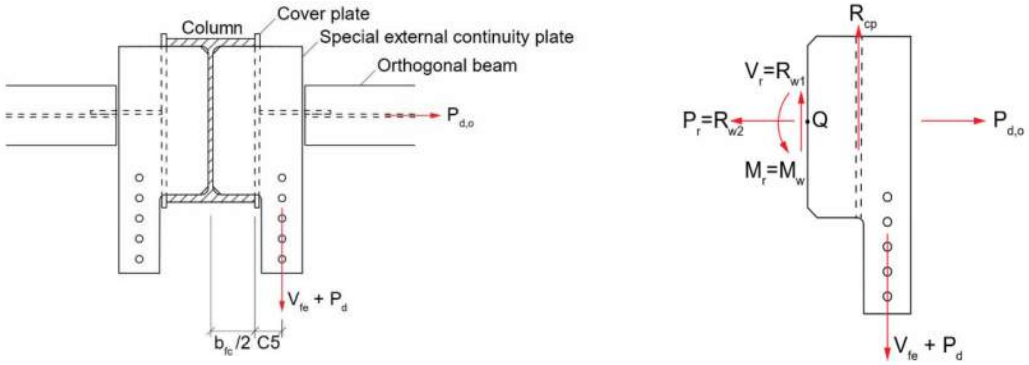
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	343.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	0.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	0.0
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
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
External continuity plate thickness	$T5$ (in)	0.750
External continuity plate gage dimension	$C5$ (in.)	2.75
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	73.36	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	14.84	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
		Special detailing not required	

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	1.46	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	66.1	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	14.09	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply		Special detailing not required	



Figures 13 and 14 PCS-0004

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth			
Orthogonal beam size			
Orthogonal beam depth	$d_{b,o}$ (in)		
In plane moment frame beam 1		W24X76	
In plane moment frame beam 2		W24X76	
Max in-plane moment frame beam depth	d_b (in)	23.9	
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0	
New continuity plate required for deeper cantilever beam		N/A	
New continuity plate width past column flange	C4 (in)	4.5	
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A	
Simply supported plate buckling coefficient	k	0.43	
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A	
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A	
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A	**N/A**

This check is N/A when OOP forces do not apply

5 NEW CONNECTIONS FOR PCS V4.0

DF101 Detailed Connection Design (W36X529 Beam)	5.1
DF102 Detailed Connection Design (W44X335 Beam)	5.2
DF103 Detailed Connection Design (BUW44X17X2.875 Beam)	5.3
DF201 Detailed Connection Design (BUW44X17X2.875 Beam)	5.4

REVIEWED FOR CODE COMPLIANCE
PCS 2024
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

Connection ID DF101

W40X431 Column and W36X529 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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS#004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Beam limitation checks			
Beam size		W36X529	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	44	
Beam depth	d_b (in.)	39.8	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	529.0	**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}	19.9	**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}	3.0	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		W40X431	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	44	
Column depth	d_c (in.)	41.3	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	431	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	2.36	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	1000.0	
Gross area of the column section	A_g (in^2)	127	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	50.9	
Column web width-to-thickness ratio	λ_{web}	25.5	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1),	$\lambda_{hd,flange}$	7.35	
Factor based on code and member type, $F(E/(R_{yc}F_{yc}))^{0.5}$	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	3.44	**OK**

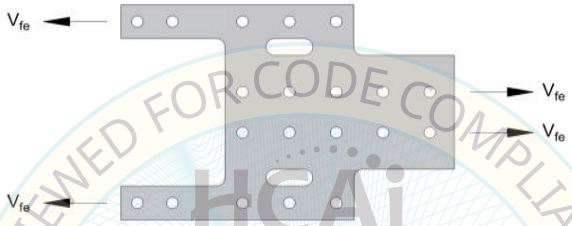
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	1960	
Column design axial load	P_u (kips)	1000.0	
Column area	A_{gc} (in^2)	127	
Story Height	H (in)	192.0	
Column depth	d_c (in)	41.3	
Beam depth	d_b (in)	39.8	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c*(F_{yc}-P_u/A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	2.53E+05	
Beam plastic section modulus	Z_b (in^3)	2330	
Minimum bay width	B (in)	420.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	116500	
Clear distance between column faces, $B-d_c$	L_h (in)	378.7	
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)	25.0	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	640.3	
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	13221	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	1.30E+05	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.95	**OK**

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

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

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)	2330	
Beam plastic bending moment , $F_{yb}Z_b$	M_p (k-in)	116500	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	116500	**OK**
Fuse plate thickness	$T2$ (in)	2	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	3	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	3	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	39.8	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	1377.1	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	1377.1	



Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1377.1	
Beam area	A_{gb} (in^2)	156.0	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	12.5	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	1389.6	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	25	
Fuse plate thickness	$T2$ (in)	2	
Top plate thickness	$T4$ (in)	3	
External continuity plate thickness	$T5$ (in)	3	
Beam depth	d_b (in)	39.8	
Column depth	d_c (in)	41.3	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	1429.3	

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

Cover plate thickness	$T1$ (in)	1.125	
Cover plate overhange dimension	$C3$ (in)	6.25	
Column depth	d_c (in)	41.3	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	53.8	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	1815.8	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	1815.8	
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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag cond	h_{cp} (in)	45.80	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	1545.8	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	1545.8	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.92	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 3. (continued)

Cover plate thickness check (AISC 341 E3.6e.2)			
Beam depth	d_b (in)	39.8	
Beam flange thickness	t_{fb} (in)	2.91	
Column flange thickness	t_{fc} (in)	2.36	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	33.98	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	36.58	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.784	
Thickness check, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	1.43	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.25	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Beam plastic section modulus	Z_b (in^3)	2330	
Beam flange thickness	t_{fb} (in)	2.91	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	4.365	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	322.0	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	2008.0	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	128150	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	143568	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.89	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1377.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	12.5	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	77.3	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	18.0	
Number of bolts per line (AISC 358 15.2(1))	n_b	18	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	3.75	
Top plate thickness	$T4$ (in)	3	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	4.904	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.4822	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	16	**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)	1389.6	
Beam flange thickness	t_{fb} (in)	2.91	
Top plate thickness	$T4$ (in)	3	
Governing material thickness, $\min(t_{fb}, T4)$	t_{min} (in)	2.9	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	10214.1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.2	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	8440.8	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	219.9	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	8660.7	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1855.5	
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1855.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	1391.6	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	1.00	**OK**

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/03/2026
CERTIFIED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	0	
Fuse plate thickness	$T2$ (in)	2	
Top plate thickness	$T4$ (in)	3	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	0.0	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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	18	
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)	622.4	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	529.0	
Wind slip demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.00	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1377.1	
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, $P_{d,total}/2$	P_d (kips)	0.0	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	103.1	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	13.4	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	18	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	3.75	
Fuse plate thickness	$T2$ (in)	2	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	7.356	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	4.8	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	16	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	2	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	1377.1	
Beam flange thickness	t_{fb} (in)	2.91	
Fuse plate thickness	$T2$ (in)	2	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	2.0	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	10214.1	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.2	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	8440.8	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	151.1	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	8591.9	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1855.5	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1855.5	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	1391.6	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.99	**OK**

REVIEWED FOR CODE COMPLIANCE
CAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 5b. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	0	
Bolt group demand from wind per line, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	0.0	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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 on the bottom flange	$n_{b,Bot}$	18	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	622.4	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	529.0	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.00	**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)	39.8	
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	*e* (in)	1.75	
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$, *1.0 otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	6.25	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	10.388	
Distance from alignment line to column	*C1* (in)	10.5	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1377.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	12.5	
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	25	
Column depth	d_c (in)	41.3	
Column flange thickness	t_{fc} (in)	2.36	
Beam depth	d_b (in)	39.8	
Fuse plate thickness	$T2$ (in)	2	
Top plate thickness	$T4$ (in)	3	
External continuity plate thickness	$T5$ (in)	3	
Cover plate thickness	$T1$ (in)	1.125	
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2T5$ for cap plate and ortho. cant./drag condit	h_{cp} (in)	45.8	
Gage dimension from the cover plate to the line of bolts on the external continuity plates	$C5$ (in)	5	
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	1.1875	
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	43.43	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $T5 + T1 + [C6]$	l_{We1} (in)	7.125	
Weld 1 size	D_{W1} (1/16 in)	17	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	16	
Weld 4 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	36.58	
Horizontal distance from center of rotation to geometric center	e_x (in)	0.23	
Vertical distance from center of rotation to geometric center	e_y (in)	0.10	

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.

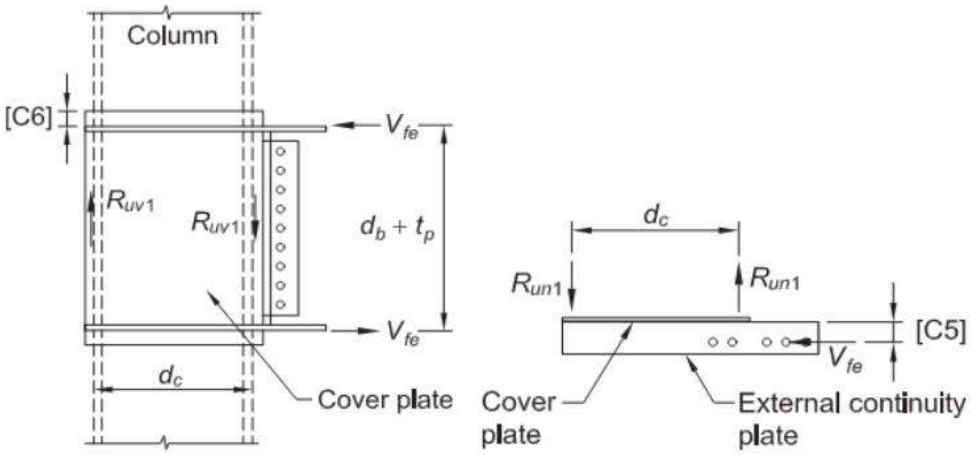
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	134572	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	29.56	34.87	46.31	1031.0
W4TL	29.11	26.42	38.59	769.1
W1LB	29.74	34.69	45.41	1031.5
W4BR	28.66	26.46	38.12	758.3
W1RC	20.43	23.86	1.50	487.4
W4TC	22.76	19.63	1.50	446.8
W1LC	20.89	23.96	1.47	500.5
W4BC	22.56	19.60	1.51	442.1
		34.87	46.31	

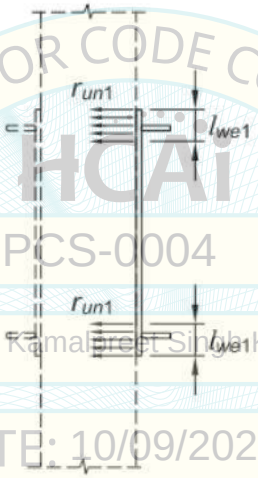
Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	34.9	
Orientation of Resultant Force	θ_r (degrees)	46.3	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.87	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	17.5	
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)	17.5	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.37	
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)	30.11	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.58	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/03/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

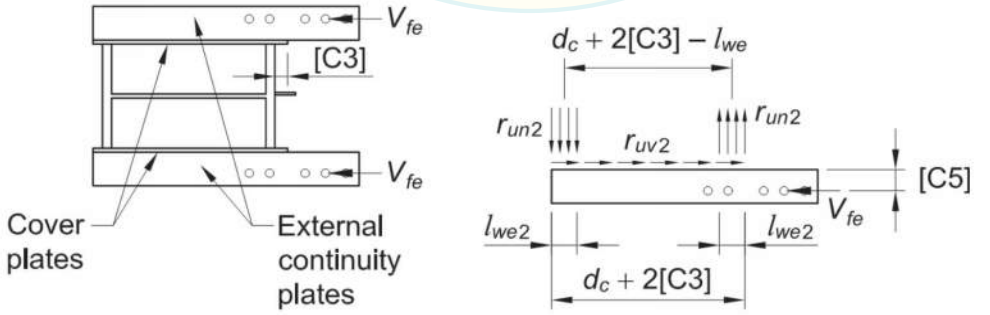
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	1377.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	12.5	
Column depth	d_c (in)	41.3	
Column flange thickness	t_{fc} (in)	2.36	
Cover plate overhang dimension	*C3* (in)	6.25	
External continuity plate dimension	*C5* (in)	5	
Weld 2 size	D_{W2} (1/16 in)	16	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	1.00	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + T1 + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	8.74	
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)	25.8	
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)	18.5	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	31.8	
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)	44.5	
External continuity plate-to-cover plate weld tearing check, $r_u / \phi r_n$	*D/C*	0.71	**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)*	1377.1	
External continiuty plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	6947.8	
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	1389.6	
External continuity plate thickness (see step 2)	*T5* (in)	3.000	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c + 2[C3])T5$	ϕR_n (kips)	4842.0	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}T5((d_c + 2*[C3])/2)^2$	ϕM_n (kips)	97687.4	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.36	**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)*	1377.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	12.5	
Column fillet dimension	*k* (in)	3.5	
Column fillet dimension	*k1* (in)	1.9	
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3]-2*l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	172.0	
Column flange width	b_{fc} (in)	16.2	
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	57.3	
Weld 2 size	D_{W2} (1/16 in)	16	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	5108.6	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	1701.5	
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)	6894.5	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	5170.9	
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	*D/C*	0.27	**OK**



(a) Applied loads *(b) Distributed shear and normal demands*

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

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1377.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	12.5	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	16	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	1235.2	
External continuity plate thickness (see step 2)	*T5* (in)	3	
External continuity plate width	*C4* (in)	10	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Net area of external continuity plate cross section, $T5([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	25.688	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	1669.7	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	1252.3	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.99	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	17	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	1312.4	
External continuity plate dimension	*C2 (in)*	4	
Cover plate overhang dimension	*C3* (in)	6.25	
External continuity plate dimension	*C5* (in)	5	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	10.25	
External continuity plate thickness (see step 2)	*T5* (in)	3	
External continuity plate width	*C4* (in)	10	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(T5)/4[C5]$	*Bonus* (in)	15.759	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Area of external continuity plate cross section with bonus, $T5([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	41.447	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	2694.0	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	2020.5	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.65	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	18	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	1389.6	
Bolt spacing (see schedule)	*s* (in)	3.75	
External continuity plate dimension	*C2 (in)*	4	
Cover plate overhang dimension	*C3* (in)	6.25	
External continuity plate dimension	*C5* (in)	5	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	14	
External continuity plate thickness (see step 2)	*T5* (in)	3	
External continuity plate width	*C4* (in)	10	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(T5)/(4[C5])$	*Bonus* (in)	29.400	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Area of external continuity plate cross section with bonus, $(T5)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	55.088	
Nominal tensile rupture capacity, A_nF_{uECP}	P_n (kips)	3580.7	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	2685.5	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.52	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	17	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	1312.4	
External continuity plate width	*C4* (in)	10	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	10.25	
External continuity plate thickness (see step 2)	*T5* (in)	3	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	30.000	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	30.750	
Net tension area, $A_{gt} - ((T5)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	25.688	
Net shear area, A_{gv}	A_{nv} (in^2)	30.750	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	2592.2	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	1944.1	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.68	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Ramandeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column		
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	18
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	1389.6
Bolt spacing (see schedule)	s (in)	3.75
External continuity plate dimension	*C2 (in)*	4
Cover plate overhang dimension	*C3* (in)	6.25
External continuity plate width	*C4* (in)	10
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	14
External continuity plate thickness (see step 2)	*T5* (in)	3
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Gross tension area, *T5[C4]*	A_{gt} (in^2)	30.000
Gross shear area, *T5[C7]*	A_{gv} (in^2)	42.000
Net tension area, A_{gt}*-((T4)(d_{hole}+1/16))*	A_{nt} (in^2)	25.688
Net shear area, A_{gv}	A_{nv} (in^2)	42.000
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	2929.7
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	2197.3
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.63 **OK**

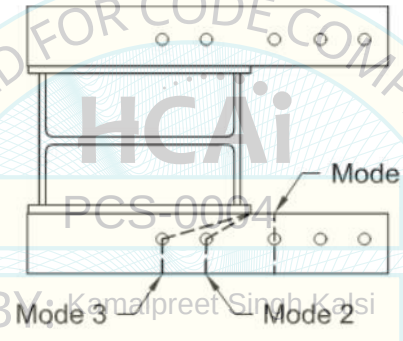


Fig. C-15.6. Tension rupture limit state for external continuity plate.

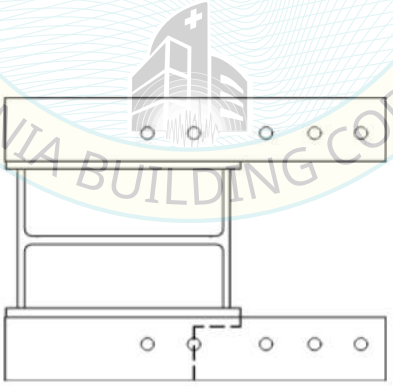


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum bay width	B (in)	420.0	
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	25.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	116500	
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	640.3	
Beam depth	d_b (in)	39.8	
Beam web thickness	t_{wb} (in)	1.61	
Beam web area, $d_b t_{wb}$	A_w (in)	64.078	
Beam web width-to-thickness ratio, h / t_w	l_{web}	19.9	
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)	1922.3	
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.33	**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)	1377.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	12.5	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	2779.1	
Beam flange thickness	t_{fb} (in)	2.91	
Beam flange width	b_{fb} (in)	17.2	
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Number of holes along each bolt line (see step 5)	n_b	18	
Length of connection on beam	$B4$ (in)	65.5	
Distance from beam centerline to bolt line	$P2$ (in)	6.9375	
Gross tension area, $(b_{fb} -2[P2])t_{fb}$	A_{gt} (in^2)	9.676	
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	381.210	
Net tension area, $A_{gt} -(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	5.493	
Net shear area, $A_{gv} -(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	234.801	
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)	9514.2	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	7135.7	
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.39	**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)	640.3	
Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	1.227	
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	103.1	
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	77.3	
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	8	
Provided number of shear tab bolts	n_n	9	**OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	31.375	
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - *1in*	$l_{tab,req}$ (in)	30	
Length of shear tab (see drawings), T_{beam} - *1in*	l_{tab} (in)	30.375	**OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	1.61	
Shear tab provided thickness	$T3$ (in)	1.375	
Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	1.4	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}t_{min}F_{u,min}n_n$	R_{nb} (kips)	2413.1	
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Edge distance on shear tab (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	3.391	
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)	2.0	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY • Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{u,min}\ (n_n\ -1)$	R_{nt1} (kips)	1729.4	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{u,min}$	R_{nt2} (kips)	100.5	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	1830.0	
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	927.8	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	927.8	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	695.8	
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.92	**OK**

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

Minimum fastener tension (AISC 360 Table J3.1)	T_b	102	
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)	51.9	
Provided number of shear tab bolts (see step 12)	n_n	9	
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	466.8	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	640.3	
Distance from alignment line to column	$C1$ (in)	10.5	
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)	6722.8	

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Slotted bolt hole width (standard slot width)	w_b (in)	1.375	
Shear tab provided thickness	$T3$ (in)	1.375	
Shear tab length (see step 12)	l_{tab} (in)	30	
Beam T dimension limit	T_{beam} (in)	31.375	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	3.3125	
Gross area of shear tab, $T3l_{tab}$	A_{gv} (in^2)	41.77	
Shear tab net area, $A_{gv} - n_n(w_b + 1/16)T3$	A_{nv} (in^2)	23.98	
Effective net area, $A_e = A_{nv}$	A_e (in^2)	23.98	
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{uST}A_{nv}$	ϕR_{nv} (kips)	701.31	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{uST}A_e$	ϕR_{nn} (kips)	1168.86	
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	0.85	**OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	640.26	
Required axial strength of the shear tab (see above)	P_u (kips)	466.80	
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)	6722.76	
Shear tab gross area, $T3l_{tab}$	A_{gv} (in^2)	41.77	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	317.16	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yST}A_{gv}$	ϕR_{nv} (kips)	1252.97	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yST}A_{gv}$	ϕR_{nn} (kips)	1879.45	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_{yST}Z_{tab}$	ϕM_n (kips)	14272.10	
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	0.60	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	30.375	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	640.3	
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	21.1	
Number of slotted bolt holes in shear tab	n_{vb}	9	
Slotted bolt hole spacing	s_{slot} (in)	3.3125	
Distance from line of bolts to shear tab weld	$C1$ (in)	10.5	
Size of weld in 1/16^{ths} of an inch	D_{W3}	23	
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)	6722.8	
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)	58.2	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	61.9	
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)	64.0	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.97	**OK**

This check is N/A when bridge plates do not apply

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e (in)	1.75	
Beam depth	d_b (in)	39.8	
Length of the shear tab (see step 12)	l_{tab} (in)	30.375	
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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (in)	2.62525	
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	2.75	**OK**
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	2.75	

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

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1377.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	12.5	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1389.6	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	3	
Top plate dimension (see schedule)	$P8$ (in)	67.75	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	203.250	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	6097.5	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	6097.5	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.23	**OK**

Rupture check			
Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.625	
Number of short slotted bolts on top plate (see step 5)	n_b	18	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	115.500	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	4504.5	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	3378.4	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.41	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

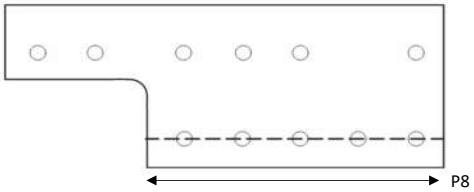


Fig. C-15.8. Shear rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1377.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	12.5
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1389.6

Yielding check (LS 1)

