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Experimental Design for Determining the Cyclic Behavior of Skewed Reduced Beam Section Moment Connections having Composite Slabs

Paul Chabaud
University of Arkansas, Fayetteville

Experimental Design for Determining the Cyclic Behavior of Skewed Reduced Beam Section Moment Connections having Composite Slabs

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Paul Chabaud
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____
Gary S. Prinz, Ph.D.
Thesis Director

____
W. Micah Hale, Ph.D.
Committee Member

____
R. Panneer Selvam, Ph.D.
Committee Member

Abstract

This paper details the experimental design for determining the cyclic behavior of skewed Reduced Beam Section (RBS) moment connections having composite slabs. The effects of composite concrete slabs on skewed RBS connections subjected to cyclic seismic prequalification loading are addressed in this test setup. Full-scale double-sided RBS SMF specimens representing both interior and exterior column connections are designed and fabricated in this work. Experimental test fixturing and a lateral load application setup within the existing Grady E. Harvell Civil Engineering Research and Education Center (CEREC) are designed. All cyclic loading protocols required for the prequalification testing are described herein. Prequalification of the skewed SMF specimens requires the application of a total story drift of 0.04 radian before a 20% reduction in plastic moment capacity (M_p).

Acknowledgments

I would like to thank the American Institute of Steel Construction (AISC) who sponsored this research project as well as W&W|AFCO Steel who fabricated all specimens and required test fixturing. This research is conducted at the Civil Engineering Research and Education Center at the University of Arkansas. I am thankful for the support of Hossein Kashefizadeh who guided me through the project, Gavin Briggs, Lane Edwards, Adam Kirschner, and Dr. Prinz for his mentorship. Finally, I would like to thank Dr. Selvam and Dr. Hale for being members of my committee.

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1. Introduction

Special Moment Frame (SMF) systems are specially designed to resist lateral forces through moment reactions at the beam-to-column connections while ensuring reliable ductility of the components. Because braces are not used, moment frame configurations offer flexibility and architectural freedom; however, moment frame designs are often governed by drift requirements rather than strength. SMFs offer the most ductility of all moment frame systems and are required to survive connection rotations of at least 0.04rad with at least 80% of the plastic moment capacity remaining [1].

Following the 1994 Northridge earthquake, Moment Frame (MF) performance became questionable as brittle fractures were observed within connection regions [2]. Additionally, plastic hinge zones developed at the column face increasing stress demand on the beam web and flanges-to-column weld connections. New design alternatives were investigated to move inelastic deformation away from vulnerable zones. One could strengthen connections or weaken the beam section. Reduced Beam Section (RBS) became the "dogbone" connection [3]. The goal consists of trimming a portion of the beam flange to force yielding into it. A reduction in beam area near the column face diminishes column panel zone demand and forces at the beam-to-column weld joints. Figure 1 illustrates the energy dissipation concept through RBS.

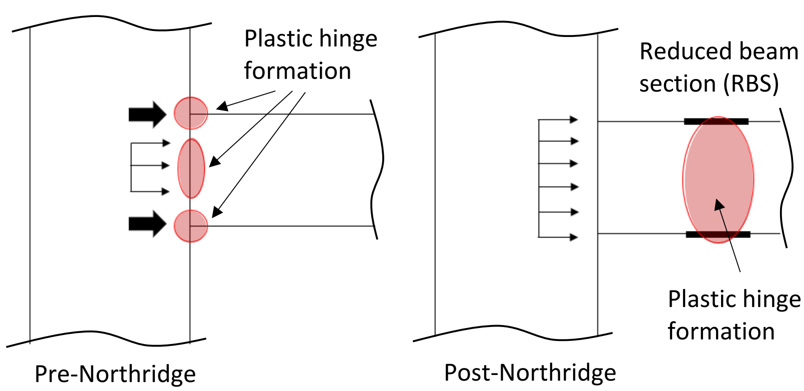


Figure 1. Energy dissipation through a non-RBS and RBS connection.