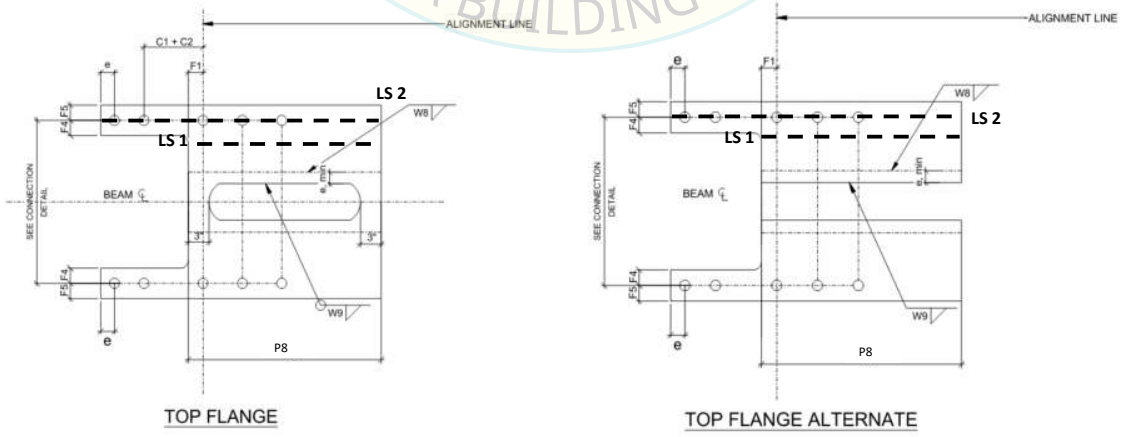
Top plate thickness (see step 2)	*T4* (in)	3
Top plate dimension (see schedule)	*P8* (in)	63.25
Fuse plate dimension	*F1 (in)*	2.00
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	189.750
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	5692.5
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	5692.5
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.24 **OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Number of oversized bolts in Zone M (see step 5)	n_m	2
Number of oversized bolts on top plate (see step 5)	n_b	18
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.625
Bolt spacing (see schedule)	*s* (in)	3.75
Fuse plate dimension	*F1 (in)*	2.00
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	81.125
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	243.375
Number of oversized bolts on top plate (see step 5)	n_b	18
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	159.000
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	6201.0
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	4650.8
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.30 **OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	1389.6
Beam flange width	b_{fb} (in)	17.2
Weld 8 size	D_{W8} (1/16 in)	9
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.56
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	62.13
Weld 9 size	D_{W9} (1/16 in)	9
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	57.25
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	11.6
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	12.5
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.93 **OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1377.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	12.5
Total number of bolts (see step 5)	n_b	18
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)	154.4

Yielding check

Top plate dimension	*P4* (in)	2.25
Top plate dimension	*P5* (in)	1.75
Top plate thickness (see step 2s)	*T4* (in)	3
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	12.000
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	600.0
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	540.0
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.29 **OK**

Rupture check

Top plate dimension	*P4* (in)	2.25
Top plate dimension	*P5* (in)	1.75
Top plate thickness (see step 2)	*T4* (in)	3
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Effective area of rupture, *([P4]+[P5]-d_O-1/16)(T4)*	A_e (in^2)	7.125
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	463.1
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	347.3
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.44 **OK**

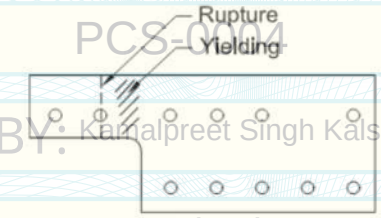


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

Entering and tightening check

Beam flange fillet dimension	k_1	2.0
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.5625
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3.5625
Provided P2 dimension	*P2* (in)	6.9375 **OK**

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1377.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	12.5
Top plate thickness (see step 2)	*T4* (in)	3
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	*P10* (in)	14.225
Inside length of the top plate	*P8* (in)	67.75
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop} T4)$	*m* (in)	17.155
Dimension, *([P8] - m) / 2*	*e* (in)	25.297
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), *[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)*	$P2_{req}$ (in)	0.0
Provided P2 dimension	*P2* (in)	6.9375 **OK**

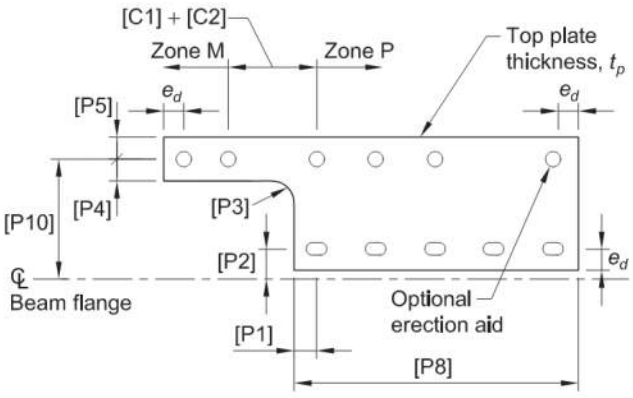


Fig. 15.6. Top plate detail.

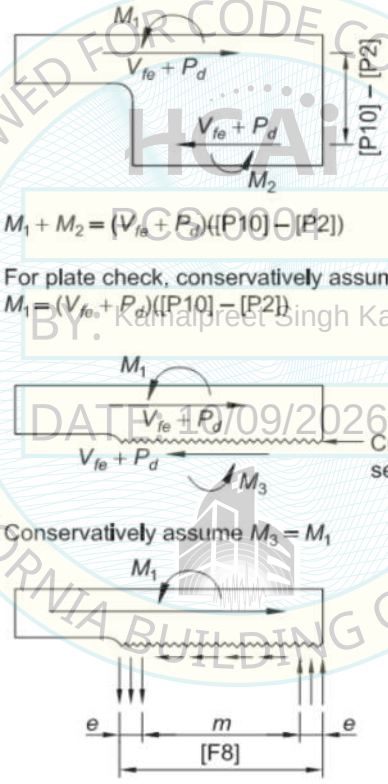


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAI
PCO-0001
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

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

Refer to AISC 358 Fig. C-15.11

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)	3.25	**OK**
Fuse plate thickness (see step 2)	$T2$ (in)	2	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	1.6	**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)	1377.1	
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)	$n_{b,\ Bot}$	18	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	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)	306.0	
Fuse plate dimension, centerline to gage line	$F7$ (in)	6.9375	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	1.7875	
Fuse plate thickness (see step 2)	$T2$ (in)	2	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
Gross area for yielding, $2([F7]+[F8])T2$	A_g (in^2)	34.900	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	1745.0	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	1570.5	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.19	**OK**

Step 18. (continued)

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

Fuse plate dimension	$F7$ (in)	6.9375	
Fuse plate dimension	$F8$ (in)	1.7875	
Fuse plate thickness (see step 2)	$T2$ (in)	2	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.563	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	28.400	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	P_n (kips)	2215.2	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	1661.4	
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.18	**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

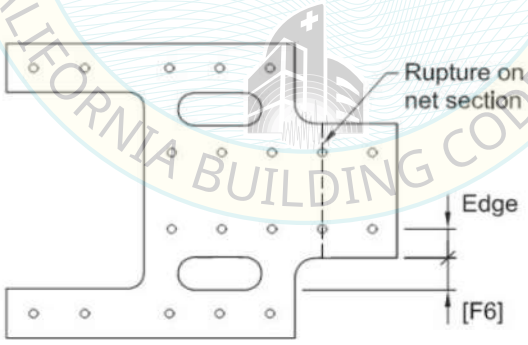


Fig. C-15.11. Limit states related to dimension [F6].

HCAi
PCS-0004
Reviewed By: Ramandeep Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	3.25	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	116500	
Fuse plate thickness (see step 2)	*T2* (in)	2	
Top plate thickness (see step 2)	*T4* (in)	3	
Beam depth	d_b (in)	39.8	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	640.3	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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_n	9	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	311.2	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	326.8	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	130.7	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}-R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	6.446	
Provided yielding region depth	*F2* (in)	6.375	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	765.0	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	1377.1	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.8	**OK**

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.50	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1377.1
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)	$n_{b,\ Bot}$	18
Number of bolts in the M region	$n_{m,\ Bot}$	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)	306.0

Yielding check

Fuse plate dimension	*F4* (in)	2.25
Fuse plate dimension	*F5* (in)	1.75
Fuse plate thickness (see step 2)	*T2* (in)	2
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Gross area of yielding, *2([F4]+[F5])(T2)*	A_g (in^2)	16
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	R_n (kips)	800.00
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	720.00
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.43 **OK**

Rupture check

Fuse plate dimension	*F4* (in)	2.25
Fuse plate dimension	*F5* (in)	1.75
Fuse plate thickness (see step 2)	*T2* (in)	2
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	9.5
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	R_n (kips)	741.0
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	555.8
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.55 **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

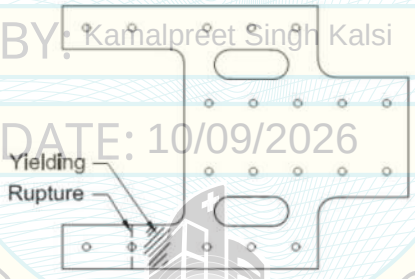


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

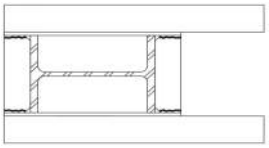
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

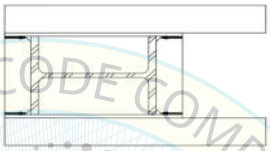
Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1377.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	12.5	
External continuity plate dimension	*C5* (in)	5	
Column depth	d_c (in)	41.3	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	6.25	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}\ T4([C3]-1")$	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**



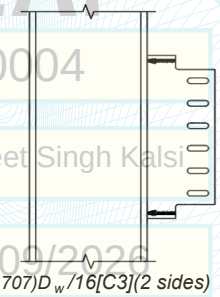
Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



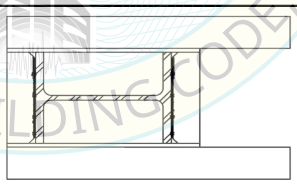
Shear tab to bridge plate weld failure check

Minimum bay width	*B* (in)	420.0	
Column depth	d_c (in)	41.3	
Beam depth	d_b (in)	39.8	
Shear force from gravity	$V_{gravity}$ (kips)	25.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	116500	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	640.3	
Distance from alignment line to column	*C1* (in)	10.5	
Weld 7 demand, $V_u\ C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	6.25	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



Bridge plate to column flange weld failure check

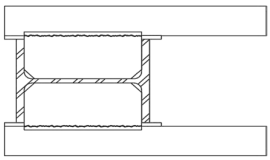
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	10.5	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	6.25	
Column flange width	b_{fc} (in)	16.2	
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	12.5	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



*These checks are N/A when bridge plates do not apply

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

Column depth	d_c (in)	41.3	
Column flange thickness	t_{fc} (in)	2.36	
Weld 4 size	D_{W4} (1/16 in)	16	
Weld 4 effective length, d_c - 2 t_{fc}	l_{eff} (in)	36.58	
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)	814.7	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	1.125	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	1234.6	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.7	**OK**



*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2025

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)	39600
Column flange width	b_{fc} (in)	16.2
Beam depth	d_b (in)	39.8
Bay width	B (in)	420.0
Number of bolts in zone M	M	2
Number of bolts in zone P	P	16
Bolt spacing	s (in)	3.75
Cover plate thickness	$T1$ (in)	1.125
Fuse plate thickness	T2 (in)	2
Top plate thickness	$T4$ (in)	3
External continuity plate dimension	$C5$ (in)	5
Fuse plate dimension	$F2$ (in)	6.375
Fuse plate dimension	$F3$ (in)	47.25
Fuse plate dimension	$F4$ (in)	2.3
Fuse plate dimension	$F6$ (in)	3.25
Fuse plate dimension	$F8$ (in)	1.7875
Distance from beam centerline to bolt line	$P2$ (in)	6.9375
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	5.97E+05
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	8.79E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	3.24E+05
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	7.52E+05
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	1.14E+05
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	5.71E+05
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	1.71E+08
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	4.92E+07
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.29 **OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

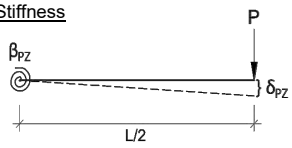
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

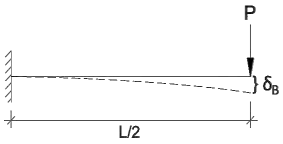


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	39.8
Beam flange thickness	t_{fb} (in)	2.91
Column depth	d_c (in)	41.3
Column flange width	b_{fc} (in)	16.2
Column web thickness	t_{wc} (in)	1.34
Column flange thickness	t_{fc} (in)	2.36
Cover plate overhang	$C3$ (in)	6.3
Cover plate thickness	$T1$ (in)	1.125
Bay width	B (in)	420.0
Story height	H (in)	192.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b(d_c + 2[C3])T1$	V_p (in^3)	7020.4
Column ratio, $(d_c - t_{fc}) / B$	α	0.093
Beam ratio, $(d_b - t_{fb}) / H$	β	0.192
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	1.54E+08
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.54E+06
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	N/A
Beam moment of inertia	I_{xb} (in^3)	39600
Required panel zone spring stiffness , $60EI_{xb} / B$ for one-sided, $120EI_{xb} / B$ for two-sided	K_{req} (k-in)	N/A
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	N/A **N/A**

*This check is N/A when connection stiffness is applied in the model analysis

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

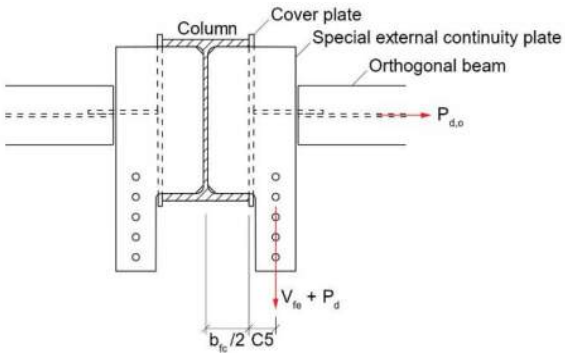
Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

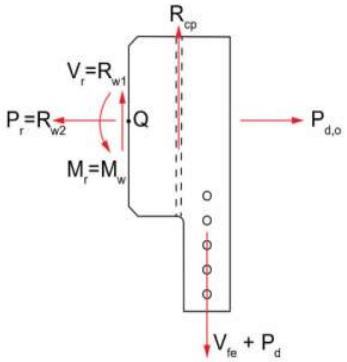
Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	1377.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	25.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	12.5
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
Column flange width	b_{fc} (in.)	16.2
Column depth	d_c (in.)	41.3
Column flange thickness	t_{fc} (in.)	2.36
Column web thickness	t_{wc} (in.)	1.34
External continuity plate thickness	$T5$ (in)	3.000
External continuity plate gage dimension	$C5$ (in.)	5.00
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	1003.57	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	109.74	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
	Special detailing not required		

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	3.54	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	878.3	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	102.66	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply	Special detailing not required		

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025





Figures 13 and 14 PCS-0004

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth		
Orthogonal beam size		
Orthogonal beam depth	$d_{b,o}$ (in)	
In plane moment frame beam 1		W36X529
In plane moment frame beam 2		NA
Max in-plane moment frame beam depth	d_b (in)	39.8
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0
New continuity plate required for deeper cantilever beam		N/A
New continuity plate width past column flange	C4 (in)	10
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A
Simply supported plate buckling coefficient	k	0.43
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A **N/A**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

Connection ID DF102

W40X431 Column and W44X335 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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS#004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Beam limitation checks			
Beam size		W44X335	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	44	
Beam depth	d_b (in.)	44	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	335.0	**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}	38	**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}	4.5	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		W40X431	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	44	
Column depth	d_c (in.)	41.3	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	431	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	2.36	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	1000.0	
Gross area of the column section	A_g (in^2)	127	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	50.9	
Column web width-to-thickness ratio	λ_{web}	25.5	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1),	$\lambda_{hd,flange}$	7.35	
Factor based on code and member type, $F(E/(R_{yc}F_{yc}))^{0.5}$	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	3.44	**OK**

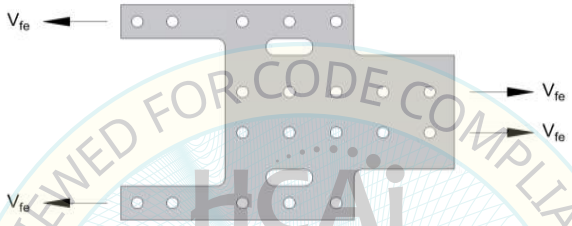
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	1960	
Column design axial load	P_u (kips)	1000.0	
Column area	A_{gc} (in^2)	127	
Story Height	H (in)	192.0	
Column depth	d_c (in)	41.3	
Beam depth	d_b (in)	44	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c*(F_{yc}-P_u/A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	2.61E+05	
Beam plastic section modulus	Z_b (in^3)	1620	
Minimum bay width	B (in)	420.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	81000	
Clear distance between column faces, $B-d_c$	L_h (in)	378.7	
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)	25.0	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	452.8	
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	9350	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	9.03E+04	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	2.89	**OK**

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

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEW FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

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)	1620	
Beam plastic bending moment , $F_{yb}Z_b$	M_p (k-in)	81000	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	81000	**OK**
Fuse plate thickness	$T2$ (in)	2	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	2	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	2	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	44	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	880.4	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	880.4	



Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	880.4	
Beam area	A_{gb} (in^2)	98.5	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	32.5	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	912.9	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	25	
Fuse plate thickness	$T2$ (in)	2	
Top plate thickness	$T4$ (in)	2	
External continuity plate thickness	$T5$ (in)	2	
Beam depth	d_b (in)	44	
Column depth	d_c (in)	41.3	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	1011.2	

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

Cover plate thickness	$T1$ (in)	0.875	
Cover plate overhange dimension	$C3$ (in)	1	
Column depth	d_c (in)	41.3	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	43.3	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	1136.6	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	1136.6	
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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag cond	h_{cp} (in)	50.00	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	1312.5	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	1312.5	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.77	**OK**

HCAI
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 3. (continued)

Cover plate thickness check (AISC 341 E3.6e.2)			
Beam depth	d_b (in)	44	
Beam flange thickness	t_{fb} (in)	1.77	
Column flange thickness	t_{fc} (in)	2.36	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	40.46	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	36.58	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.856	
Thickness check, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	1.02	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.25	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Beam plastic section modulus	Z_b (in^3)	1620	
Beam flange thickness	t_{fb} (in)	1.77	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	2.655	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	224.2	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	1395.8	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	89100	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	99797	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.89	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	880.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	32.5	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	77.3	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	11.8	
Number of bolts per line (AISC 358 15.2(1))	n_b	12	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	3.75	
Top plate thickness	$T4$ (in)	2	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	4.703	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.3750	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	6	**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	6	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	912.9	
Beam flange thickness	t_{fb} (in)	1.77	
Top plate thickness	$T4$ (in)	2	
Governing material thickness, min$(t_{fb}, T4)$	t_{min} (in)	1.8	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	4141.8	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.2	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	3322.1	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	133.7	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	3455.8	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1237.0	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1237.0	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	927.8	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.98	**OK**

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/03/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	0	
Fuse plate thickness	$T2$ (in)	2	
Top plate thickness	$T4$ (in)	2	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	0.0	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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	12	
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)	414.9	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	352.7	
Wind slip demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.00	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	880.4	
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, $P_{d,total}/2$	P_d (kips)	0.0	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	103.1	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	8.5	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	12	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	3.75	
Fuse plate thickness	$T2$ (in)	2	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	4.703	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.4	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	6	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	6	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	880.4	
Beam flange thickness	t_{fb} (in)	1.77	
Fuse plate thickness	$T2$ (in)	2	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	1.8	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	4141.8	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.2	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	3322.1	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	133.7	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	3455.8	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1237.0	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1237.0	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	927.8	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.95	**OK**

REVIEWED FOR CODE COMPLIANCE
CAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 5b. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	0	
Bolt group demand from wind per line, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	0.0	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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 on the bottom flange	$n_{b,Bot}$	12	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	414.9	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	352.7	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.00	**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)	44	
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	*e* (in)	1.75	
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$, *1.0 otherwise*	α	0	
Cover plate overhang dimension	*C3* (in)	1	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	4.39	
Distance from alignment line to column	*C1* (in)	5.25	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	880.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	32.5	
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	25	
Column depth	d_c (in)	41.3	
Column flange thickness	t_{fc} (in)	2.36	
Beam depth	d_b (in)	44	
Fuse plate thickness	$T2$ (in)	2	
Top plate thickness	$T4$ (in)	2	
External continuity plate thickness	$T5$ (in)	2	
Cover plate thickness	$T1$ (in)	0.875	
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2T5$ for cap plate and ortho. cant./drag condit	h_{cp} (in)	50	
Gage dimension from the cover plate to the line of bolts on the external continuity plates	$C5$ (in)	3.5	
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	0.7500	
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	48.50	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $T5 + T1 + [C6]$	l_{We1} (in)	5.875	
Weld 1 size	D_{W1} (1/16 in)	10	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	10	
Weld 4 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	36.58	
Horizontal distance from center of rotation to geometric center	e_x (in)	0.36	
Vertical distance from center of rotation to geometric center	e_y (in)	0.41	

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.

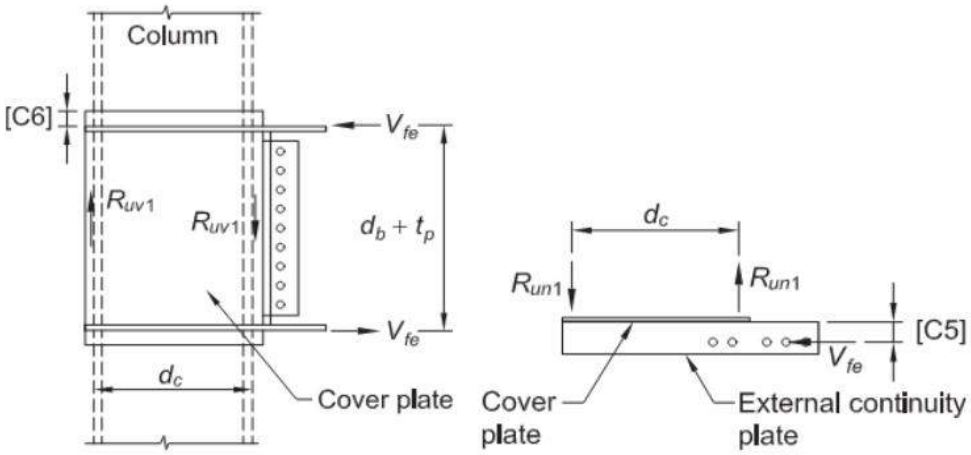
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	90991	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	31.56	23.51	50.00	742.2
W4TL	31.11	16.56	36.01	515.2
W1LB	31.41	23.34	48.02	733.1
W4BR	30.01	16.59	35.81	497.9
W1RC	20.31	15.60	2.53	316.7
W4TC	25.17	12.13	1.66	305.3
W1LC	21.03	15.70	2.45	330.1
W4BC	24.35	12.05	1.71	293.5
		23.51	50.00	

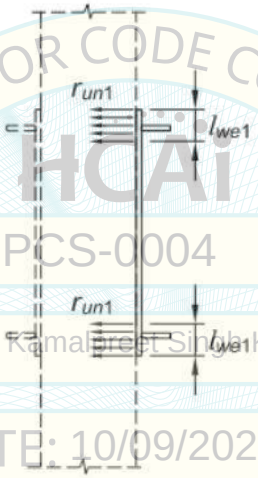
Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	23.5	
Orientation of Resultant Force	θ_r (degrees)	50.0	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.97	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	11.0	
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)	11.0	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.34	
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)	18.18	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.60	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/03/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

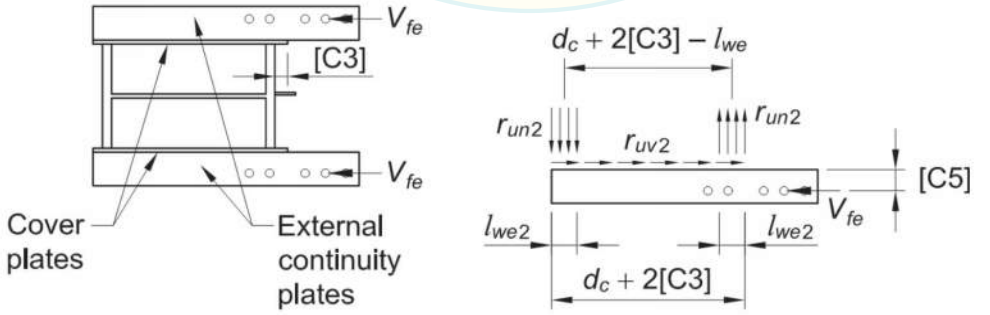
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	880.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	32.5	
Column depth	d_c (in)	41.3	
Column flange thickness	t_{fc} (in)	2.36	
Cover plate overhang dimension	*C3* (in)	1	
External continuity plate dimension	*C5* (in)	3.5	
Weld 2 size	D_{W2} (1/16 in)	13	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	0.81	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), t_{fc} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	3.42	
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)	21.1	
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)	24.4	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	32.3	
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)	36.2	
External continuity plate-to-cover plate weld tearing check, $r_u/\phi r_n$	*D/C*	0.89	**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)*	880.4	
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	3195.3	
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	912.9	
External continuity plate thickness (see step 2)	*T5* (in)	2.000	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	2598.0	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	42185.0	
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.43	**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)*	880.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	32.5	
Column fillet dimension	*k* (in)	3.5	
Column fillet dimension	*k1* (in)	1.9	
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	151.8	
Column flange width	b_{fc} (in)	16.2	
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	57.3	
Weld 2 size	D_{W2} (1/16 in)	13	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	3662.1	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	1382.4	
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)	5186.5	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	3889.9	
Weld 2 demand/capacity check for cap plate configuration, $R_u/\phi R_n$	*D/C*	0.23	**OK**



(a) Applied loads *(b) Distributed shear and normal demands*

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

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	880.4
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	32.5
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	7
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	532.5
External continuity plate thickness (see step 2)	*T5* (in)	2
External continuity plate width	*C4* (in)	7.75
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Net area of external continuity plate cross section, $T5([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	12.625
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	820.6
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	615.5
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, $n_{rup1} + 1$	n_{rup}	8
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	608.6
External continuity plate dimension	*C2 (in)*	-1.5
Cover plate overhang dimension	*C3* (in)	1
External continuity plate dimension	*C5* (in)	3.5
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	-0.5
External continuity plate thickness (see step 2)	*T5* (in)	2
External continuity plate width	*C4* (in)	7.75
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(T5)/4[C5]$	*Bonus* (in)	0.036
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Area of external continuity plate cross section with bonus, $T5([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	12.661
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	822.9
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	617.2
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, $n_{rup1} + 2$	n_{rup}	9
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	684.7
Bolt spacing (see schedule)	*s* (in)	3.75
External continuity plate dimension	*C2 (in)*	-1.5
Cover plate overhang dimension	*C3* (in)	1
External continuity plate dimension	*C5* (in)	3.5
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	3.25
External continuity plate thickness (see step 2)	*T5* (in)	2
External continuity plate width	*C4* (in)	7.75
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(T5)/(4[C5])$	*Bonus* (in)	1.509
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Area of external continuity plate cross section with bonus, $(T5)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	14.134
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	918.7
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	689.0
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.99 **OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	8
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	608.6
External continuity plate width	*C4* (in)	7.75
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	-0.5
External continuity plate thickness (see step 2)	*T5* (in)	2
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Gross tension area, *T5[C4]*	A_{gt} (in^2)	15.500
Gross shear area, *T5[C9]*	A_{gv} (in^2)	1.000
Net tension area, $A_{gt} - ((T5)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	12.625
Net shear area, A_{gv}	A_{nv} (in^2)	1.000
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	850.6
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	638.0
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.95 **OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Ramandeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column		
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	9
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	684.7
Bolt spacing (see schedule)	s (in)	3.75
External continuity plate dimension	$C2$ (in)	-1.5
Cover plate overhang dimension	$C3$ (in)	1
External continuity plate width	$C4$ (in)	7.75
External continuity plate dimension, $[C2] + [C3] + s$	$C7$ (in)	3.25
External continuity plate thickness (see step 2)	$T5$ (in)	2
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Gross tension area, $T5[C4]$	A_{gt} (in^2)	15.500
Gross shear area, $T5[C7]$	A_{gv} (in^2)	6.500
Net tension area, $A_{gt}-((T4)(d_{hole}+1/16))$	A_{nt} (in^2)	12.625
Net shear area, A_{gv}	A_{nv} (in^2)	6.500
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	1015.6
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	761.7
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.90 **OK**

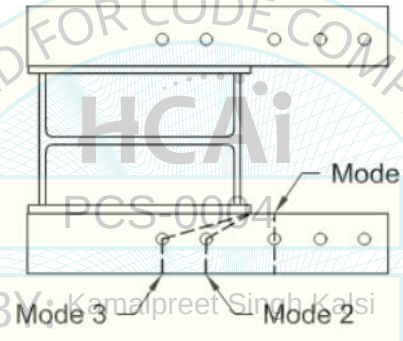


Fig. C-15.6. Tension rupture limit state for external continuity plate.

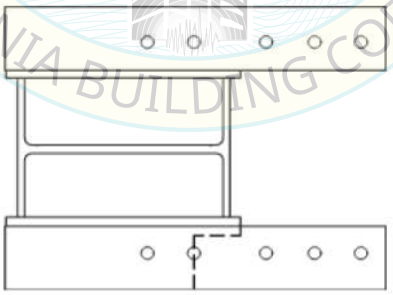


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum bay width	B (in)	420.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	25.0
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	81000
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	452.8
Beam depth	d_b (in)	44
Beam web thickness	t_{wb} (in)	1.03
Beam web area, $d_b t_{wb}$	A_w (in)	45.32
Beam web width-to-thickness ratio, h / t_w	l_{web}	38
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)	1359.6
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.33 **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)	880.4
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	32.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	1825.9
Beam flange thickness	t_{fb} (in)	1.77
Beam flange width	b_{fb} (in)	15.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375
Number of holes along each bolt line (see step 5)	n_b	12
Length of connection on beam	$B4$ (in)	43
Distance from beam centerline to bolt line	$P2$ (in)	3
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in^2)	17.523
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	152.220
Net tension area, $A_{gt} - (t_{fb}(d_{hole,std} + 1/16))$	A_{nt} (in^2)	14.979
Net shear area, $A_{gv} - (t_{fb}(d_{hole,std} + 1/16)(2n_b - 1))$	A_{nv} (in^2)	93.699
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)	4627.9
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	3470.9
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.53 **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)	452.8
Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	1.227
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	103.1
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	77.3
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	6
Provided number of shear tab bolts	n_n	6 **OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	38
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - *1in*	$l_{tab,req}$ (in)	37
Length of shear tab (see drawings), T_{beam} - *1in*	l_{tab} (in)	37 **OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	1.03
Shear tab provided 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), $2.4d_{bolt}t_{min}F_{u,min}n_n$	R_{nb} (kips)	731.3
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375
Edge distance on shear tab (AISC 360 Table J3.4)	e (in)	1.625
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	6.750
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)	5.4

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY • Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{u,min}\ (n_n\ -1)$	R_{nt1} (kips)	1310.2	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e\ -\ d_{hole,std}\ /\ 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{u,min}$	R_{nt2} (kips)	45.7	
Total tearout strength, $R_{nt1}\ +\ R_{nt2}$	R_{nt} (kips)	1355.9	
Nominal shear strength of all bolts, $F_{nv}A_{bolt}n_n$	R_{nv} (kips)	618.5	
Controlling strength, $min(R_{nb},\ R_{nt,}\ R_{nv})$	R_n (kips)	618.5	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	463.9	
Shear tab bolt shear demand/capacity ratio, $R_u\ /\ \phi R_n$	D/C	0.98	**OK**

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

Minimum fastener tension (AISC 360 Table J3.1)	T_b	102	
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)	51.9	
Provided number of shear tab bolts (see step 12)	n_n	6	
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	311.2	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	452.8	
Distance from alignment line to column	$C1$ (in)	5.25	
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)	2377.1	

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Slotted bolt hole width (standard slot width)	w_b (in)	1.375	
Shear tab provided thickness	$T3$ (in)	0.625	
Shear tab length (see step 12)	l_{tab} (in)	37	
Beam T dimension limit	T_{beam} (in)	38	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Slotted bolt hole spacing, $(T_{beam}\ -\ 1"\ -\ 2e_d)\ /\ (n_n\ -\ 1)$	s_{slot} (in)	6.6875	
Gross area of shear tab, $T3l_{tab}$	A_{gv} (in^2)	23.13	
Shear tab net area, $A_{gv}\ -n_n(w_b+1/16)T3$	A_{nv} (in^2)	17.73	
Effective net area, $A_e\ =A_{nv}$	A_e (in^2)	17.73	
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{uST}A_{nv}$	ϕR_{nv} (kips)	518.73	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{uST}A_e$	ϕR_{nn} (kips)	864.55	
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	0.71	**OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	452.78	
Required axial strength of the shear tab (see above)	P_u (kips)	311.20	
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)	2377.09	
Shear tab gross area, $T3l_{tab}$	A_{gv} (in^2)	23.13	
Shear tab plastic section modulus, $(l_{tab}/2)^2\ T3$	Z_{tab} (in^3)	213.91	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yST}A_{gv}$	ϕR_{nv} (kips)	693.75	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yST}A_{gv}$	ϕR_{nn} (kips)	1040.63	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_{yST}Z_{tab}$	ϕM_n (kips)	9625.78	
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**

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	37
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	452.8
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	12.2
Number of slotted bolt holes in shear tab	n_{vb}	6
Slotted bolt hole spacing	s_{slot} (in)	6.6875
Distance from line of bolts to shear tab weld	$C1$ (in)	5.25
Size of weld in 1/16^{ths} of an inch	D_{W3}	9
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)	2377.1
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)	21.0
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	24.3
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)	25.1
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.97 **OK**

This check is N/A when bridge plates do not apply

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e (in)	1.75
Beam depth	d_b (in)	44
Length of the shear tab (see step 12)	l_{tab} (in)	37
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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (in)	2.95
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	3 **OK**
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	3

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

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	880.4
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	32.5
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	912.9

Yielding check		
Top plate thickness (see step 2)	$T4$ (in)	2
Top plate dimension (see schedule)	$P8$ (in)	45.25
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	90.500
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	2715.0
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	2715.0
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.34 **OK**

Rupture check		
Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.625
Number of short slotted bolts on top plate (see step 5)	n_b	12
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	51.500
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	2008.5
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	1506.4
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.61 **OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

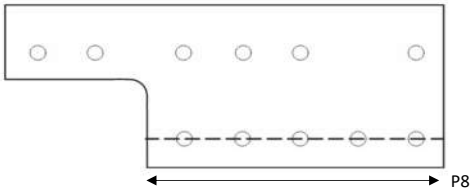


Fig. C-15.8. Shear rupture of net section on top plate.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	880.4
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	32.5
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	912.9

Yielding check (LS 1)

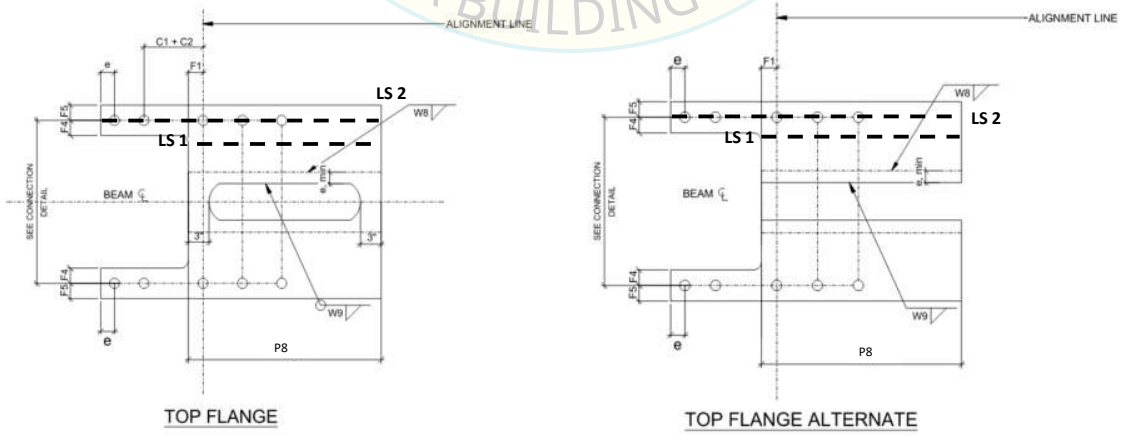
Top plate thickness (see step 2)	*T4* (in)	2
Top plate dimension (see schedule)	*P8* (in)	40.75
Fuse plate dimension	*F1 (in)*	2.00
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	81.500
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	2445.0
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	2445.0
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.37 **OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Number of oversized bolts in Zone M (see step 5)	n_m	6
Number of oversized bolts on top plate (see step 5)	n_b	12
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.625
Bolt spacing (see schedule)	*s* (in)	3.75
Fuse plate dimension	*F1 (in)*	2.00
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	62.875
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	125.750
Number of oversized bolts on top plate (see step 5)	n_b	12
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	88.250
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	3441.8
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	2581.3
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.35 **OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	912.9
Beam flange width	b_{fb} (in)	15.9
Weld 8 size	D_{W8} (1/16 in)	9
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.56
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	39.63
Weld 9 size	D_{W9} (1/16 in)	9
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	34.75
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	12.3
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	12.5
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.98 **OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	880.4
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	32.5
Total number of bolts (see step 5)	n_b	12
Number of bolts in the M region (see step 5)	n_m	6
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)	456.5

Yielding check

Top plate dimension	*P4* (in)	2.25
Top plate dimension	*P5* (in)	4.125
Top plate thickness (see step 2s)	*T4* (in)	2
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	12.750
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	637.5
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	573.8
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.80 **OK**

Rupture check

Top plate dimension	*P4* (in)	2.25
Top plate dimension	*P5* (in)	4.125
Top plate thickness (see step 2)	*T4* (in)	2
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Effective area of rupture, *([P4]+[P5]-d_O-1/16)(T4)*	A_e (in^2)	9.500
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	617.5
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	463.1
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.99 **OK**

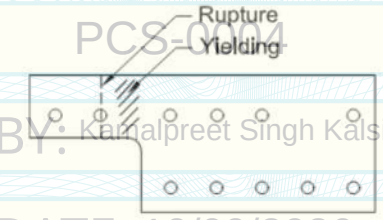


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

Entering and tightening check

Beam flange fillet dimension	k_1	1.8
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.5625
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	3.3125
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)	880.4
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	32.5
Top plate thickness (see step 2)	*T4* (in)	2
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	*P10* (in)	12.475
Inside length of the top plate	*P8* (in)	45.25
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop} T4)$	*m* (in)	16.906
Dimension, *([P8] - m) / 2*	*e* (in)	14.172
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), *[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)*	$P2_{req}$ (in)	0.0
Provided P2 dimension	*P2* (in)	3 **OK**

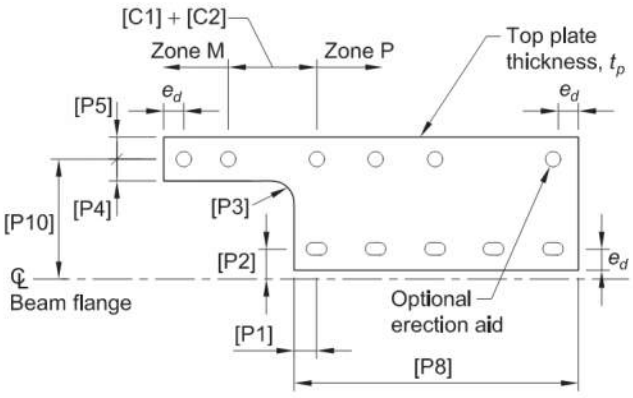


Fig. 15.6. Top plate detail.

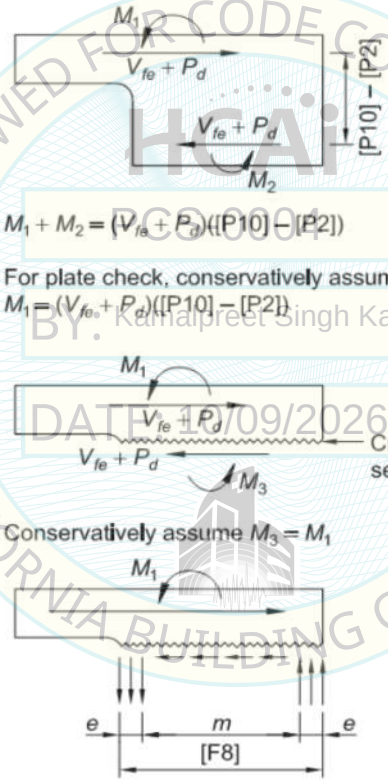


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAI
PCO-0001
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

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

Refer to AISC 358 Fig. C-15.11

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)	4	**OK**
Fuse plate thickness (see step 2)	$T2$ (in)	2	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	2.0	**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)	880.4	
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)	$n_{b,\ Bot}$	12	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	6	
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)	880.4	
Fuse plate dimension, centerline to gage line	$F7$ (in)	3	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	3.225	
Fuse plate thickness (see step 2)	$T2$ (in)	2	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
Gross area for yielding, $2([F7]+[F8])T2$	A_g (in^2)	24.900	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	1245.0	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	1120.5	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.79	**OK**

Step 18. (continued)

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)	3.225	
Fuse plate thickness (see step 2)	$T2$ (in)	2	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.563	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	18.400	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	P_n (kips)	1435.2	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	1076.4	
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.82	**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

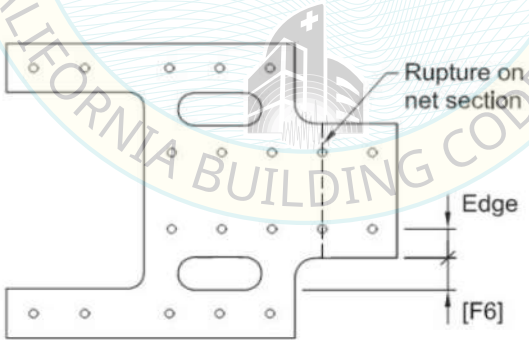


Fig. C-15.11. Limit states related to dimension [F6].

HCAi
PCS-0004
Reviewed By: Ramandeep Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	4
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	81000
Fuse plate thickness (see step 2)	*T2* (in)	2
Top plate thickness (see step 2)	*T4* (in)	2
Beam depth	d_b (in)	44
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	452.8
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102
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_n	6
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	207.5
Resistance factor for bolt slip	ϕ	0.7
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	217.8
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	87.1
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	4.116
Provided yielding region depth	*F2* (in)	4.000 **OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	480.0
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	880.4
Lower bound ratio	V_{fe}/V_y *LB*	1.7
Ratio	V_{fe}/V_y	1.8 **OK**

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85
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]* <= *1.85*	*F6 / F2*	0.97 **OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	880.4
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)	$n_{b,\ Bot}$	12
Number of bolts in the M region	$n_{m,\ Bot}$	6
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)	880.4

Yielding check

Fuse plate dimension	*F4* (in)	2.25
Fuse plate dimension	*F5* (in)	3.25
Fuse plate thickness (see step 2)	*T2* (in)	2
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Gross area of yielding, *2([F4]+[F5])(T2)*	A_g (in^2)	22
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	R_n (kips)	1100.00
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	990.00
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.89 **OK**

Rupture check

Fuse plate dimension	*F4* (in)	2.25
Fuse plate dimension	*F5* (in)	3.25
Fuse plate thickness (see step 2)	*T2* (in)	2
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	15.5
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	R_n (kips)	1209.0
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	906.8
Rupture demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.97 **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

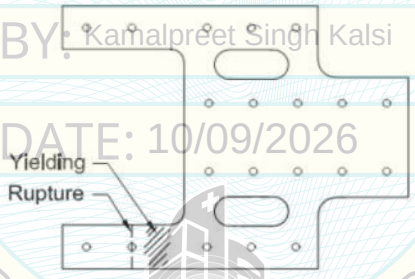


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

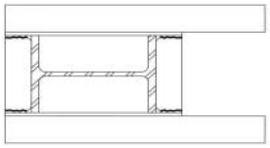
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

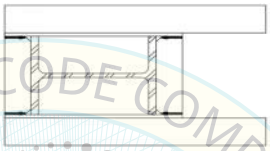
Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	880.4	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0	
Drag force per line of bolts, $P_d / 2$	P_d (kips)	32.5	
External continuity plate dimension	$C5$ (in)	3.5	
Column depth	d_c (in)	41.3	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	$C3$ (in)	1	
Bridge plate thickness (not greater than 2W5)	$T4$ (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}\ T4([C3]-1")$	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	D/C	N/A	**N/A**



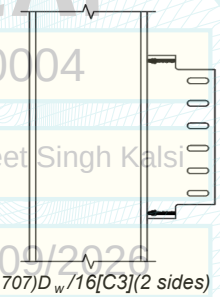
Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	N/A	**N/A**



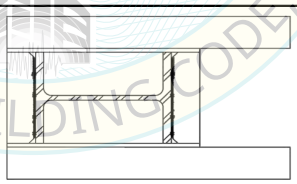
Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	420.0	
Column depth	d_c (in)	41.3	
Beam depth	d_b (in)	44	
Shear force from gravity	$V_{gravity}$ (kips)	25.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	81000	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	452.8	
Distance from alignment line to column	$C1$ (in)	5.25	
Weld 7 demand, $V_u\ C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	$C3$ (in)	1	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	N/A	**N/A**



Bridge plate to column flange weld failure check

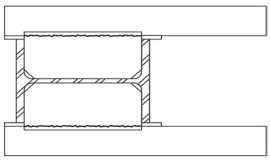
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	$C1$ (in)	5.25	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	$C3$ (in)	1	
Column flange width	b_{fc} (in)	16.2	
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	2	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	N/A	**N/A**