AISC 358-16 [1] establishes specifications for prequalified SMF connections to accommodate special architectural features. Even though the AISC 358-16 [1] provisions present a wide range of design procedures for orthogonal (zero-degree skew) connections, it is unclear how much a beam can skew and to what extent. The objective of this experimental design is to explore the limits of out-of-plane skew RBS connections having composite slabs.

1.1 Background

Structures are designed to be earthquake resistant. Some regions of the structure are specially designed to dissipate energy through controlled inelastic deformations. If yielding is reached, steel deforms permanently without returning to its original shape, and energy is absorbed. If not specifically considered in the design process, the occurrence of dissipative zones is difficult to predict due to complex member geometry, load shedding/redistribution, and uncertainties in material properties. To improve control over plastic hinge/dissipative zones, Plumier [4] suggested that trimming beam flanges near the beam-to-column connection would decrease demand at the column connection welds. The goal of RBS SMF design is to force yielding to occur within the RBS in a safe and ductile way.

Kashefizadeh [2] investigated the skewed beam-to-column connection performance computing finite element models and conducting full-scale experimental tests. First, a dynamic system-level computed twenty-eight models to compare skewed and unskewed RBS and Welded Unreinforced Flange-Welded Web (WUF-W) connections. The member size varied between shallow, medium, and deep. A 20-degree skew angle was assigned. The two-sided frame computational model included a composite slab in its analysis to represent realistic boundary conditions. Furthermore, six one-sided configurations were fabricated and tested. Three levels of skew were considered: 10, 20, and 30 degrees. The general design specifications along with the seismic requirements for testing MF were following AISC 358-16 [1]. The prototypes were tested horizontally. The experiments showed that the moment resistant, the column twist demand, and the RBS flange yielding decreased as the skew level increased. The dynamic system investigation showed that an increase in skew angle would result in a reduction of column axial force demand and residual drift. Skewed connections increased column twist demand due to a reduction of the moment of inertia of the column. Finally, the composite slab increased the torsional stiffness of the connection diminishing column twist.

Jones et al. [3] experimented on orthogonal double-sided RBS connections supporting a composite floor slab. This study investigated the concrete slab behavior within a resisting frame and its influence on RBS deformation. In total, eight specimens were designed with varying panel zone strength: balanced, strong, and very weak. A 25 mm gap was intentionally left between the column and the floor slab to avoid unwanted effects on the connection. All specimens exhibited acceptable performance by attaining a total story of 0.04 rad and at least 0.03 rad without any signs of fracture. The composite specimen did not demonstrate a significant difference in strain demand from a bare steel frame. The slab increased demand in the bottom flange section as expected and had a minor effect on strain demand after reaching fracture. The specimen handled a peak load 17% greater than a prototype without a slab. Overall, the slab did not cause an early fracture. Instead, it provided additional stability to the RBS connection, which increased rotation and load capacity. The RBS produced an earlier web buckling, at the plastic hinge zone.

Prinz and Richards [5] evaluated demands on reduced beam section connections with out-of-plane skew. Finite element techniques were used to investigate the influence of skewed members on RBS connections. An experimental and a realistic model were modeled to simulate realistic practice conditions. Medium, shallow, and deep skewed members ranging from 0, 10, 20 to 30 degrees were tested individually. A finite element modeling software, ABAQUS, showed a correlation between the amount of skew, column twisting, and strain demand in the RBS. The 30-degree specimen recorded the greatest column twisting and yielding at the flange column tips. It also demonstrated the greatest rotational connection capacity resulting in a less severe buckling than the unskewed specimen.

Dominguez and Prinz [6] investigated the behavior of a skewed RBS connection in a SMF supporting a composite slab. This study compared the behavior of bare steel and a composite specimen subjected to a cyclic loading protocol. The column twist, moment capacity, and plastic strain demand of the connection were assessed using finite element methods. Shallow, medium, and deep sections were skewed to three levels: 10, 20, and 30 degrees. Both prototypes reached a rotation of 0.04 rad prior to a 20% reduction in moment capacity meeting the AISC 341-16 [7] requirements. The compressive strength of the slab had a negligible effect on the connection behavior. However, the slab increased the connection moment capacity at larger rotations delaying the buckling of the beam during positive moment cycles.

2. Prototype Frame for Specimen Design

The prototype frame considered in this study represents a double-sided column configuration as would be found on an interior MF bay. The MF geometry considered is 30 ft long and 16 ft tall with the considered specimen taken from inflection-point splices in the frame configuration shown in Figure 2. In Figure 2 the section investigated comes from the design of a 6-story building. In the specimen taken from the prototype frame configuration, two medium-depth beams are laterally connected to a medium-depth column. Each framing member is skewed to 15 degrees, representing the architectural features of the prototype frame. This skew orientation was determined by a research oversight committee at AISC and is based on the previous analytical work of Prinz and Richards [5]. As will be shown in the following sections covering the member design, member selection satisfies the strong-column weak-beam ratio for SMF design established by AISC 341-16 [7]. Appendix A details the design of the prototype moment frame; however, Figure 3 showing a flowchart detailing the column-beam ratio requirements is provided herein for the considered design steps. Table 1 details the specimen geometry and steel properties resulting from the design flowchart. Floor beams are supplied to support the composite concrete slabs in compliance with [8].

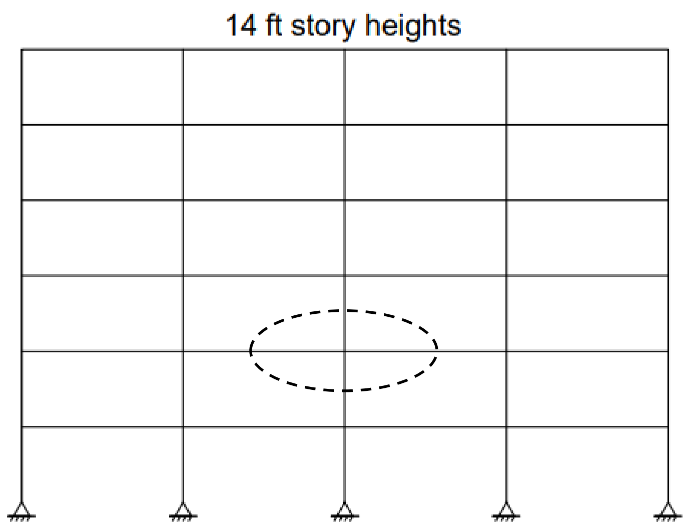


Figure 2. Specimen side view in a multistory building.