*These checks are N/A when bridge plates do not apply

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

Column depth	d_c (in)	41.3	
Column flange thickness	t_{fc} (in)	2.36	
Weld 4 size	D_{W4} (1/16 in)	10	
Weld 4 effective length, $d_c - 2\ t_{fc}$	l_{eff} (in)	36.58	
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)	509.2	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	0.750	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	823.1	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6	**OK**



*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2025
CALIFORNIA BUILDING CODE 2025

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)	31100
Column flange width	b_{fc} (in)	16.2
Beam depth	d_b (in)	44
Bay width	B (in)	420.0
Number of bolts in zone M	M	6
Number of bolts in zone P	P	6
Bolt spacing	s (in)	3.75
Cover plate thickness	$T1$ (in)	0.875
Fuse plate thickness	T2 (in)	2
Top plate thickness	$T4$ (in)	2
External continuity plate dimension	$C5$ (in)	3.5
Fuse plate dimension	$F2$ (in)	4
Fuse plate dimension	$F3$ (in)	14.5
Fuse plate dimension	$F4$ (in)	2.3
Fuse plate dimension	$F6$ (in)	4
Fuse plate dimension	$F8$ (in)	3.225
Distance from beam centerline to bolt line	$P2$ (in)	3
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	2.24E+05
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	4.48E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	4.29E+04
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	1.56E+05
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	3.54E+04
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	1.51E+05
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	6.07E+07
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	3.87E+07
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.64 **OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

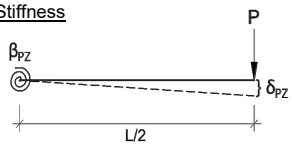
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

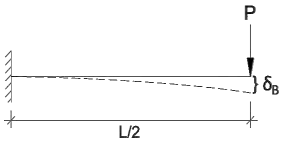


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b	(in)	44
Beam flange thickness	t_{fb}	(in)	1.77
Column depth	d_c	(in)	41.3
Column flange width	b_{fc}	(in)	16.2
Column web thickness	t_{wc}	(in)	1.34
Column flange thickness	t_{fc}	(in)	2.36
Cover plate overhang	$C3$	(in)	1.0
Cover plate thickness	$T1$	(in)	0.875
Bay width	B	(in)	420.0
Story height	H	(in)	192.0
Panel zone factor, $d_b d_c t_{wc} + 2d_b(d_c + 2[C3])T1$	V_p	(in^3)	5769.1
Column ratio, $(d_c - t_{fc}) / B$	α		0.093
Beam ratio, $(d_b - t_{fb}) / H$	β		0.220
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps}	(k-in)	1.37E+08
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs}	(k-in)	1.67E+06
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ}	(k-in)	N/A
Beam moment of inertia	I_{xb}	(in^3)	31100
Required panel zone spring stiffness, *60EI xb / B for one-sided, 120EI xb / B for two-sided*	K_{req}	(k-in)	N/A
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*		N/A **N/A**

*This check is N/A when connection stiffness is applied in the model analysis

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

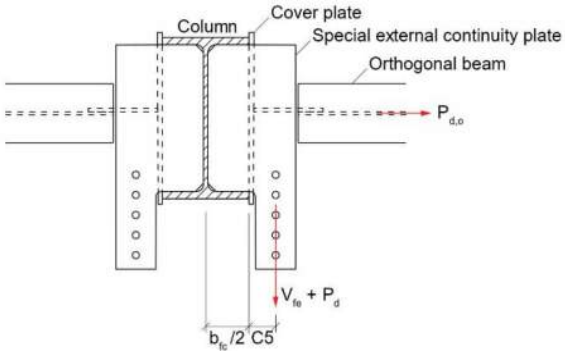
Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

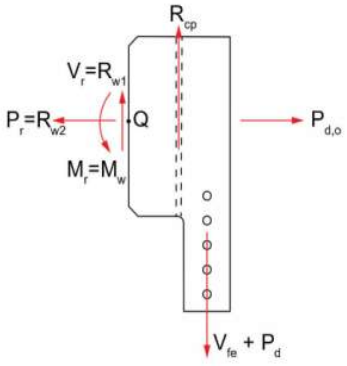
Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	880.4
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	65.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	32.5
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
Column flange width	b_{fc} (in.)	16.2
Column depth	d_c (in.)	41.3
Column flange thickness	t_{fc} (in.)	2.36
Column web thickness	t_{wc} (in.)	1.34
External continuity plate thickness	$T5$ (in)	2.000
External continuity plate gage dimension	$C5$ (in.)	3.50
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	669.05	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	73.16	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
	Special detailing not required		

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	3.54	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	585.5	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	68.44	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply	Special detailing not required		

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025





Figures 13 and 14 PCS-0004

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth		
Orthogonal beam size		
Orthogonal beam depth	$d_{b,o}$ (in)	
In plane moment frame beam 1		W44X335
In plane moment frame beam 2		NA
Max in-plane moment frame beam depth	d_b (in)	44
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0
New continuity plate required for deeper cantilever beam		N/A
New continuity plate width past column flange	C4 (in)	7.75
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A
Simply supported plate buckling coefficient	k	0.43
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A **N/A**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

Connection ID DF103

W40X431 Column and BUW44X1.875X16.9X2.875 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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

HCAi
REVIEWED FOR CODE COMPLIANCE
PCS#004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Beam limitation checks			
Beam size		BUW44X1.875X16.9X2.875	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	44	
Beam depth	d_b (in.)	44	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	574.7	**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}	20.4	**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}	2.9	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		W40X431	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	44	
Column depth	d_c (in.)	41.3	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	431	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	2.36	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	1000.0	
Gross area of the column section	A_g (in^2)	127	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	50.9	
Column web width-to-thickness ratio	λ_{web}	25.5	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1),	$\lambda_{hd,flange}$	7.35	
Factor based on code and member type, $F(E/(R_{yc}F_{yc}))^{0.5}$	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	3.44	**OK**

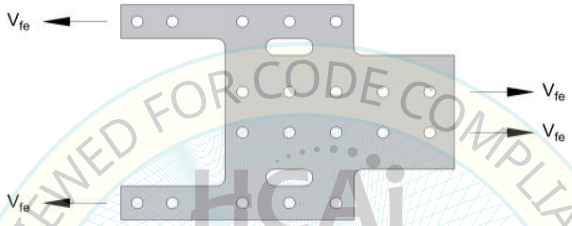
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	1960	
Column design axial load	P_u (kips)	1000.0	
Column area	A_{gc} (in^2)	127	
Story Height	H (in)	192.0	
Column depth	d_c (in)	41.3	
Beam depth	d_b (in)	44	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c*(F_{yc}-P_u/A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	2.61E+05	
Beam plastic section modulus	Z_b (in^3)	2683.9715	
Minimum bay width	B (in)	420.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	134199	
Clear distance between column faces, $B-d_c$	L_h (in)	378.7	
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)	25.0	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	733.7	
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	15152	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	1.49E+05	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.75	**OK**

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

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)	2683.9715	
Beam plastic bending moment , $F_{yb}Z_b$	M_p (k-in)	134199	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	134199	**OK**
Fuse plate thickness	$T2$ (in)	2.25	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	3	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	3	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	44	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	1439.1	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	0.0	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	1439.1	



REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAI
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1439.1	
Beam area	A_{gb} (in^2)	168.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	27.5	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	1466.6	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	25	
Fuse plate thickness	$T2$ (in)	2.25	
Top plate thickness	$T4$ (in)	3	
External continuity plate thickness	$T5$ (in)	3	
Beam depth	d_b (in)	44	
Column depth	d_c (in)	41.3	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	1652.7	

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

Cover plate thickness	$T1$ (in)	1.125	
Cover plate overhange dimension	$C3$ (in)	4.25	
Column depth	d_c (in)	41.3	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	49.8	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	1680.8	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	1680.8	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.87	**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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag cond	h_{cp} (in)	50.00	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	1687.5	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	1687.5	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.98	**OK**

Step 3. (continued)

Cover plate thickness check (AISC 341 E3.6e.2)			
Beam depth	d_b (in)	44	
Beam flange thickness	t_{fb} (in)	2.875	
Column flange thickness	t_{fc} (in)	2.36	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	38.25	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	36.58	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.831	
Thickness check, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	1.35	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.25	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Beam plastic section modulus	Z_b (in^3)	2683.9715	
Beam flange thickness	t_{fb} (in)	2.875	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	4.313	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	354.7	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	2329.3	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	147618	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	166543	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.89	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	77.3	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	19.0	
Number of bolts per line (AISC 358 15.2(1))	n_b	19	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	3.75	
Top plate thickness	$T4$ (in)	3	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	5.125	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.6001	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	10	**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	9	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	1466.6	
Beam flange thickness	t_{fb} (in)	2.875	
Top plate thickness	$T4$ (in)	3	
Governing material thickness, min$(t_{fb}, T4)$	t_{min} (in)	2.9	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	10651.9	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.2	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	8829.8	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	217.2	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	9047.1	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1958.6	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1958.6	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	1468.9	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	1.00	**OK**

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/03/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	0	
Fuse plate thickness	$T2$ (in)	2	
Top plate thickness	$T4$ (in)	3	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	0.0	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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	19	
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)	657.0	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	558.4	
Wind slip demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.00	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1439.1	
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, $P_{d,total}/2$	P_d (kips)	0.0	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	103.1	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	14.0	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	19	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	3.75	
Fuse plate thickness	$T2$ (in)	2.25	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	6.833	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	4.5	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	10	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	9	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	1439.1	
Beam flange thickness	t_{fb} (in)	2.875	
Fuse plate thickness	$T2$ (in)	2.25	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	2.3	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	10651.9	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.2	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	8829.8	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	170.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	8999.9	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1958.6	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1958.6	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	1468.9	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.98	**OK**

REVIEWED FOR CODE COMPLIANCE
CAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 5b. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	0	
Bolt group demand from wind per line, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	0.0	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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 on the bottom flange	$n_{b,Bot}$	19	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	657.0	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	558.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.00	**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)	44	
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	*e* (in)	1.75	
Beam flange width factor, *0 if* $b_{fb} < b_{fc}$, *1.0 otherwise*	α	1	
Cover plate overhang dimension	*C3* (in)	4.25	
Required distance from alignment line to column (AISC 358 Eq. 15.6-10), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	8.64	
Distance from alignment line to column	*C1* (in)	8.75	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	27.5	
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	25	
Column depth	d_c (in)	41.3	
Column flange thickness	t_{fc} (in)	2.36	
Beam depth	d_b (in)	44	
Fuse plate thickness	$T2$ (in)	2.25	
Top plate thickness	$T4$ (in)	3	
External continuity plate thickness	$T5$ (in)	3	
Cover plate thickness	$T1$ (in)	1.125	
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2T5$ for cap plate and ortho. cant./drag condit	h_{cp} (in)	50	
Gage dimension from the cover plate to the line of bolts on the external continuity plates	$C5$ (in)	3.75	
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	1.0625	
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	47.88	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $T5 + T1 + [C6]$	l_{We1} (in)	7.125	
Weld 1 size	D_{W1} (1/16 in)	15	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	15	
Weld 4 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	36.58	
Horizontal distance from center of rotation to geometric center	e_x (in)	0.24	
Vertical distance from center of rotation to geometric center	e_y (in)	0.24	

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.

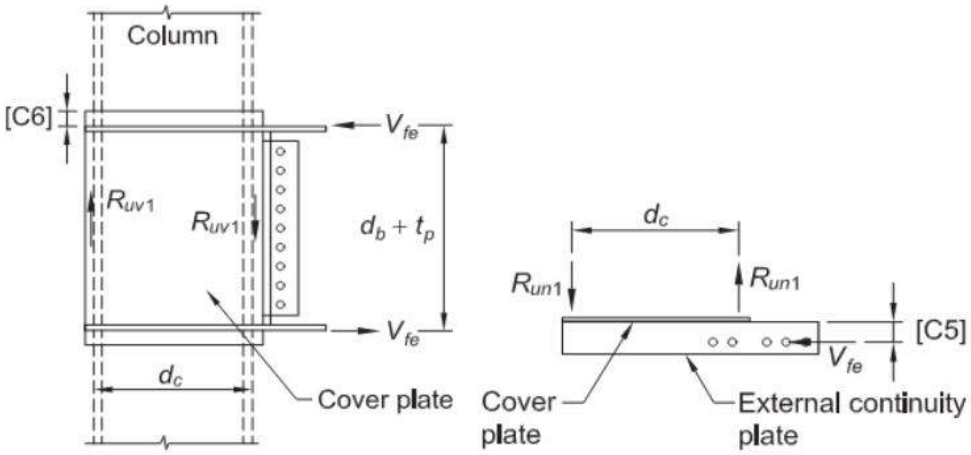
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	135178	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	31.27	34.64	49.27	1083.1
W4TL	30.89	24.82	36.02	766.7
W1LB	31.24	34.46	48.02	1076.5
W4BR	30.22	24.84	35.80	750.6
W1RC	20.42	22.98	2.01	469.2
W4TC	24.99	18.14	1.40	453.3
W1LC	20.91	23.08	1.96	482.6
W4BC	24.52	18.07	1.43	443.1
		34.64	49.27	

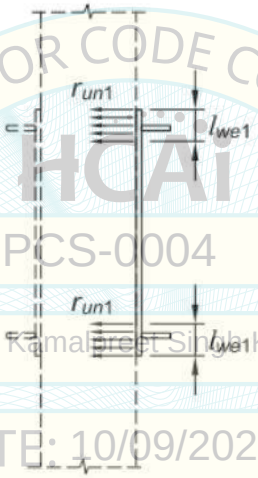
Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	34.6	
Orientation of Resultant Force	θ_r (degrees)	49.3	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.96	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	18.1	
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)	18.1	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.35	
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)	27.13	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.67	**OK**

HCAi PCS-0004 BY: Kamalpreet Singh Kalsi DATE: 10/03/2026 REVIEWED FOR CODE COMPLIANCE CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

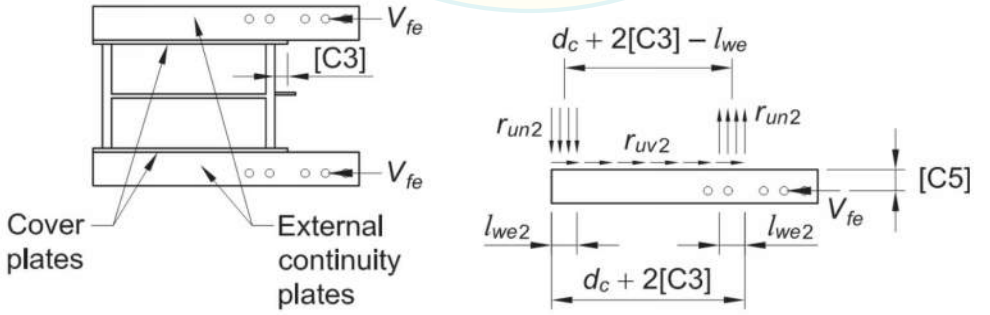
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	1439.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5
Column depth	d_c (in)	41.3
Column flange thickness	t_{fc} (in)	2.36
Cover plate overhang dimension	*C3* (in)	4.25
External continuity plate dimension	*C5* (in)	3.75
Weld 2 size	D_{W2} (1/16 in)	17
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	1.06
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), t_{fc} + *T1* + *[C3]* - l_{hb2}	$l_{we2,eff}$ (in)	6.67
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)	29.5
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)	20.1
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2+r_{un}^2}$	r_u (kip/in)	35.7
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)	47.3
External continuity plate-to-cover plate weld tearing check, $r_u/\phi r_n$	*D/C*	0.75 **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)*	1439.1
External continiuty plate moment demand, $(\Sigma V_{fe}+P_d)[C5]$	M_u (kips)	5499.9
Weld 2 total demand, $\Sigma V_{fe}+P_d$	R_u (kips)	1466.6
External continuity plate thickness (see step 2)	*T5* (in)	3.000
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3 **OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c+2[C3])T5$	ϕR_n (kips)	4482.0
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}T5((d_c+2*[C3])/2)^2$	ϕM_n (kips)	83701.4
Demand/Capacity Ratio, $R_u/\phi R_n + M_u/\phi M_n$	*D/C*	0.39 **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)*	1439.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5
Column fillet dimension	*k* (in)	3.5
Column fillet dimension	*k1* (in)	1.9
Weld 2 length oriented longitudinally for loading, $2(d_c+2[C3]-2*l_{hb2})+2*(d_c-2k)$	$l_{w2,l}$ (in)	163.8
Column flange width	b_{fc} (in)	16.2
Weld 2 length oriented transverse to loading, $2*b_{fc}+2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	57.3
Weld 2 size	D_{W2} (1/16 in)	17
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	5167.6
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	1807.8
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)	7104.1
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	5328.1
Weld 2 demand/capacity check for cap plate configuration, $R_u/\phi R_n$	*D/C*	0.28 **OK**



(a) Applied loads *(b) Distributed shear and normal demands*

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

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	10	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	771.9	
External continuity plate thickness (see step 2)	*T5* (in)	3	
External continuity plate width	*C4* (in)	6.75	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Net area of external continuity plate cross section, $T5([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	15.938	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	1035.9	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	777.0	
Mode 1 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.99	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	11	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	849.1	
External continuity plate dimension	*C2 (in)*	0	
Cover plate overhang dimension	*C3* (in)	4.25	
External continuity plate dimension	*C5* (in)	3.75	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	4.25	
External continuity plate thickness (see step 2)	*T5* (in)	3	
External continuity plate width	*C4* (in)	6.75	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(T5)/4[C5]$	*Bonus* (in)	3.613	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Area of external continuity plate cross section with bonus, $T5([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	19.550	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	1270.8	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	953.1	
Mode 2 rupture demand/capacity Ratio, $P_u / \phi P_n$	*D/C*	0.89	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	12	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	926.3	
Bolt spacing (see schedule)	*s* (in)	3.75	
External continuity plate dimension	*C2 (in)*	0	
Cover plate overhang dimension	*C3* (in)	4.25	
External continuity plate dimension	*C5* (in)	3.75	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	8	
External continuity plate thickness (see step 2)	*T5* (in)	3	
External continuity plate width	*C4* (in)	6.75	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(T5)/(4[C5])$	*Bonus* (in)	12.800	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Area of external continuity plate cross section with bonus, $(T5)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	28.738	
Nominal tensile rupture capacity, A_nF_{uECP}	P_n (kips)	1867.9	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	1401.0	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.66	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	11	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	849.1	
External continuity plate width	*C4* (in)	6.75	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	4.25	
External continuity plate thickness (see step 2)	*T5* (in)	3	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	20.250	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	12.750	
Net tension area, $A_{gt} - ((T5)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	15.938	
Net shear area, A_{gv}	A_{nv} (in^2)	12.750	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	1418.4	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	1063.8	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.80	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Ramandeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column		
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	12
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	926.3
Bolt spacing (see schedule)	s (in)	3.75
External continuity plate dimension	*C2 (in)*	0
Cover plate overhang dimension	*C3* (in)	4.25
External continuity plate width	*C4* (in)	6.75
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	8
External continuity plate thickness (see step 2)	*T5* (in)	3
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Gross tension area, *T5[C4]*	A_{gt} (in^2)	20.250
Gross shear area, *T5[C7]*	A_{gv} (in^2)	24.000
Net tension area, A_{gt}*-((T4)(d_{hole}+1/16))*	A_{nt} (in^2)	15.938
Net shear area, A_{gv}	A_{nv} (in^2)	24.000
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	1755.9
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	1317.0
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.70 **OK**

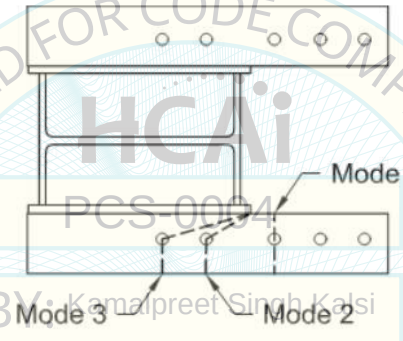


Fig. C-15.6. Tension rupture limit state for external continuity plate.

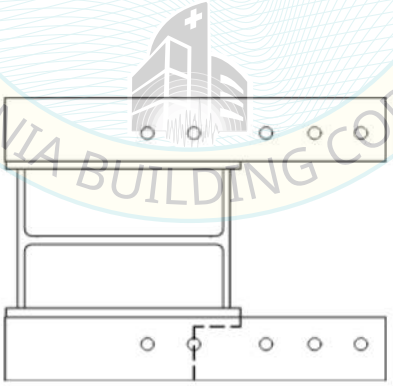


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

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

Minimum bay width	B (in)	420.0
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	25.0
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	134199
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	733.7
Beam depth	d_b (in)	44
Beam web thickness	t_{wb} (in)	1.875
Beam web area, $d_b t_{wb}$	A_w (in)	82.5
Beam web width-to-thickness ratio, h / t_w	l_{web}	20.4
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)	2475.0
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.30 **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)	1439.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	2933.3
Beam flange thickness	t_{fb} (in)	2.875
Beam flange width	b_{fb} (in)	16.9
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375
Number of holes along each bolt line (see step 5)	n_b	19
Length of connection on beam	$B4$ (in)	69.25
Distance from beam centerline to bolt line	$P2$ (in)	4.0625
Gross tension area, $(b_{fb} - 2[P2])t_{fb}$	A_{gt} (in^2)	25.228
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	398.188
Net tension area, $A_{gt} - (t_{fb}(d_{hole,std} + 1/16))$	A_{nt} (in^2)	21.095
Net shear area, $A_{gv} - (t_{fb}(d_{hole,std} + 1/16)(2n_b - 1))$	A_{nv} (in^2)	245.273
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)	10936.9
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	8202.6
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.36 **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)	733.7
Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	1.227
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	103.1
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	77.3
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	9
Provided number of shear tab bolts	n_n	10 **OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	34.5
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - *1in*	$l_{tab,req}$ (in)	34
Length of shear tab (see drawings), T_{beam} - *1in*	l_{tab} (in)	33.5 **OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	1.875
Shear tab provided thickness	$T3$ (in)	1.375
Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	1.4
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}t_{min}F_{u,min}n_n$	R_{nb} (kips)	2681.3
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375
Edge distance on shear tab (AISC 360 Table J3.4)	e (in)	1.625
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	3.361
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)	2.0

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY • Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c1}\ t_{min}\ F_{u,min}\ (n_n\ -1)$	R_{nt1} (kips)	1917.1	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2l_{c2}\ t_{min}\ F_{u,min}$	R_{nt2} (kips)	100.5	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	2017.6	
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	1030.8	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1030.8	
Factored shear capacity of all bolts, $0.75*R_n$	ϕR_n (kips)	773.1	
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.95	**OK**

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

Minimum fastener tension (AISC 360 Table J3.1)	T_b	102	
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)	51.9	
Provided number of shear tab bolts (see step 12)	n_n	10	
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	518.7	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	733.7	
Distance from alignment line to column	$C1$ (in)	8.75	
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)	6420.2	

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Slotted bolt hole width (standard slot width)	w_b (in)	1.375	
Shear tab provided thickness	$T3$ (in)	1.375	
Shear tab length (see step 12)	l_{tab} (in)	34	
Beam T dimension limit	T_{beam} (in)	Other	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	3.3125	
Gross area of shear tab, $T3l_{tab}$	A_{gv} (in^2)	46.06	
Shear tab net area, $A_{gv} - n_n(w_b + 1/16)T3$	A_{nv} (in^2)	26.30	
Effective net area, $A_e = A_{nv}$	A_e (in^2)	26.30	
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75)0.6F_{uST}A_{nv}$	ϕR_{nv} (kips)	769.18	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75)F_{uST}A_e$	ϕR_{nn} (kips)	1281.97	
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	0.99	**OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	733.73	
Required axial strength of the shear tab (see above)	P_u (kips)	518.67	
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)	6420.16	
Shear tab gross area, $T3l_{tab}$	A_{gv} (in^2)	46.06	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	385.77	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0)0.6F_{yST}A_{gv}$	ϕR_{nv} (kips)	1381.88	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9)F_{yST}A_{gv}$	ϕR_{nn} (kips)	2072.81	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9)F_{yST}Z_{tab}$	ϕM_n (kips)	17359.80	
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	0.51	**OK**

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	33.5	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	733.7	
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	21.9	
Number of slotted bolt holes in shear tab	n_{vb}	10	
Slotted bolt hole spacing	s_{slot} (in)	3.3125	
Distance from line of bolts to shear tab weld	$C1$ (in)	8.75	
Size of weld in 1/16^{ths} of an inch	D_{W3}	20	
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)	6420.2	
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)	49.3	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	53.9	
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)	55.7	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.97	**OK**

This check is N/A when bridge plates do not apply

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e (in)	1.75	
Beam depth	d_b (in)	44	
Length of the shear tab (see step 12)	l_{tab} (in)	33.5	
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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (in)	2.845	
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	2.875	**OK**
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	2.875	

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

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	27.5	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1466.6	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	3	
Top plate dimension (see schedule)	$P8$ (in)	71.5	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	214.500	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	6435.0	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	6435.0	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.23	**OK**

Rupture check			
Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.625	
Number of short slotted bolts on top plate (see step 5)	n_b	19	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	121.875	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	4753.1	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	3564.8	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.41	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

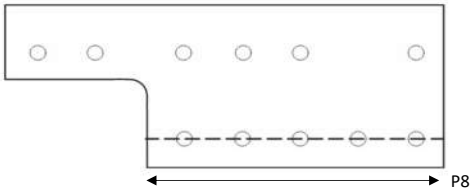


Fig. C-15.8. Shear rupture of net section on top plate.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	27.5
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1466.6

Yielding check (LS 1)

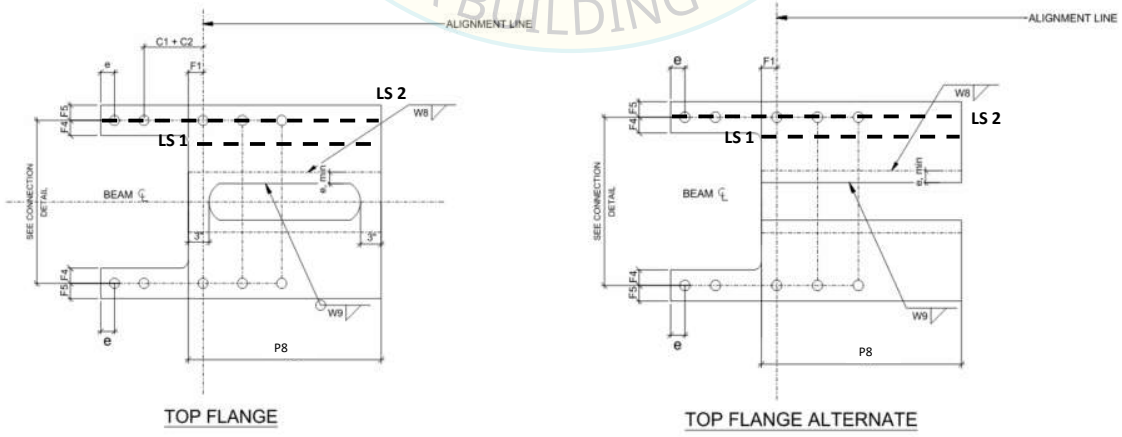
Top plate thickness (see step 2)	*T4* (in)	3
Top plate dimension (see schedule)	*P8* (in)	67
Fuse plate dimension	*F1 (in)*	2.00
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	201.000
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	6030.0
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	6030.0
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.24 **OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Number of oversized bolts in Zone M (see step 5)	n_m	9
Number of oversized bolts on top plate (see step 5)	n_b	19
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.625
Bolt spacing (see schedule)	*s* (in)	3.75
Fuse plate dimension	*F1 (in)*	2.00
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	105.375
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	316.125
Number of oversized bolts on top plate (see step 5)	n_b	19
Net shear area of top plate, $A_g -(T4d_O n_b)$	A_n (in^2)	227.063
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	8855.4
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	6641.6
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.22 **OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	1466.6
Beam flange width	b_{fb} (in)	16.9
Weld 8 size	D_{W8} (1/16 in)	9
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.56
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	65.88
Weld 9 size	D_{W9} (1/16 in)	9
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	61.00
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	11.6
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	12.5
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.92 **OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	27.5
Total number of bolts (see step 5)	n_b	19
Number of bolts in the M region (see step 5)	n_m	9
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)	694.7

Yielding check

Top plate dimension	*P4* (in)	2.25
Top plate dimension	*P5* (in)	4.25
Top plate thickness (see step 2s)	*T4* (in)	3
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	19.500
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	975.0
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	877.5
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.79 **OK**

Rupture check

Top plate dimension	*P4* (in)	2.25
Top plate dimension	*P5* (in)	4.25
Top plate thickness (see step 2)	*T4* (in)	3
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Effective area of rupture, *([P4]+[P5]-d_O-1/16)(T4)*	A_e (in^2)	14.625
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	950.6
Factored tensile rupture capacity, *0.75P_n*	ϕP_n (kips)	713.0
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.97 **OK**

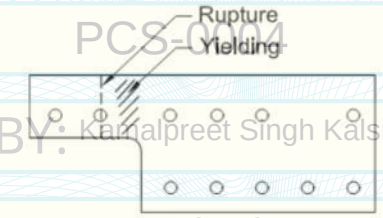


Fig. C-15.9. Tension rupture of net section on top plate.

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

Entering and tightening check

Beam flange fillet dimension	k_1	FALSE
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.5625
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	1.5625
Provided P2 dimension	*P2* (in)	4.0625 **OK**

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	27.5
Top plate thickness (see step 2)	*T4* (in)	3
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	*P10* (in)	12.975
Inside length of the top plate	*P8* (in)	71.50
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop} T4)$	*m* (in)	18.107
Dimension, *([P8] - m) / 2*	*e* (in)	26.697
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), *[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)*	$P2_{req}$ (in)	0.0
Provided P2 dimension	*P2* (in)	4.0625 **OK**

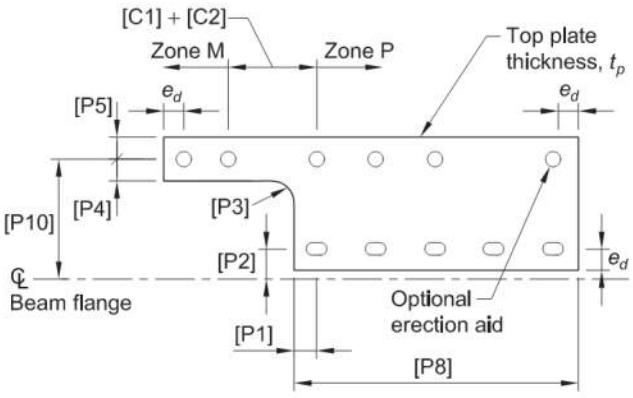


Fig. 15.6. Top plate detail.

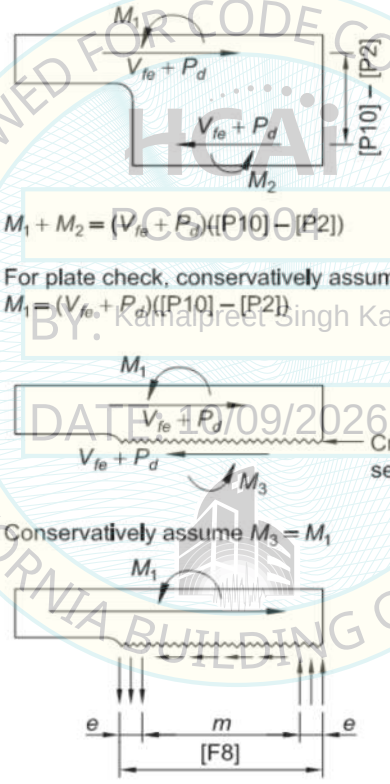


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAI
PCO-0001
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

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

Refer to AISC 358 Fig. C-15.11

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)	3.875	**OK**
Fuse plate thickness (see step 2)	$T2$ (in)	2.25	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	1.7	**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)	1439.1	
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)	$n_{b,\ Bot}$	19	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	9	
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)	1363.4	
Fuse plate dimension, centerline to gage line	$F7$ (in)	4.0625	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	2.7875	
Fuse plate thickness (see step 2)	$T2$ (in)	2.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
Gross area for yielding, $2([F7]+[F8])T2$	A_g (in^2)	30.825	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	1541.3	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	1387.1	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.98	**OK**

Step 18. (continued)

Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Fuse plate dimension	$F7$ (in)	4.0625	
Fuse plate dimension	$F8$ (in)	2.7875	
Fuse plate thickness (see step 2)	$T2$ (in)	2.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.563	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	23.513	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	P_n (kips)	1834.0	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	1375.5	
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	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

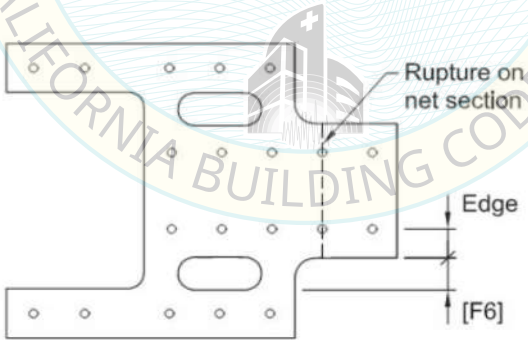


Fig. C-15.11. Limit states related to dimension [F6].

HCAi
PCS-0004
Reviewed By: Ramandeep Singh Kalsi
DATE: 10/09/2026
APPROVED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	3.875	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	134199	
Fuse plate thickness (see step 2)	*T2* (in)	2.25	
Top plate thickness (see step 2)	*T4* (in)	3	
Beam depth	d_b (in)	44	
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42	
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	733.7	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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_n	10	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	345.8	
Resistance factor for bolt slip	ϕ	0.7	
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	363.1	
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	145.2	
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	5.978	
Provided yielding region depth	*F2* (in)	5.875	**OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	793.1	
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	1439.1	
Lower bound ratio	V_{fe}/V_y *LB*	1.7	
Ratio	V_{fe}/V_y	1.8	**OK**

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85	
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]* <= *1.85*	*F6 / F2*	0.65	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1
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)	$n_{b,\ Bot}$	19
Number of bolts in the M region	$n_{m,\ Bot}$	9
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)	1363.4

Yielding check

Fuse plate dimension	*F4* (in)	2.25
Fuse plate dimension	*F5* (in)	4.625
Fuse plate thickness (see step 2)	*T2* (in)	2.25
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Gross area of yielding, *2([F4]+[F5])(T2)*	A_g (in^2)	30.9375
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	R_n (kips)	1546.88
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	1392.19
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.98 **OK**

Rupture check

Fuse plate dimension	*F4* (in)	2.25
Fuse plate dimension	*F5* (in)	4.625
Fuse plate thickness (see step 2)	*T2* (in)	2.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	23.625
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	R_n (kips)	1842.8
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	1382.1
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

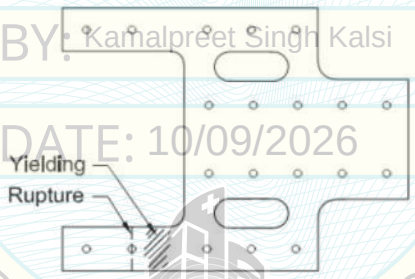


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

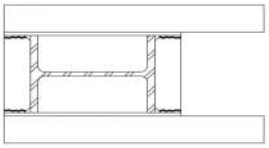
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

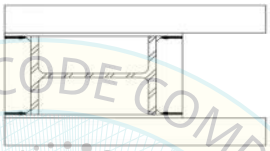
Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, P_d / 2	P_d (kips)	27.5	
External continuity plate dimension	*C5* (in)	3.75	
Column depth	d_c (in)	41.3	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	*C3* (in)	4.25	
Bridge plate thickness (not greater than 2W5)	*T4* (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}\ T4([C3]-1")$	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	*D/C*	N/A	**N/A**



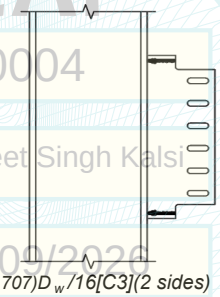
Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



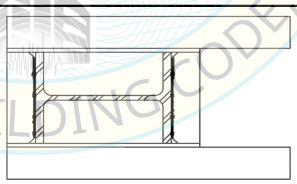
Shear tab to bridge plate weld failure check

Minimum bay width	*B* (in)	420.0	
Column depth	d_c (in)	41.3	
Beam depth	d_b (in)	44	
Shear force from gravity	$V_{gravity}$ (kips)	25.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	134199	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	733.7	
Distance from alignment line to column	*C1* (in)	8.75	
Weld 7 demand, $V_u\ C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
Cover plate overhang	*C3* (in)	4.25	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



Bridge plate to column flange weld failure check

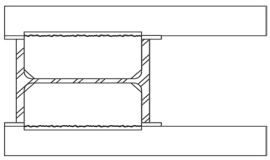
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	*C1* (in)	8.75	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	*C3* (in)	4.25	
Column flange width	b_{fc} (in)	16.2	
Weld 6 effective length, $MIN(b_{fc}$ - 2", 2[C3])	l_{eff} (in)	8.5	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	*D/C*	N/A	**N/A**



*These checks are N/A when bridge plates do not apply

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

Column depth	d_c (in)	41.3	
Column flange thickness	t_{fc} (in)	2.36	
Weld 4 size	D_{W4} (1/16 in)	15	
Weld 4 effective length, d_c - 2 t_{fc}	l_{eff} (in)	36.58	
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)	763.7	
Shear plate thickness, t_{sp} >= *1.1*W4 or 3/8"*	t_{sp} (in)	1.125	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	1234.6	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	*D/C*	0.6	**OK**



*This check is N/A when shear plates do not apply

HCAi
PCS-0004
REVIEWED FOR CODE COMPLIANCE
CBY: Kamalpreet Singh Kalsi
DATE: 10/09/2025
CALIFORNIA BUILDING CODE

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)	49898.203
Column flange width	b_{fc} (in)	16.2
Beam depth	d_b (in)	44
Bay width	B (in)	420.0
Number of bolts in zone M	M	9
Number of bolts in zone P	P	10
Bolt spacing	s (in)	3.75
Cover plate thickness	$T1$ (in)	1.125
Fuse plate thickness	T2 (in)	2.25
Top plate thickness	$T4$ (in)	3
External continuity plate dimension	$C5$ (in)	3.75
Fuse plate dimension	$F2$ (in)	5.875
Fuse plate dimension	$F3$ (in)	25.75
Fuse plate dimension	$F4$ (in)	2.3
Fuse plate dimension	$F6$ (in)	3.875
Fuse plate dimension	$F8$ (in)	2.7875
Distance from beam centerline to bolt line	$P2$ (in)	4.0625
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	4.20E+05
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	7.64E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	1.68E+05
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	3.39E+05
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	8.21E+04
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	3.96E+05
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	1.48E+08
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	6.20E+07
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.42 **OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

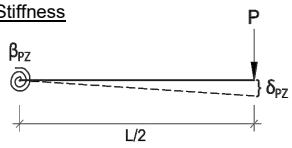
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

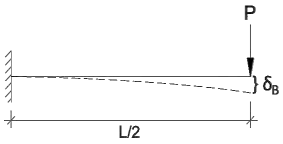


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	44	
Beam flange thickness	t_{fb} (in)	2.875	
Column depth	d_c (in)	41.3	
Column flange width	b_{fc} (in)	16.2	
Column web thickness	t_{wc} (in)	1.34	
Column flange thickness	t_{fc} (in)	2.36	
Cover plate overhang	$C3$ (in)	4.3	
Cover plate thickness	$T1$ (in)	1.125	
Bay width	B (in)	420.0	
Story height	H (in)	192.0	
Panel zone factor, $d_b d_c t_{wc} + 2d_b(d_c + 2[C3])T1$	V_p (in^3)	7365.2	
Column ratio, $(d_c - t_{fc}) / B$	α	0.093	
Beam ratio, $(d_b - t_{fb}) / H$	β	0.214	
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	1.72E+08	
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	1.64E+06	
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	N/A	
Beam moment of inertia	I_{xb} (in^3)	49898.203	
Required panel zone spring stiffness, *60EI$_{xb}$ / B for one-sided, 120EI$_{xb}$ / B for two-sided*	K_{req} (k-in)	N/A	
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	N/A	**N/A**

*This check is N/A when connection stiffness is applied in the model analysis

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

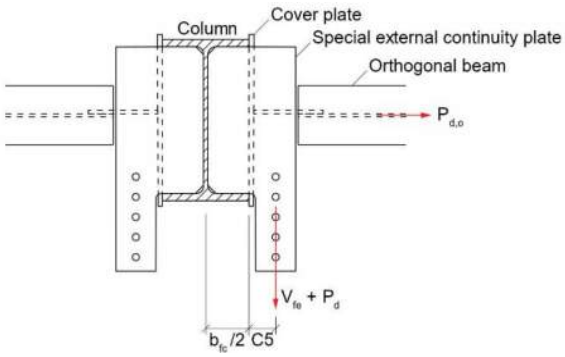
Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

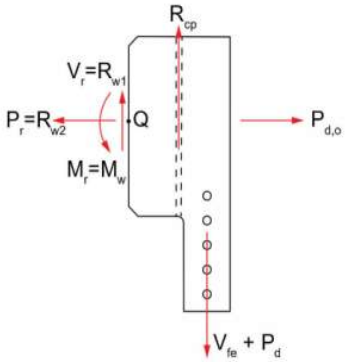
Connection demands (one side)			
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	27.5	
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0	
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0	
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)	0.0	
Column flange width	b_{fc} (in.)	16.2	
Column depth	d_c (in.)	41.3	
Column flange thickness	t_{fc} (in.)	2.36	
Column web thickness	t_{wc} (in.)	1.34	
External continuity plate thickness	$T5$ (in)	3.000	
External continuity plate gage dimension	$C5$ (in.)	3.75	
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A	
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A	
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A	

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	1003.57	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	109.74	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
		Special detailing not required	

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	3.54	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	878.3	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	102.66	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply		Special detailing not required	

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025





Figures 13 and 14 PCS-0004

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth		
Orthogonal beam size		
Orthogonal beam depth	$d_{b,o}$ (in)	
In plane moment frame beam 1		BUW44X1.875X16.9X2.875
In plane moment frame beam 2		NA
Max in-plane moment frame beam depth	d_b (in)	44
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0
New continuity plate required for deeper cantilever beam		N/A
New continuity plate width past column flange	C4 (in)	6.75
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A
Simply supported plate buckling coefficient	k	0.43
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A **N/A**

This check is N/A when OOP forces do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

DuraFuse Frames® Calculations

HCAI PCS SUBMITTAL V4.0

Connection ID DF201

W40X593 Column and BUW44X1.875X16.9X2.875 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
Fuse plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yFuse} (ksi)	50
Top plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yTop} (ksi)	50
External continuity plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yECP} (ksi)	50
Cover plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yCover} (ksi)	50
Shear tab plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{yST} (ksi)	50
Shear plate specified minimum yield stress (2016 AISC Manual Table 2-5)	F_{ySP} (ksi)	50
Bridge plate specified minimum yield stress (2016 AISC Manual Table 2-5)	$F_{yBridge}$ (ksi)	50
	ONLY ONE PLATE GRADE APPLIES.	**OK**
Fuse plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uFuse} (ksi)	65
Top plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uTop} (ksi)	65
External continuity plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uECP} (ksi)	65
Cover plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uCover} (ksi)	65
Shear tab plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uST} (ksi)	65
Shear plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	F_{uSP} (ksi)	65
Bridge plate specified minimum tensile stress (2016 AISC Manual Table 2-5)	$F_{uBridge}$ (ksi)	65
	ONLY ONE PLATE GRADE APPLIES.	**OK**
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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS#004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Beam limitation checks			
Beam size		BUW44X1.875X16.9X2.875	
Beam depth limit (DuraFuse Frames HCAI Design Guide 3.5.1(2))	$d_{b,limit}$ (in.)	44	
Beam depth	d_b (in.)	44	**OK**
Beam weight limit (DuraFuse Frames HCAI Design Guide 1.e)	$W_{b,limit}$ (lb/ft)	575	
Beam weight	W_b (lb/ft)	574.7	**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}	20.4	**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}	2.9	**OK**
Beam lateral bracing limit (AISC 358 15.3.1(5))	*See bracing calculations*		

Column limitation checks			
Column size		W40X593	
Column depth limit (AISC 358 15.3.2(2))	$d_{c,limit}$ (in.)	44	
Column depth	d_c (in.)	43	
Column weight limit (AISC 358 15.3.2(3))	$W_{c,limit}$ (lb/ft)	No Limit	
Column weight	W_c (lb/ft)	593	**OK**
Column flange thickness limit (AISC 358 15.3.2(4))	$t_{cf,limit}$ (in)	No Limit	
Column flange thickness	t_{cf} (in)	3.23	**OK**
Axial force on the column (provided by EOR)	P_u *(kips)*	1500.0	
Gross area of the column section	A_g (in^2)	174	
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) 22 Code	$\lambda_{hd,web}$	NA	
Column web width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1) 16 Code	$\lambda_{hd,web}$	50.6	
Column web width-to-thickness ratio	λ_{web}	19.1	**OK**
Column flange width-to-thickness ratio limit (AISC 358 15.3.2(5), AISC 341 Table D1.1),	$\lambda_{hd,flange}$	7.35	
Factor based on code and member type, $F(E/(R_{yc}F_{yc}))^{0.5}$	F	0.32	
Column flange width-to-thickness ratio	λ_{flange}	2.58	**OK**