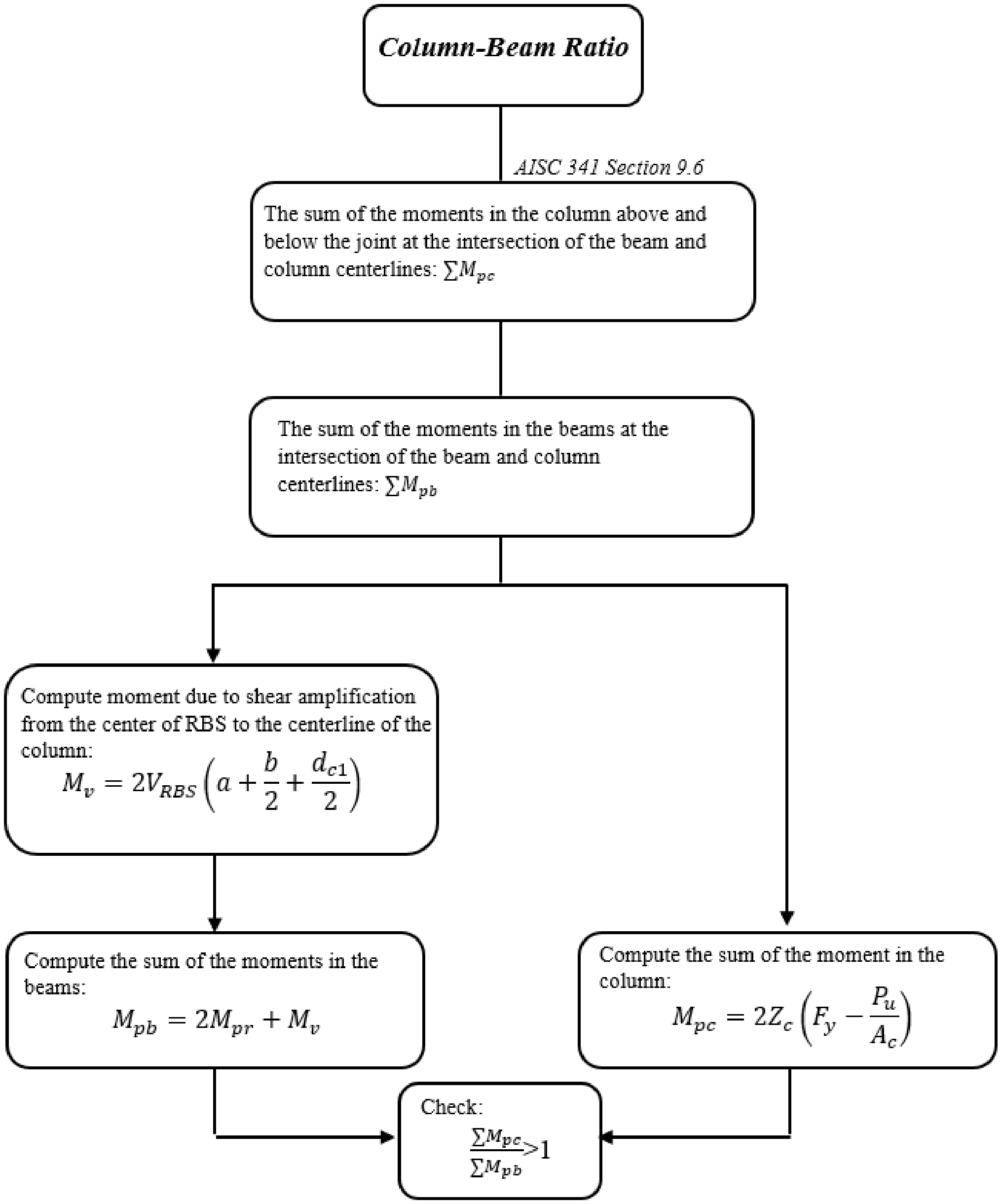


Figure 3. Column-beam combination criteria.

Table 1. Specimen detailing.

Member	Steel Section	Length (ft)	Grade	Skew (degrees)
Beam	W24x76	15	A992	15
Column	W24x131	16	A992	-

A cyclic loading protocol provided by [7] will be conducted to simulate earthquake loading at an experimental scale. A 220-kip hydraulic actuator will laterally exert the loading sequence at a vertical height of 14 ft. Loads will gradually increase until the prototype reaches failure. Figure 4 locates the line of action.