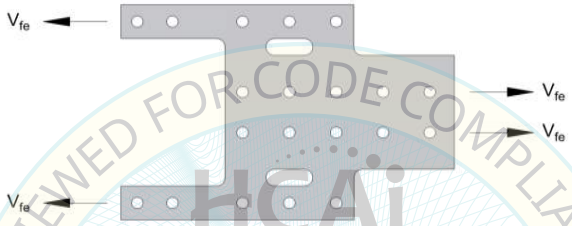
Column-beam moment ratios (AISC 358 15.4(a))			
Column plastic section modulus	Z_c (in^3)	2760	
Column design axial load	P_u (kips)	1500.0	
Column area	A_{gc} (in^2)	174	
Story Height	H (in)	192.0	
Column depth	d_c (in)	43	
Beam depth	d_b (in)	44	
Cover plate dimension, vertical extension outside of panel zone	$C6$ (in)	3	
Sum of column flexural strengths, projected to beam center line (AISC 358 15.4a), $\Sigma Z_c*(F_{yc}-P_u/A_{gc})*(H/2)/(H/2-d_b/2-d_c/4-[C6])$	ΣM^*_{pc} (k-in)	3.64E+05	
Beam plastic section modulus	Z_b (in^3)	2683.9715	
Minimum bay width	B (in)	420.0	
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	134199	
Clear distance between column faces, $B-d_c$	L_h (in)	377.0	
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)	25.0	
Beam shear (AISC 358 15.4a), $2M_{pr}/L_h + V_g$	V_b (kips)	736.9	
Increase in moment from beam shear, $V_b(d_c/2)$	M_{uv} (k-in)	15844	
Sum of beam strengths, projected to column centerline (AISC 358 15.4a), $\Sigma(M_{pr} + M_{uv})$	ΣM^*_{pb} (k-in)	3.00E+05	
Column-beam moment ratio, $\Sigma M^*_{pc} / \Sigma M^*_{pb}$	*Ratio > 1.0*	1.21	**OK**

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

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

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)	2683.9715	
Beam plastic bending moment , $F_{yb}Z_b$	M_p (k-in)	134199	
Maximum probable moment at the fuse location (AISC 358 15.6), $M_u < M_{pr} <= M_p$	M_{pr} (k-in)	134199	**OK**
Fuse plate thickness	$T2$ (in)	2.25	
Fuse plate specified maximum thickness	$T2_{max}$ (in)	2.25	**OK**
Top plate thickness	$T4$ (in)	3	
Top plate specified maximum thickness	$T4_{max}$ (in)	3	**OK**
External continuity plate thickness	$T5$ (in)	3	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
Beam depth	d_b (in)	44	
Maximum probable force at beam flange level per bolt line, $M_{pr} / (2(d_b + T2/2 + T4/2))$	V_{fe} (kips)	1439.1	
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 $M_{cantilever} / (2(d_b + T2/2 + T4/2))$	V_{fe2} (kips)	1439.1	
Sum of maximum probable force per bolt line in connection, $V_{fe} + V_{fe2}$	ΣV_{fe} (kips)	2878.3	



REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAI
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

Step 3. Design the cover plates

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	2878.3	
Beam area	A_{gb} (in^2)	168.9	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per cover plate, $P_{d,total}/2$	P_d (kips)	27.5	
Required cover plate horizontal shear strength (AISC 358 Eq. 15.6-2), $\Sigma V_{fe} + P_d$	$R_{u,horiz}$ (kip)	2905.8	
Required vertical shear from orthogonal gravity beam (from EOR)	$V_{grav\text{-}ortho}$ (kips)	25	
Fuse plate thickness	$T2$ (in)	2.25	
Top plate thickness	$T4$ (in)	3	
External continuity plate thickness	$T5$ (in)	3	
Beam depth	d_b (in)	44	
Column depth	d_c (in)	43	
Required cover plate vertical shear strength, $\Sigma V_{fe}(d_b + T2/2 + T4/2) / d_c + P_d(d_b + T2/2 + T4/2) / 2d_c + V_{grav\text{-}ortho}/2$	$R_{u,vert}$ (kips)	3148.3	

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

Cover plate thickness	$T1$ (in)	2.25	
Cover plate overhange dimension	$C3$ (in)	6	
Column depth	d_c (in)	43	
Cover plate horizontal dimension, $d_c + 2[C3]$	d_{cp} (in)	55	
Cover plate nominal horizontal capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1d_{cp}$	$R_{n,horiz}$ (kips)	3712.5	
Cover plate factored horizontal capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,horiz}$ (kips)	3712.5	
Shear yielding demand/capacity ratio, $R_{u,horiz} / \phi R_{n,horiz}$	D/C	0.78	**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, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2*T5$ for cap plate and ortho. cant./drag cond	h_{cp} (in)	50.00	
Cover plate nominal vertical capacity (AISC 360 Eq. J4-3), $0.6F_{yCover}T1h_{cp}$	$R_{n,vert}$ (kips)	3375.0	
Cover plate factored vertical capacity (AISC 360 Eq. J4-3), $1.0R_n$	$\phi R_{n,vert}$ (kips)	3375.0	
Shear yielding demand/capacity ratio, $R_{u,vert} / \phi R_{n,vert}$	D/C	0.93	**OK**

Step 3. (continued)

Cover plate thickness check (AISC 341 E3.6e.2)			
Beam depth	d_b (in)	44	
Beam flange thickness	t_{fb} (in)	2.875	
Column flange thickness	t_{fc} (in)	3.23	
Panel zone vertical dimension, $d_b - 2t_{fb}$	d_z (in)	38.25	
Panel zone horizontal diemension, $d_c - 2t_{fc}$	w_z (in)	36.54	
Required cover plate thickness (AISC 358 Eq. 15.6-3), $(d_z + w_z)/90$	t_{req} (in)	0.831	
Thickness check, $T1 / t_{req}$	$T1 / t_{req} > 1.0$	2.71	**OK**

Step 4. Determine the maximum bolt diameter

Bolt diameter	d_{bolt} (in)	1.25	
Beam flange standard hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Beam plastic section modulus	Z_b (in^3)	2683.9715	
Beam flange thickness	t_{fb} (in)	2.875	
Bolt hole area in section, $(d_{hole,std} + 1/8)t_{fb}$	$A_{bolthole}$ (in^2)	4.313	
Bolt hole plastic section modulus, $4A_{bolthole}(d_b - t_{fb})/2$	$Z_{boltholes}$ (in3)	354.7	
Beam plastic section modulus of the net section, $Z_b - Z_{bolt\ holes}$	$Z_{b,net}$	2329.3	
Beam yield moment, $Z_b R_{yb} F_{yb}$	M_{pe} (k-in)	147618	
Beam fracture moment, $Z_{b,net} R_{tb} F_{ub}$	M_{fr} (k-in)	166543	
Beam Yield / Fracture Ratio	M_{pe} / M_{fr}	0.89	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2 / 4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	77.3	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	19.0	
Number of bolts per line (AISC 358 15.2(1))	n_b	19	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	3.75	
Top plate thickness	$T4$ (in)	3	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uTop} R_{tp} T4)]$	$F2_a$ (in)	5.125	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	3.6001	
Number of bolts on- and ahead-of the alignment line (zone P)	n_p	11	**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	8	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	1466.6	
Beam flange thickness	t_{fb} (in)	2.875	
Top plate thickness	$T4$ (in)	3	
Governing material thickness, $\min(t_{fb}, T4)$	t_{min} (in)	2.9	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	10651.9	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.2	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	8829.8	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	217.2	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	9047.1	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1958.6	
Controlling strength, $\min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1958.6	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	1468.9	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	1.00	**OK**

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/03/2026
CERTIFIED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 5a. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	0	
Fuse plate thickness	$T2$ (in)	2	
Top plate thickness	$T4$ (in)	3	
Bolt group demand from wind per line, $M_u/(2(d_b + T2/2 + T4/2))$	R_u *(kip)*	0.0	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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	19	
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)	657.0	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	558.4	
Wind slip demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.00	**OK**

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1439.1	
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, $P_{d,total}/2$	P_d (kips)	0.0	
Bolt diameter	d_{bolt} (in)	1.25	
Area of single bolt, $\pi * d_{bolt}^2/4$	A_{bolt} (in^2)	1.227	
Bolt shear strength, $\phi F_{nv} A_b$	ϕr_n (kips)	103.1	
Required number of bolts required for bolt shear, roundup, $(V_{fe} + P_d)/\phi r_n$	$n_{b,req}$	14.0	
Number of bolts per line on bottom flange (AISC 358 15.2(1))	$n_{b,Bot}$	19	**OK**
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.75	
Bolt spacing (see schedule)	s (in)	3.75	
Fuse plate thickness	$T2$ (in)	2.25	
Approximate longitudinal dimension of each fuse region on the plate (AISC 358 Eq. 15.6-7), $V_{fe}/[2(0.6F_{uFuse} R_{tp} T2)]$	$F2_a$ (in)	6.833	
Minimum number of bolts on- and ahead-of the alignment line (AISC 358 Eq. 15.6-6), $[2[F2_a]+3in - 2e]/s + 1$	$n_{p,min}$	4.5	
Number of bolts on- and ahead-of the alignment line on bottom flange (zone P)	$n_{p,Bot}$	11	**OK**
Number of bolts behind the alignment line on the outside lines on bottom flange (zone M) (AISC 358 Eq. 15.6-8), nb - np	$n_{m,Bot}$	8	

Flange bolt limit states check			
Total force on bolts per line, $V_{fe} + P_d$	R_u (kips)	1439.1	
Beam flange thickness	t_{fb} (in)	2.875	
Fuse plate thickness	$T2$ (in)	2.25	
Governing material thickness, min$(t_{fb}, T2)$	t_{min} (in)	2.3	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4 d_{bolt} t_{min} F_{u,min} n_b$	R_{nb} (kips)	10651.9	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
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)	2.2	
Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1} t_{min} F_{u,min} (n_b - 1)$	R_{nt1} (kips)	8829.8	
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, $e - d_O / 2$	l_{c2} (in)	1.0	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2} t_{min} F_{u,min}$	R_{nt2} (kips)	170.0	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	8999.9	
Nominal shear strength of all bolts per line, $F_{nv} A_{bolt} n_b$	R_{nv} (kips)	1958.6	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1958.6	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	1468.9	
Beam flange bolt shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.98	**OK**

REVIEWED FOR CODE COMPLIANCE
CAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 5b. (continued)

Bolt slip for wind load check			
Connection flexural demand from wind load combination (from EOR)	M_u (k-in)	0	
Bolt group demand from wind per line, $M_u/(2(d_b+T2/2 + T4/2))$	R_u *(kip)*	0.0	
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102	
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 on the bottom flange	$n_{b,Bot}$	19	
Resistance factor for bolt slip	ϕ	0.85	
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_{b,Bot}$	R_n (kips)	657.0	
Total factored bolt slip resistance, $0.85R_n$	ϕR_n (kips)	558.4	
Wind slip demand/capacity ratio, R_u / ϕR_n	*D/C*	0.00	**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)	44	
Minimum edge distance for bolt in oversized hole (AISC 360 Table J3.4)	*e* (in)	1.75	
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), $\gamma_{max} d_b + e + \alpha[C3]$	$C1_{req}$ (in)	10.39	
Distance from alignment line to column	*C1* (in)	11.625	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 7. Size the cover plate to column flange weld (W1) (refer to AISC 358 Fig. C-15.4)

Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} (kips)	2878.3	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per weld, $P_{d,total}/2$	P_d (kips)	27.5	
Load from perpendicular gravity beam to cover plate (input to ICOR calculation)	$V_{grav\text{-}ortho}$ (kips)	25	
Column depth	d_c (in)	43	
Column flange thickness	t_{fc} (in)	3.23	
Beam depth	d_b (in)	44	
Fuse plate thickness	$T2$ (in)	2.25	
Top plate thickness	$T4$ (in)	3	
External continuity plate thickness	$T5$ (in)	3	
Cover plate thickness	$T1$ (in)	2.25	
Cover plate dimension, vertical extension outside panel zone	$C6$ (in)	3	
Cover plate height, $d_b + 2[C6]$, $d_b + [C6] - T5$ or just $d_b - 2T5$ for cap plate and ortho. cant./drag condit	h_{cp} (in)	50	
Gage dimension from the cover plate to the line of bolts on the external continuity plates	$C5$ (in)	4	
Weld 1 required holdback, each end (AISC 358 15.5.1)	l_{hb1} (in)	2.0000	
Weld 1 length, $h_{cp} - 2l_{hb1}$	l_{W1} (in)	46.00	
Weld 1 effective length for resisting orthogonal demands (AISC 358 Eq. 15.6-14), $T5 + T1 + [C6]$	l_{We1} (in)	8.250	
Weld 1 size	D_{W1} (1/16 in)	30	
Weld 1 minimum	D_{W1min} (1/16 in)	3	**OK**
Weld 4 size (weld 4 used if D_{W1} exceeds 12, 0 indicates shear plates are not used)	D_{W4} (1/16 in)	22	
Weld 4 minimum (when shear plates are required)	D_{W4} (1/16 in)	3	**OK**
Weld 4 length (0 indicates shear plates are not used), $d_c - 2t_{fc}$	l_{W4} (in)	36.54	
Horizontal distance from center of rotation to geometric center	e_x (in)	0.16	
Vertical distance from center of rotation to geometric center	e_y (in)	0.13	

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.

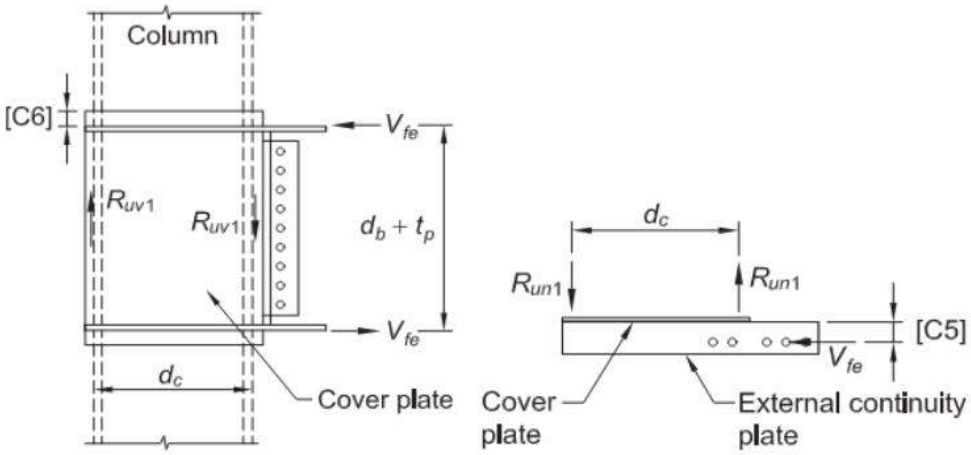
Summation of nodal moments about center of rotation, $(\Sigma V_{fe} + P_d)*L$	ΣRL (k-in)	232937	

Sample Node Point	L_R (in)	R (k)	θ_r (deg)	$R*L_R$ (k-in)
W1RT	31.13	66.91	46.73	2082.9
W4TL	30.75	34.82	35.99	1070.6
W1LB	31.17	66.63	45.97	2077.0
W4BR	30.35	34.96	35.77	1061.0
W1RC	21.34	43.04	1.58	918.7
W4TC	24.88	27.55	1.22	685.5
W1LC	21.67	43.17	1.55	935.6
W4BC	24.63	27.50	1.23	677.3
		66.91	46.73	

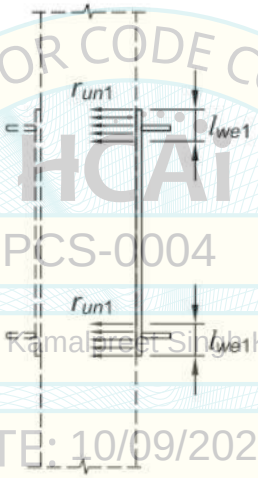
Weld strength check according to AISC 360 Chapter J

Force on Critical Weld Segment	R_{crit} (kips)	66.9	
Orientation of Resultant Force	θ_r (degrees)	46.7	
Length of weld segments for Instantaneous Center of Rotation method	l_{seg} (in)	0.92	
In-plane shear demand on weld, $(R_{crit}/l_{seg})((\Sigma V_{fe}+P_d/2)*(d_b+T2/2+T4/2)/\Sigma RL)$	r_{uv} (kip/in)	42.1	
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)	42.1	
Ratio of critical element deformation to its deformation at the maximum stress	p_i (in)	1.18	
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)	54.55	
Cover plate to flange weld tearing check, $r_u / \phi r_n$	D/C	0.77	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/03/2026
CALIFORNIA BUILDING CODE, 2025



(a) In-plane *(b) Out-of-plane*



(c) Out-of-plane distributed

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

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 8. Size the external continuity plate-to-cover plate weld (W2) (refer to AISC 358 Fig. C-15.5)

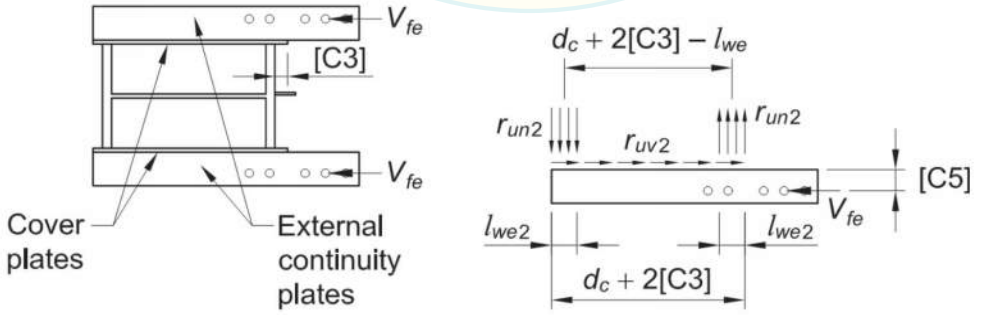
Sum of maximum probable force per bolt line in connection (see step 2)	ΣV_{fe} *(kips)*	2878.3	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5	
Column depth	d_c (in)	43	
Column flange thickness	t_{fc} (in)	3.23	
Cover plate overhang dimension	*C3* (in)	6	
External continuity plate dimension	*C5* (in)	4	
Weld 2 size	D_{W2} (1/16 in)	22	
Weld 2 holdback required, each end (AISC 358 15.5(1)), $D_{W2}/16$	l_{hb2} (in)	1.38	
Weld 2 effective length for normal demands (AISC 358 Eq. 15.6-17), $t_{fc} + T1 + [C3] - l_{hb2}$	$l_{we2,eff}$ (in)	10.11	
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)	52.8	
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)	27.3	
Resultant demand on weld 2 at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kip/in)	59.5	
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)	61.2	
External continuity plate-to-cover plate weld tearing check, $r_u / \phi r_n$	*D/C*	0.97	**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)*	2878.3	
External continiuty plate moment demand, $(\Sigma V_{fe} + P_d)[C5]$	M_u (kips)	11623.0	
Weld 2 total demand, $\Sigma V_{fe} + P_d$	R_u (kips)	2905.8	
External continuity plate thickness (see step 2)	*T5* (in)	3.000	
External continuity plate specified maximum thickness	$T5_{max}$ (in)	3	**OK**
External continiuty plate shear capacity (AISC 360 Eq. J4-3), $(1.0)(0.6)F_{yECP}(d_c + 2[C3])T5$	ϕR_n (kips)	4950.0	
External continiuty plate moment capacity (AISC 360 Section J4.5), $0.9F_{yECP}\,T5((d_c + 2*[C3])/2)^2$	ϕM_n (kips)	102093.8	
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)*	2878.3	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5	
Column fillet dimension	*k* (in)	4.4	
Column fillet dimension	*k1* (in)	2.1	
Weld 2 length oriented longitudinally for loading, $2(d_c + 2[C3]-2*l_{hb2}) + 2*(d_c - 2k)$	$l_{w2,l}$ (in)	172.9	
Column flange width	b_{fc} (in)	16.7	
Weld 2 length oriented transverse to loading, $2*b_{fc} + 2*(b_{fc}-2k1)$	$l_{w2,t}$ (in)	58.3	
Weld 2 size	D_{W2} (1/16 in)	22	
Nominal strength of longitudinally loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{w2,l}$	R_{nwl} (kips)	7057.7	
Nominal strength of transverse loaded welds (AISC 360 Table J2.5), $0.6F_{EXX}(0.707D_{W2}/16)l_{W2,t}$	R_{nwt} (kips)	2380.3	
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)	9569.6	
Factored capacity of weld, $0.75R_n$	ϕR_n (kips)	7177.2	
Weld 2 demand/capacity check for cap plate configuration, $R_u / \phi R_n$	*D/C*	0.40	**OK**



(a) Applied loads *(b) Distributed shear and normal demands*

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

HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

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

Refer to AISC 358 Fig. C-15.6, 7

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

Maximum probable force per bolt line (see step 2)	V_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5	
Number of bolts ahead of rupture, $n_b - n_m$ or if C2<0 & abs(C2)>C3 then *round up((abs(C2)-C3)/s) + n_p*	n_{rup1}	12	
Mode 1 bolt rupture demand, $(V_{fe} + P_d)n_{rup1}/n_b$	P_u (kips)	926.3	
External continuity plate thickness (see step 2)	*T5* (in)	3	
External continuity plate width	*C4* (in)	8.625	
Diameter of standard bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Net area of external continuity plate cross section, $T5([C4]-(d_{hole,std}+1/16))$	A_n (in^2)	21.563	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	1401.6	
Maximum factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	1051.2	
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, $n_{rup1} + 1$	n_{rup}	13	
Mode 2 bolt rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	1003.5	
External continuity plate dimension	*C2 (in)*	-7.875	
Cover plate overhang dimension	*C3* (in)	6	
External continuity plate dimension	*C5* (in)	4	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	-1.875	
External continuity plate thickness (see step 2)	*T5* (in)	3	
External continuity plate width	*C4* (in)	8.625	
Diagonal bonus (AISC 360 B4.3b), $[C9]^2(T5)/4[C5]$	*Bonus* (in)	0.659	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Area of external continuity plate cross section with bonus, $T5([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	22.222	
Nominal tensile rupture capacity, $F_{uECP}A_n$	P_n (kips)	1444.4	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	1083.3	
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, $n_{rup1} + 2$	n_{rup}	14	
Mode 3 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	P_u (kips)	1080.7	
Bolt spacing (see schedule)	*s* (in)	3.75	
External continuity plate dimension	*C2 (in)*	-7.875	
Cover plate overhang dimension	*C3* (in)	6	
External continuity plate dimension	*C5* (in)	4	
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	1.875	
External continuity plate thickness (see step 2)	*T5* (in)	3	
External continuity plate width	*C4* (in)	8.625	
Diagonal bonus (AISC 360 B4.3b), $[C7]^2(T5)/(4[C5])$	*Bonus* (in)	0.659	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Area of external continuity plate cross section with bonus, $(T5)([C4]-d_{hole,std}-1/16) + Bonus$	A_n (in^2)	22.222	
Nominal tensile rupture capacity, $A_n F_{uECP}$	P_n (kips)	1444.4	
Factored tensile rupture capacity, $0.75P_n$	ϕP_n (kips)	1083.3	
Mode 3 rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	1.00	**OK**

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

Number of bolts ahead of rupture, $n_{rup1} + 1$	n_{rup}	13	
Mode 4 rupture demand, $(V_{fe} + P_d)n_{rup}/n_b$	R_u (kips)	1003.5	
External continuity plate width	*C4* (in)	8.625	
External continuity plate dimension, *[C2] + [C3]*	*C9* (in)	-1.875	
External continuity plate thickness (see step 2)	*T5* (in)	3	
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375	
Gross tension area, *T5[C4]*	A_{gt} (in^2)	25.875	
Gross shear area, *T5[C9]*	A_{gv} (in^2)	5.625	
Net tension area, $A_{gt} - ((T5)*(d_{hole,std}+1/16))$	A_{nt} (in^2)	21.563	
Net shear area, A_{gv}	A_{nv} (in^2)	5.625	
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{up}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	1570.3	
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	1177.7	
Mode 4 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.85	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Ramandeep Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 9. (continued)

Mode 5: Block shear through the second bolt hole in the direction of the column		
Number of bolts ahead of rupture, $n_{rup1} + 2$	n_{rup}	14
Mode 5 rupture demand, $(V_{fe} + P_d)*n_{rup}/n_b$	R_u (kips)	1080.7
Bolt spacing (see schedule)	s (in)	3.75
External continuity plate dimension	*C2 (in)*	-7.875
Cover plate overhang dimension	*C3* (in)	6
External continuity plate width	*C4* (in)	8.625
External continuity plate dimension, *[C2] + [C3] + s*	*C7* (in)	1.875
External continuity plate thickness (see step 2)	*T5* (in)	3
Diameter of bolt holes (see step 4)	$d_{hole,std}$ (in)	1.375
Gross tension area, *T5[C4]*	A_{gt} (in^2)	25.875
Gross shear area, *T5[C7]*	A_{gv} (in^2)	5.625
Net tension area, A_{gt}*-((T4)(d_{hole}+1/16))*	A_{nt} (in^2)	21.563
Net shear area, A_{gv}	A_{nv} (in^2)	5.625
Nominal block shear capacity (AISC 360 Eq. J4-5), $0.6F_{uECP}A_{nv} + F_{uECP}A_{nt} < 0.6F_{yECP}A_{gv} + F_{uECP}A_{nt}$	R_n (kips)	1570.3
Factored block shear capacity, $0.75R_n$	ϕR_n (kips)	1177.7
Mode 5 block shear demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.92 **OK**

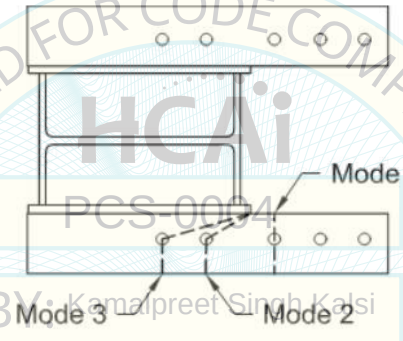


Fig. C-15.6. Tension rupture limit state for external continuity plate.

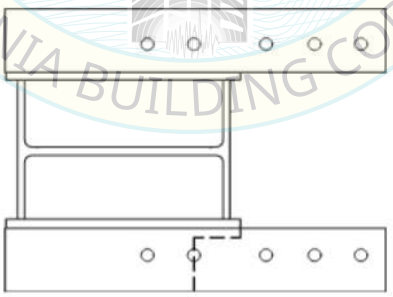


Fig. C-15.7. Block shear limit state for external continuity plate.

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Minimum bay width	B (in)	420.0	
Shear force from gravity (from EOR)	$V_{gravity}$ (kips)	25.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	134199	
Beam shear demand, $(2M_{pr}/(B-d_c))+V_{gravity}$	V_u (kips)	736.9	
Beam depth	d_b (in)	44	
Beam web thickness	t_{wb} (in)	1.875	
Beam web area, $d_b t_{wb}$	A_w (in)	82.5	
Beam web width-to-thickness ratio, h / t_w	l_{web}	20.4	
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)	2475.0	
Beam shear strength demand/capacity ratio, $V_u / \phi V_n$	D/C	0.30	**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)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_{d,total}/2$	P_d (kips)	27.5	
Total force on beam flange, $2(V_{fe} + P_d)$	R_u (kips)	2933.3	
Beam flange thickness	t_{fb} (in)	2.875	
Beam flange width	b_{fb} (in)	16.9	
Diameter of standard bolt holes in beam flange (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Number of holes along each bolt line (see step 5)	n_b	19	
Length of connection on beam	$B4$ (in)	69.25	
Distance from beam centerline to bolt line	$P2$ (in)	4.0625	
Gross tension area, $(b_{fb} -2[P2])t_{fb}$	A_{gt} (in^2)	25.228	
Gross shear area, $2t_{fb}[B4]$	A_{gv} (in^2)	398.188	
Net tension area, $A_{gt} -(t_{fb}(d_{hole,std}+1/16))$	A_{nt} (in^2)	21.095	
Net shear area, $A_{gv} -(t_{fb}(d_{hole,std}+1/16)(2n_b-1))$	A_{nv} (in^2)	245.273	
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)	10936.9	
Factored rupture capacity, $0.75R_n$	ϕR_n (kips)	8202.6	
Demand/capacity ratio, $R_u / \phi R_n$	D/C	0.36	**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)	736.9	
Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Area of single bolt, $\pi d_{bolt}^2/4$	A_{bolt} (in^2)	1.227	
Nominal shear capacity of a single bolt, $F_{nv}A_{bolt}$	r_n (kips)	103.1	
Factored shear capacity of a single bolt, $0.75r_n$	ϕr_n (kips)	77.3	
Required number of shear tab bolts (AISC 358 Eq. 15.6-21), $R_u / \phi r_n$	$n_{n,req}$	10	
Provided number of shear tab bolts	n_n	10	**OK**

Shear tab geometry requirements

Beam T dimension limit	T_{beam} (in)	34.5	
Required length of shear tab (AISC 358 Eq. 15.6-22), T_{beam} - *1in*	$l_{tab,req}$ (in)	34	
Length of shear tab (see drawings), T_{beam} - *1in*	l_{tab} (in)	33.5	**OK**

Web bolt limit states check

Beam web thickness	t_{wb} (in)	1.875	
Shear tab provided thickness	$T3$ (in)	1.5	
Governing material thickness, $min(t_{wb}, T3)$	t_{min} (in)	1.5	
Bearing strength at bolt holes (AISC 360 Eq. J3-6a), $2.4d_{bolt}t_{min}F_{u,min}n_n$	R_{nb} (kips)	2925.0	
Diameter of standard bolt hole (AISC 360 Table J3.3)	$d_{hole,std}$ (in)	1.375	
Edge distance on shear tab (AISC 360 Table J3.4)	e (in)	1.625	
Bolt spacing (based on provided shear tab bolts, n_n, and shear tab geometry, l_{tab})	s (in)	3.361	
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)	2.0	

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY • Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

Step 12. (continued)

Tearout strength at inner bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c1}\ t_{min}\ F_{u,min}\ (n_n - 1)$	R_{nt1} (kips)	2091.4	
Clear distance, parallel to applied force, between the edge of the hole and the edge of the material, $e - d_{hole,std} / 2$	l_{c2} (in)	0.9	
Tearout strength at edge bolt holes (AISC 360 Eq. J3-6c), $1.2 l_{c2}\ t_{min}\ F_{u,min}$	R_{nt2} (kips)	109.7	
Total tearout strength, $R_{nt1} + R_{nt2}$	R_{nt} (kips)	2201.1	
Nominal shear strength of all bolts, $F_{nv} A_{bolt} n_n$	R_{nv} (kips)	1030.8	
Controlling strength, $min(R_{nb}, R_{nt}, R_{nv})$	R_n (kips)	1030.8	
Factored shear capacity of all bolts, $0.75 * R_n$	ϕR_n (kips)	773.1	
Shear tab bolt shear demand/capacity ratio, $R_u / \phi R_n$	D/C	0.95	**OK**

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

Minimum fastener tension (AISC 360 Table J3.1)	T_b	102	
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), $1.5 T_b \mu D_u h_f n_s$	F_b (kips)	51.9	
Provided number of shear tab bolts (see step 12)	n_n	10	
Shear tab required normal strength (AISC 358 Eq. 15.6-23), $n_n F_b$	P_u (kips)	518.7	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	736.9	
Distance from alignment line to column	$C1$ (in)	11.625	
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)	8566.8	

Shear tab rupture through net section check

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Slotted bolt hole width (standard slot width)	w_b (in)	1.375	
Shear tab provided thickness	$T3$ (in)	1.5	
Shear tab length (see step 12)	l_{tab} (in)	34	
Beam T dimension limit	T_{beam} (in)	Other	
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	e (in)	1.625	
Slotted bolt hole spacing, $(T_{beam} - 1" - 2e_d) / (n_n - 1)$	s_{slot} (in)	3.3125	
Gross area of shear tab, $T3 l_{tab}$	A_{gv} (in^2)	50.25	
Shear tab net area, $A_{gv} - n_n (w_b + 1/16) T3$	A_{nv} (in^2)	28.69	
Effective net area, $A_e = A_{nv}$	A_e (in^2)	28.69	
Shear tab factored capacity for shear rupture (AISC 360 Eq. J4-4), $(0.75) 0.6 F_{uST} A_{nv}$	ϕR_{nv} (kips)	839.11	
Shear tab factored capacity for tensile rupture (AISC 360 Eq. J4-2), $(0.75) F_{uST} A_e$	ϕR_{nn} (kips)	1398.52	
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	0.73	**OK**

Shear tab yielding check

Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	736.93	
Required axial strength of the shear tab (see above)	P_u (kips)	518.67	
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)	8566.80	
Shear tab gross area, $T3 l_{tab}$	A_{gv} (in^2)	50.25	
Shear tab plastic section modulus, $(l_{tab}/2)^2 T3$	Z_{tab} (in^3)	420.84	
Shear tab factored capacity for shear yielding (AISC 360 Eq. J4-3), $(1.0) 0.6 F_{yST} A_{gv}$	ϕR_{nv} (kips)	1507.50	
Shear tab factored capacity for tensile yielding (AISC 360 Eq. J4-1), $(0.9) F_{yST} A_{gv}$	ϕR_{nn} (kips)	2261.25	
Shear tab factored capacity for flexural capacity, (AISC 360 Eq. F2-1), $(0.9) F_{yST} Z_{tab}$	ϕM_n (kips)	18937.97	
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	0.56	**OK**

REVIEWED FOR CODE COMPLIANCE
HCAI
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE 2025

Step 13. (continued)

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 - 2T5 (see drawings detail 3/SDF-04)	l_{w3} (in)	33.5	
Required shear strength of the shear tab and the beam (see step 10)	V_u (kips)	736.9	
Weld 3 shear demand, V_u / l_{tab}	r_{uv} (k/in)	22.0	
Number of slotted bolt holes in shear tab	n_{vb}	10	
Slotted bolt hole spacing	s_{slot} (in)	3.3125	
Distance from line of bolts to shear tab weld	$C1$ (in)	11.625	
Size of weld in 1/16^{ths} of an inch	D_{W3}	24	
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)	8566.8	
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)	60.8	
Resultant weld demand at critical location, $\sqrt{r_{uv}^2 + r_{un}^2}$	r_u (kips)	64.6	
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)	66.8	
Shear tab demand/capacity ratio, $r_u / \phi r_n$	D/C	0.97	**OK**

This check is N/A when bridge plates do not apply

Step 14. Determine the shear tab slot dimensions

Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Minimum edge distance for slotted holes (AISC 360 Table J3.4, J3.5)	e (in)	1.75	
Beam depth	d_b (in)	44	
Length of the shear tab (see step 12)	l_{tab} (in)	33.5	
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), $\gamma_{max}(d_b/2 + l_{tab}/2 - e) + d_{bolt}/2$	$S1_{req}$ (in)	2.845	
Provided inside shear tab slot dimension (AISC 358 Eq. 15.6-26)	$S1$ (in)	2.875	**OK**
Provided outside shear tab slot dimension, $[S2] = [S1]$	$S2$ (in)	2.875	

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

Refer to AISC 358 Fig. C-15.8

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	27.5	
Required shear strength of top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1466.6	

Yielding check			
Top plate thickness (see step 2)	$T4$ (in)	3	
Top plate dimension (see schedule)	$P8$ (in)	71.5	
Gross shear area of top plate, $(P8)(T4)$	A_g (in^2)	214.500	
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	6435.0	
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	6435.0	
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	D/C	0.23	**OK**

Rupture check			
Bolt diameter (see step 4)	d_{bolt} (in)	1.25	
Length of short slotted bolt holes (AISC 360 Table J3.3)	l_s (in)	1.625	
Number of short slotted bolts on top plate (see step 5)	n_b	19	
Net shear area of top plate, $A_g - (T4 l_s n_b)$	A_n (in^2)	121.875	
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_n F_{uTop}$	R_n (kips)	4753.1	
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	3564.8	
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	D/C	0.41	**OK**

Note: For the alternate top plate (applicable only to HCAI projects), this check is conservative and does not need to be checked for the top plate.

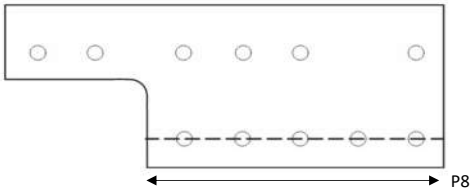


Fig. C-15.8. Shear rupture of net section on top plate.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 15 Alternate. Check the top plate for tensile yield and tensile rupture (AISC *Specification* Chapter J)

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	27.5
Required tensile strength of the alternate top plates (AISC 358 Eq. 15.6-27), $V_{fe} + P_d$	R_u (kips)	1466.6

Yielding check (LS 1)

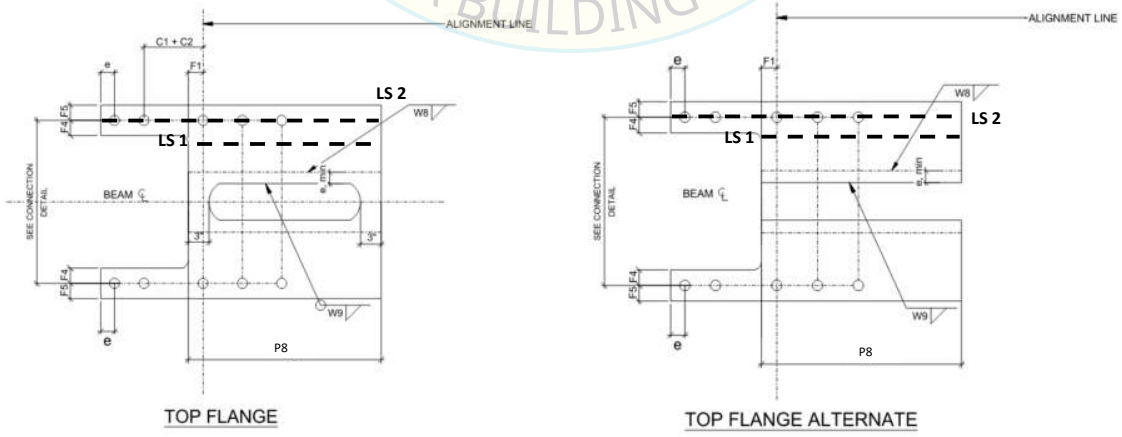
Top plate thickness (see step 2)	*T4* (in)	3
Top plate dimension (see schedule)	*P8* (in)	67
Fuse plate dimension	*F1 (in)*	2.00
Gross shear area of top plate, *(P8)(T4)*	A_g (in^2)	201.000
Nominal shear yielding capacity (AISC 360 Eq. J4-3), $0.6F_{yTop}A_g$	R_n (kips)	6030.0
Factored shear yielding capacity, $1.0R_n$	ϕR_n (kips)	6030.0
Top plate shear yielding demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.24 **OK**

Rupture check (LS 2)

Bolt diameter (see step 4)	d_{bolt} (in)	1.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Number of oversized bolts in Zone M (see step 5)	n_m	8
Number of oversized bolts on top plate (see step 5)	n_b	19
Bolt minimum edge distance for oversized hole (AISC 360 Table J3.4)	*e* (in)	1.625
Bolt spacing (see schedule)	*s* (in)	3.75
Fuse plate dimension	*F1 (in)*	2.00
Top plate dimension, $e + s*(n_m-1) + C1 + C2 + -F1 + P8$	*Dim 1* (in)	96.625
Gross shear area of top plate, *(Dim 2)(T4)*	A_g (in^2)	289.875
Number of oversized bolts on top plate (see step 5)	n_b	19
Net shear area of top plate, $A_g-(T4d_O n_b)$	A_n (in^2)	200.813
Nominal shear rupture capacity (AISC 360 Eq. J4-4), $0.6A_nF_{uTop}$	R_n (kips)	7831.7
Factored shear rupture capacity, $0.75R_n$	ϕR_n (kips)	5873.8
Top plate shear rupture demand/capacity ratio, $R_u / \phi R_n$	*D/C*	0.25 **OK**

Size the top plate alternative plate welds (W8 and W9)

Required strength of alternate top plates (per side)	R_u *(kips)*	1466.6
Beam flange width	b_{fb} (in)	16.9
Weld 8 size	D_{W8} (1/16 in)	9
Weld 8 holdback required, each end (AISC 358 15.5(1)), $D_{W8}/16$	l_{hb8} (in)	0.56
Weld 8 length, $P8 - 2l_{hb8}$	l_{w8} (in)	65.88
Weld 9 size	D_{W9} (1/16 in)	9
Weld 9 length, *P8 - 2*3"*	l_{w9} (in)	61.00
Longitudinal shear demand on weld 8 & 9 (AISC 358 Eq. 15.6-15), $R_u /(l_{w8}+l_{w9})$	r_{uv} (kip/in)	11.6
Factored capacity of critical weld segment (AISC 360 Eq. J2-4), $0.75(0.6)(F_{EXX})(0.707)(D_{W8}/16)(1")$	ϕr_n (kip/in)	12.5
Alternative top plate-to-beam flange weld tearing check, $r_{uv}/\phi r_n$	*D/C*	0.92 **OK**



REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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)

Refer to AISC 358 Fig. C-15.9

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	27.5
Total number of bolts (see step 5)	n_b	19
Number of bolts in the M region (see step 5)	n_m	8
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)	617.5

Yielding check

Top plate dimension	*P4* (in)	2.25
Top plate dimension	*P5* (in)	3.625
Top plate thickness (see step 2s)	*T4* (in)	3
Gross area, *([P4]+[P5])*(T4)*	A_g (in^2)	17.625
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yTop} A_g$	P_n (kips)	881.3
Factored tensile yielding capacity, $0.9P_n$	ϕP_n (kips)	793.1
Top plate tensile yielding demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.78 **OK**

Rupture check

Top plate dimension	*P4* (in)	2.25
Top plate dimension	*P5* (in)	3.625
Top plate thickness (see step 2)	*T4* (in)	3
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Effective area of rupture, *([P4]+[P5]-d_O-1/16)(T4)*	A_e (in^2)	12.750
Nominal tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uTop} A_e$	P_n (kips)	828.8
Factored tensile rupture capacity, *0.75P_n*	ϕP_n (kips)	621.6
Top plate tensile rupture demand/capacity ratio, $P_u / \phi P_n$	*D/C*	0.99 **OK**

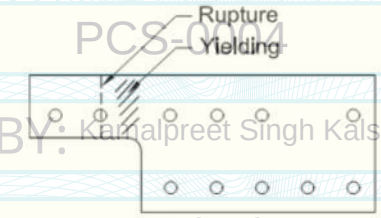


Fig. C-15.9. Tension rupture of net section on top plate.

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE 2025

Step 17 Check that P2 on the top plate is sufficient for entering and tightening requirements and for yielding

Refer to AISC 358 Fig. C15-10

Entering and tightening check

Beam flange fillet dimension	k_1	FALSE
Bolt hole diameter used for rupture limit state calculations (see step 16), d_O	d_h (in)	1.5625
Required P2 dimension for tightening, (AISC 358 Eq. 15.6-29), $k_1 + d_h$	$P2_{req}$ (in)	1.5625
Provided P2 dimension	*P2* (in)	4.0625 **OK**

Top plate yielding due to combined flexure and shear check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
Drag force per line of bolts, $P_{d,total} / 2$	P_d (kips)	27.5
Top plate thickness (see step 2)	*T4* (in)	3
Distance from beam center line to outside bolt line (see AISC 358 Figure 15.6)	*P10* (in)	14.6
Inside length of the top plate	*P8* (in)	71.50
Dimension, $(V_{fe} + P_d)/((0.9)(0.6)F_{yTop} T4)$	*m* (in)	18.107
Dimension, *([P8] - m) / 2*	*e* (in)	26.697
Required P2 dimension for yielding and combined flexure (AISC 358 Eq. 15.6-30), *[P10]-(0.9F_{yTop})e(m+e)T4 / (V_{fe} + P_d)*	$P2_{req}$ (in)	0.0
Provided P2 dimension	*P2* (in)	4.0625 **OK**

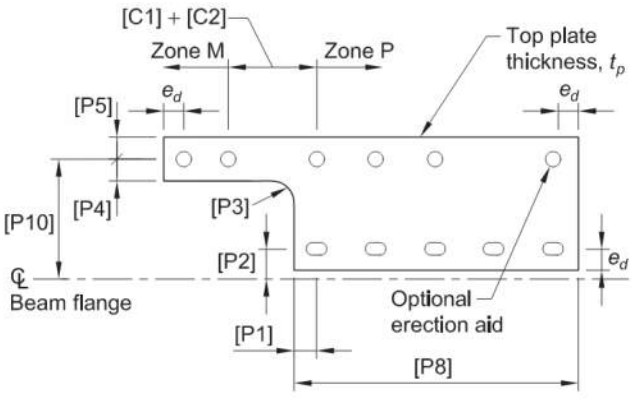


Fig. 15.6. Top plate detail.