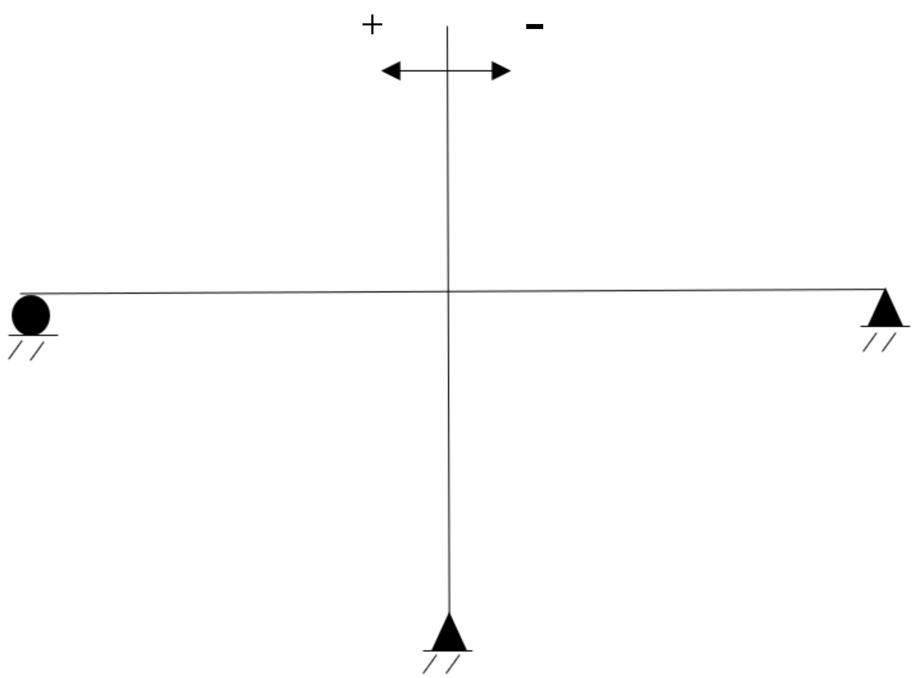
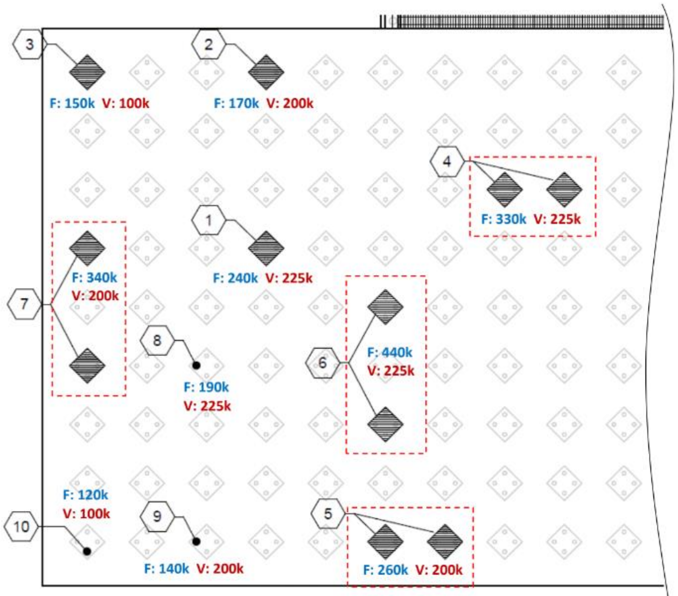


Figure 4. Loading application.

In addition, to comply with MF testing requirements, a bracing system is provided. The experiment will take place at the Grady E Harvell Civil Engineering Research and Education Center (CEREC) at the University of Arkansas. Figure 5 is a top view of the strong floor and a table listing the strong floor location, direction, and nominal capacity.



Case Number	Load Location	Load Direction	Nominal Capacity (kips)
1	Single interior anchor assembly	up	240
		down	240
		horizontal	225
2	Single edge anchor assembly	up	170
		down	200
		horizontal	200
3	Single corner anchor assembly	up	150
		down	175
		horizontal	100
4	Two adjacent interior anchor assemblies loaded simultaneously	up	330
		down	480
		horizontal	225
5	Two adjacent exterior anchor assemblies loaded simultaneously (no corner anchor assemblies)	up	260
		down	400
		horizontal	200
6	Two alternating interior anchor assemblies loaded simultaneously	up	440
		down	480
		horizontal	225
7	Two alternating exterior anchor assemblies loaded simultaneously (no corner anchor assemblies)	up	340
		down	400
		horizontal	200
8	Single interior anchor rod	up	190
		down	240
		horizontal	225
9	Single edge anchor rod	up	140
		down	200
		horizontal	200
10	Single corner anchor rod	up	120
		down	175
		horizontal	100

Figure 5. Strong floor capacity

2.1 Specimen Design, RBS Design, and Fabrication Drawings

The specimen is an assembly of two W24x76 beams connected to a W24x131 column. Members are designed according to the AISC Steel Construction Manual [8]. Figure 6 is a flowchart detailing the column-beam design checks. RBS and out-of-plane skew designs follow AISC [1,7] procedures. Appendix A, B, and C detail calculations and fabrication drawings. Figure 7 compares two multistory configurations with orthogonal (zero-degree) connections and out-of-plane skewed connections (non-zero degree).