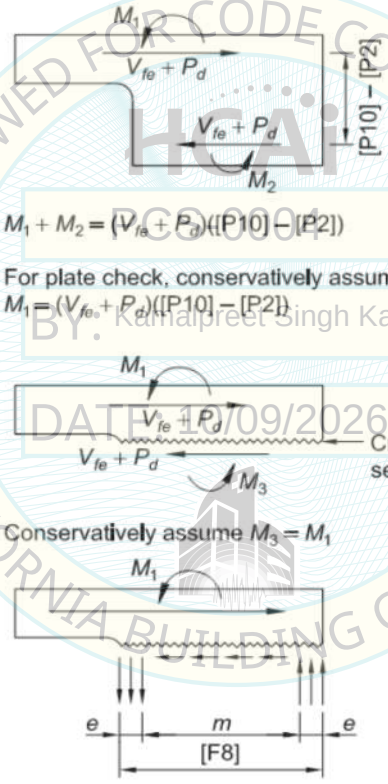


Fig. C-15.10. Lower bound model to check top plate yield strength for combined shear and flexure.

REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025
HCAI
PCO-0001
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026

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

Refer to AISC 358 Fig. C-15.11

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)	4	**OK**
Fuse plate thickness (see step 2)	$T2$ (in)	2.25	
Maximum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{max}$	4.25	
Minimum width-thickness ratio of yielding region (AISC 358 15.3(4))	$(F6/T2)_{min}$	1.5	
Provided width-thickness ratio of yielding region	$F6/T2$	1.8	**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)	1439.1	
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)	$n_{b,\ Bot}$	19	
Number of bolts in the M region (see step 5)	$n_{m,\ Bot}$	8	
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)	1211.9	
Fuse plate dimension, centerline to gage line	$F7$ (in)	4.0625	
Fuse plate dimension, gage line to outside edge	$F8$ (in)	4.2875	
Fuse plate thickness (see step 2)	$T2$ (in)	2.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625	
Gross area for yielding, $2([F7]+[F8])T2$	A_g (in^2)	37.575	
Tensile yielding capacity (AISC 360 Eq. D2-1), $R_{yp} A_g$	P_n (kips)	1878.8	
Factored tensile rupture capacity, $0.9P_n$	ϕP_n (kips)	1690.9	
Demand/Capacity Ratio, $R_{u,fp} / \phi P_n$	D/C	0.72	**OK**

Step 18. (continued)

Rupture in the net section ahead of the first yielding region check (see AISC 358 Figure C-15.11)			
Fuse plate dimension	$F7$ (in)	4.0625	
Fuse plate dimension	$F8$ (in)	4.2875	
Fuse plate thickness (see step 2)	$T2$ (in)	2.25	
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.563	
Effective net area of rupture, $2([F7]+[F8]-d_O-1/16)T2$	A_e (in^2)	30.263	
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	P_n (kips)	2360.5	
Factored tensile rupture capacity, $0.75R_n$	ϕP_n (kips)	1770.4	
Rupture demand/capacity ratio, $R_{u,fp} / \phi P_n$	D/C	0.68	**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

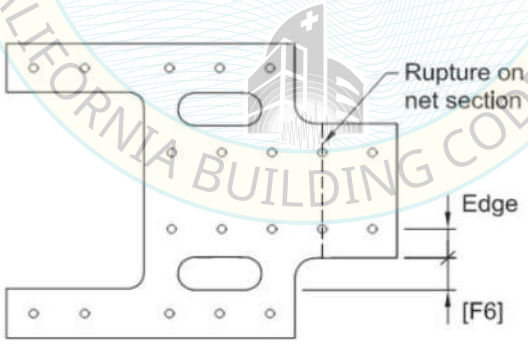


Fig. C-15.11. Limit states related to dimension [F6].

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
REVIEWED FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Step 19. Determine the yielding region depth [F2]

Width of fuse plate cut-out	*F6* (in)	4
Maximum probable moment at the fuse, (AISC 358 15.6, see step 2)	M_{pr} (k-in)	134199
Fuse plate thickness (see step 2)	*T2* (in)	2.25
Top plate thickness (see step 2)	*T4* (in)	3
Beam depth	d_b (in)	44
Strain hardening factor, $R_{tp}F_{uFuse}/R_yF_{yFuse}$	C_{pr}	1.42
Beam shear demand (see step 10), $(2M_{pr}/(B\text{-}d_c))+V_{gravity}$	V_u (kips)	736.9
Bolt pretension (AISC 360 Table J3.1)	T_b (kips)	102
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_n	10
Nominal shear capacity of all bolts per line (AISC 360 Eq. J3-4), $\mu D_u h_f T_b n_s n_n$	R_n (kips)	345.8
Resistance factor for bolt slip	ϕ	0.7
Nominal shear capacity of all shear tab bolts per line, (AISC 358 Eq. 15.6-23, see step 5a), $1.5\phi R_n$	ϕR_n (kips)	363.1
Shear tab bolt slip force mulitplied by 0.4, ϕR_n *0.4	R_{tab} (kips)	145.2
Maximum yielding region depth (AISC 358 Eq. 15.6-34), $(0.9M_{pr}\text{-}R_{tab}(d_b+T2/2+T4/2)/2)/((d_b+T2/2+T4/2)*4T2(0.6R_{yp}F_{yFuse}C_{pr})$	$F2_{max}$ (in)	5.978
Provided yielding region depth	*F2* (in)	5.875 **OK**
Fuse yield force (one side), $2(0.6F_{yFuse})[F2]T2$	V_y (kips)	793.1
Maximum probable force at beam bottom flange level corresponding to maximum moment (one side), (see step 2)	V_{fe} (kips)	1439.1
Lower bound ratio	V_{fe}/V_y *LB*	1.7
Ratio	V_{fe}/V_y	1.8 **OK**

Fuse plate yielding region geometry

Maximum width-depth ratio of the yielding regions of the fuse plates (AISC 358 15.3(5))	$(F6/F2)_{max}$	1.85
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]* <= *1.85*	*F6 / F2*	0.67 **OK**

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

Refer to AISC 358 Fig. C-15.12

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1
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)	$n_{b,\ Bot}$	19
Number of bolts in the M region	$n_{m,\ Bot}$	8
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)	1211.9

Yielding check

Fuse plate dimension	*F4* (in)	2.25
Fuse plate dimension	*F5* (in)	4
Fuse plate thickness (see step 2)	*T2* (in)	2.25
Oversized hole size (AISC 358-22 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Gross area of yielding, *2([F4]+[F5])(T2)*	A_g (in^2)	28.125
Tensile yielding capacity (AISC 360 Eq. D2-1), $F_{yFuse} A_g$	R_n (kips)	1406.25
Factored tensile rupture Capacity, $0.9R_n$	ϕR_n (kips)	1265.63
Yielding demand/capacity ratio, $R_{u,fpn} / \phi R_n$	*D/C*	0.96 **OK**

Rupture check

Fuse plate dimension	*F4* (in)	2.25
Fuse plate dimension	*F5* (in)	4
Fuse plate thickness (see step 2)	*T2* (in)	2.25
Oversized hole size (AISC 358 15.5.2(2), AISC 360 Table J3.3)	d_O (in)	1.5625
Effective net area of rupture, $2([F4]+[F5]-d_O-1/16)(T2)$	A_e (in^2)	20.8125
Tensile rupture capacity (AISC 360 Eq. D2-2), $F_{uFuse} R_{tp} A_e$ [a]	R_n (kips)	1623.4
Factored tensile rupture capacity, $0.75R_n$	ϕR_n (kips)	1217.5
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

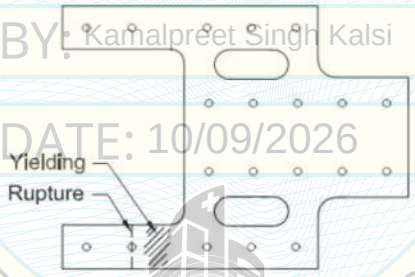


Fig. C-15.12. Fuse fracture limit states that are checked in Step 20 of the design procedure.

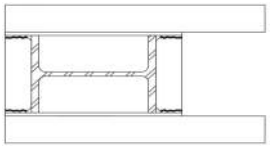
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS 0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

MISCELLANEOUS CALCULATIONS

Bridge plate calculations

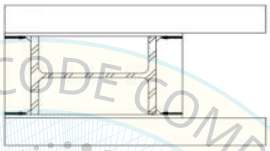
Bridge plate tension failure check

Maximum probable force per bolt line, (see step 2)	V_{fe} (kips)	1439.1	
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0	
Drag force per line of bolts, $P_d / 2$	P_d (kips)	27.5	
External continuity plate dimension	$C5$ (in)	4	
Column depth	d_c (in)	43	
Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	P_u (kips)	N/A	
Cover plate overhang	$C3$ (in)	6	
Bridge plate thickness (not greater than 2W5)	$T4$ (in)	0.000	
Bridge plate factored tensile capacity, $0.9*F_{yBridge}\ T4([C3]-1")$	ϕP_n (kips)	N/A	
Tension failure demand/capacity ratio, $P_u/\phi P_n$	D/C	N/A	**N/A**



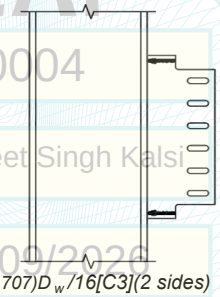
Bridge plate to cover plate weld failure check

Bridge plate tensile demand, $(V_{fe}+P_d)[C5]/(d_c+[C3])$	R_u (kips)	N/A	
Weld 5 size	D_{W5} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	N/A	**N/A**



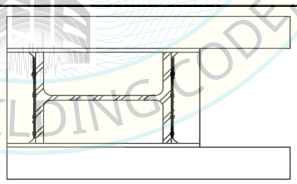
Shear tab to bridge plate weld failure check

Minimum bay width	B (in)	420.0	
Column depth	d_c (in)	43	
Beam depth	d_b (in)	44	
Shear force from gravity	$V_{gravity}$ (kips)	25.0	
Maximum proabable moment at fuse location (see Step 2)	M_{pr} (k-in)	134199	
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	736.9	
Distance from alignment line to column	$C1$ (in)	11.625	
Weld 7 demand, $V_u\ C1/d_b$	R_u (kips)	N/A	
Weld 7 size	D_{W7} (1/16 in)	0	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	N/A	**N/A**



Bridge plate to column flange weld failure check

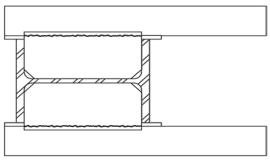
Total vertical force at alignment line, $2M_{pr}/(B-d_c) + V_{gravity}$	V_u (kips)	N/A	
Distance from alignment line to column	$C1$ (in)	11.625	
Weld 6 demand, $V_u[C1]/d_b$	R_u (kips)	N/A	
Weld 6 size	D_{W6} (1/16 in)	0	
Cover plate overhang	$C3$ (in)	6	
Column flange width	b_{fc} (in)	16.7	
Weld 6 effective length, $MIN(b_{fc} - 2", 2[C3])$	l_{eff} (in)	12	
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)	N/A	
Weld failure demand/capacity ratio, $R_u/\phi R_n$	D/C	N/A	**N/A**



*These checks are N/A when bridge plates do not apply

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

Column depth	d_c (in)	43	
Column flange thickness	t_{fc} (in)	3.23	
Weld 4 size	D_{W4} (1/16 in)	22	
Weld 4 effective length, $d_c - 2\ t_{fc}$	l_{eff} (in)	36.54	
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)	1118.9	
Shear plate thickness, t_{sp} >= $1.1*W4$ or $3/8"$	t_{sp} (in)	1.625	
Shear plate factored shear capacity (AISC 360 J4-3), $1.0(0.6)F_{ySP}t_{sp}(d_c-2t_{fc})$	ϕR_n (kips)	1781.3	
Shear plate demand/capacity ratio, $R_u/\phi R_n$	D/C	0.6	**OK**



*This check is N/A when shear plates do not apply

REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2025
CALIFORNIA BUILDING CODE 2025

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)	49898.203
Column flange width	b_{fc} (in)	16.7
Beam depth	d_b (in)	44
Bay width	B (in)	420.0
Number of bolts in zone M	M	8
Number of bolts in zone P	P	11
Bolt spacing	s (in)	3.75
Cover plate thickness	$T1$ (in)	2.25
Fuse plate thickness	T2 (in)	2.25
Top plate thickness	$T4$ (in)	3
External continuity plate dimension	$C5$ (in)	4
Fuse plate dimension	$F2$ (in)	5.875
Fuse plate dimension	$F3$ (in)	29.5
Fuse plate dimension	$F4$ (in)	2.3
Fuse plate dimension	$F6$ (in)	4
Fuse plate dimension	$F8$ (in)	4.2875
Distance from beam centerline to bolt line	$P2$ (in)	4.0625
Fuse region 1 stiffness, $(2[F2] + [F3])T2G / [F4]$	K_1 (k/in)	4.62E+05
Fuse region 2 stiffness, $2[F2]T2G / [F6]$	K_2 (k/in)	7.40E+04
Fuse region 3 stiffness, $2(0.37)ET2[F2]^3/[F6]^3$	K_3 (k/in)	1.53E+05
Fuse region 4 stiffness, $(2[F2] + [F3])T2G / [F8]$	K_4 (k/in)	2.42E+05
Fuse stiffness, $2/(1/K_1 + 1/K_2 + 1/K_3 + 1/K_4)$	K_{bottom} (k/in)	7.59E+04
Top plate stiffness, $(2P + M - 1)sT4G/(b_{fc}/2 + T1 + [C5] - [P2])$	K_{top} (k/in)	3.47E+05
Connection stiffness, $(d_b + T2/2 + T4/2)^2/(1/K_{top} + 1/K_{bottom})$	K_S (k/in)	1.35E+08
Required stiffness, $18EI_{xb}/B$	K_{req} (k/in)	6.20E+07
Connection stiffness check, K_{req} / K_S	*Ratio < 1*	0.46 **OK**

CERTIFIED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

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

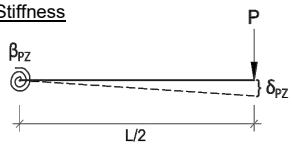
Show that effective panel zone stiffness ≥ 10*beam flexural stiffness

Effective Panel Zone Stiffness

$K_{PZ} = P/\delta_{PZ}$

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

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

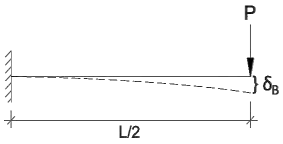


Beam Stiffness

$K_B = P/\delta_B$

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

$K_B = 3EI/(L/2)^3$



$K_{PZ} \geq 10K_B$

$\beta_{PZ}/(L/2)^2 \geq 10*3EI/(L/2)^3$

---> $\beta_{PZ} \geq 60EI/L$ (double for two-sided connection)

Beam depth	d_b (in)	44	
Beam flange thickness	t_{fb} (in)	2.875	
Column depth	d_c (in)	43.0	
Column flange width	b_{fc} (in)	16.7	
Column web thickness	t_{wc} (in)	1.79	
Column flange thickness	t_{fc} (in)	3.23	
Cover plate overhang	$C3$ (in)	6.0	
Cover plate thickness	$T1$ (in)	2.25	
Bay width	B (in)	420.0	
Story height	H (in)	192.0	
Panel zone factor, $d_b d_c t_{wc} + 2d_b(d_c + 2[C3])T1$	V_p (in^3)	14276.7	
Column ratio, $(d_c - t_{fc}) / B$	α	0.095	
Beam ratio, $(d_b - t_{fb}) / H$	β	0.214	
Web and cover plate stiffness, $GV_p / (1-\alpha-\beta)^2$	K_{ps} (k-in)	3.35E+08	
Flange stiffness, $0.78Gb_{fc}(t_{fc}^2) / (1-\alpha-\beta)^2$	K_{fs} (k-in)	3.19E+06	
Panel zone spring stiffness, $K_{ps} + K_{fs}$	β_{PZ} (k-in)	N/A	
Beam moment of inertia	I_{xb} (in^3)	49898.203	
Required panel zone spring stiffness , $60EI_{xb} / B$ for one-sided, $120EI_{xb} / B$ for two-sided	K_{req} (k-in)	N/A	
Rigid panel zone check, $K_{req} < \beta_{PZ}$	*Ratio < 1*	N/A	**N/A**

*This check is N/A when connection stiffness is applied in the model analysis

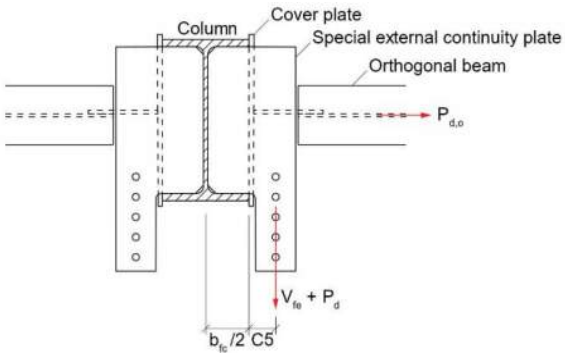
REVIEWED FOR CODE COMPLIANCE
HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
CALIFORNIA BUILDING CODE, 2025

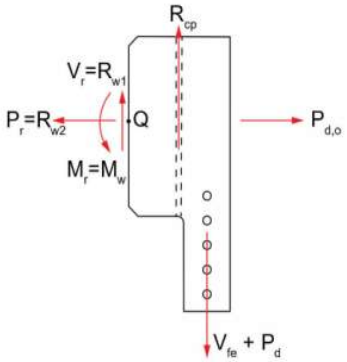
Additional calculations for connections with orthogonal forces from cantilevers or drags (no bridge plates considered)

Connection demands (one side)		
In-plane force corresponding to max fuse shear (in one external continuity plate), (see step 2)	ΣV_{fe} (kips)	2878.3
Total drag force transmitted through the connection (from EOR)	$P_{d,total}$ (kips)	55.0
In-plane drag load (in one external continuity plate), $P_{dtotal}/2$	P_d (kips)	27.5
Total orthogonal drag load (from EOR)	$P_{d,do,\ total}$ (kips)	0.0
Orthogonal drag load (in top or bottom external continuity plate), $P_{d,do,total}\ /\ 2$	$P_{d,do}$ (kips)	0.0
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)	0.0
Column flange width	b_{fc} (in.)	16.7
Column depth	d_c (in.)	43.0
Column flange thickness	t_{fc} (in.)	3.23
Column web thickness	t_{wc} (in.)	1.79
External continuity plate thickness	$T5$ (in)	3.000
External continuity plate gage dimension	$C5$ (in.)	4.00
Plate-to column flange force, $[(\Sigma V_{fe} + P_d)\ /\ (1+0.5t_{wc}\ /\ T5)]$	R_{cp} (kips)	N/A
Plate-to-column web connection design force (shear), $(\Sigma V_{fe} + P_d) - R_{cp}$	R_{w1} (kips)	N/A
Plate-to-column web connection design force (normal), $=P_{d,o}$	R_{w2} (kips)	N/A

External continuity plate section check (at the coverplate)			
Moment demand at critical net section, $(\Sigma V_{fe} + P_d)[C5]$	M_r (k-in)	N/A	
Normal demand at critical net section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at critical net section, $(\Sigma V_{fe} + P_d)$	V_r (kips)	N/A	
Plate plastic section modulus, $(d_c - 2t_{fc})^2\ T5/4$	Z (in^3)	1001.38	
Plate section area, $(d_c - 2t_{fc})\ T5$	A (in^2)	109.62	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
	Special detailing not required		

External continuity plate section check (at the column web)			
Moment demand at section, $(\Sigma V_{fe} + P_d)(b_{fc}/2 + [C5])-R_{cp}(b_{fc}/2)$	M_r (k-in)	N/A	
Normal demand at section, $=P_{d,o}$	P_r (kips)	N/A	
Shear demand at section, $=R_{w1}$	V_r (kips)	N/A	
Corner clip dimension	K_c (in)	4.41	
Plate plastic section modulus, $(d_c-2K_c)^2\ T5/4$	Z (in^3)	876.2	
Plate section area, $(d_c-2K_c)T5$	A (in^2)	102.54	
Moment capacity at section, $0.9F_{yECP}Z$	M_c (k-in)	N/A	
Normal capacity at section, $0.9F_{yECP}A$	P_c (kips)	N/A	
Shear capacity at section, $0.75(0.6F_{uECP})A$	V_c (kips)	N/A	
Interaction equation, $(M_r / M_c)+(P_r / P_c)^2+(V_r / V_c)^4$ (see AISC Manual Eq 9-1)	D/C	N/A	**N/A**
These checks are N/A when OOP forces do not apply	Special detailing not required		





Figures 13 and 14 PCS-0004

HCAi
PCS-0004
BY: Kamalpreet Singh Kalsi
DATE: 10/09/2026
FOR CODE COMPLIANCE
CALIFORNIA BUILDING CODE, 2025

Additional calculations for connections with orthogonal forces (continued)

Added continuity plate when orthogonal beam depth is deeper than moment frame beam depth		
Orthogonal beam size		
Orthogonal beam depth	$d_{b,o}$ (in)	
In plane moment frame beam 1		BUW44X1.875X16.9X2.875
In plane moment frame beam 2		BUW44X1.875X16.9X2.875
Max in-plane moment frame beam depth	d_b (in)	44
Total orthogonal force at continuity plate level, $P_{d,do} + P_{d,mo}$	$P_{d,o}$ (kips)	0.0
New continuity plate required for deeper cantilever beam		N/A
New continuity plate width past column flange	C4 (in)	8.625
New continuity plate thickness, cantilever beam flange thickness + 1/8"	t_{cont} (in)	N/A
Simply supported plate buckling coefficient	k	0.43
Allowable buckling stress, $k\pi^2E/(12*(1-v^2) * (t/b)^2$	σ_{cr} (ksi)	N/A
Total orthogonal stress at continuity plate level, $P_{d,o}/(C4*t_{cont})$	σ_{dem} (ksi)	N/A
Continuity plate buckling check, $\sigma_{dem} / \sigma_{cr}$	DCR	N/A **N/A**

This check is N/A when OOP forces do not apply