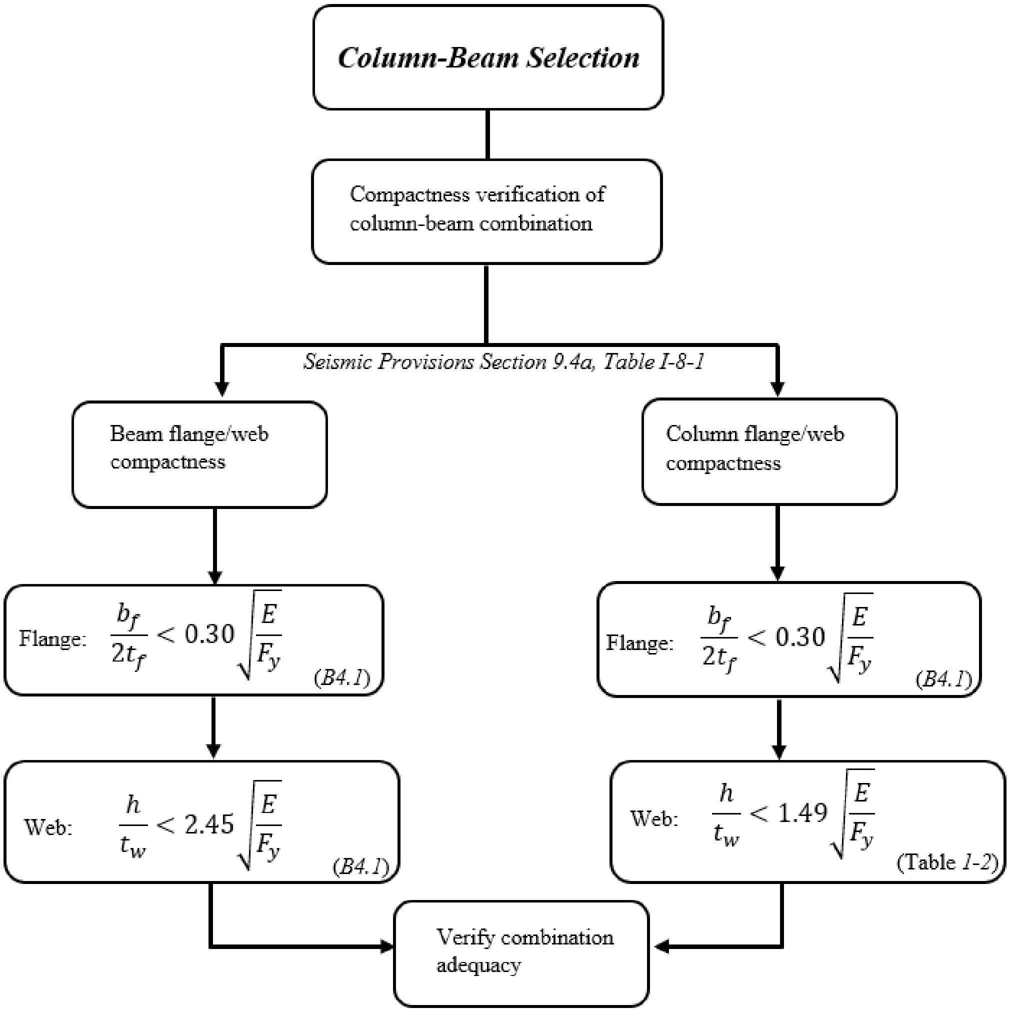


Figure 6. Column-beam adequacy

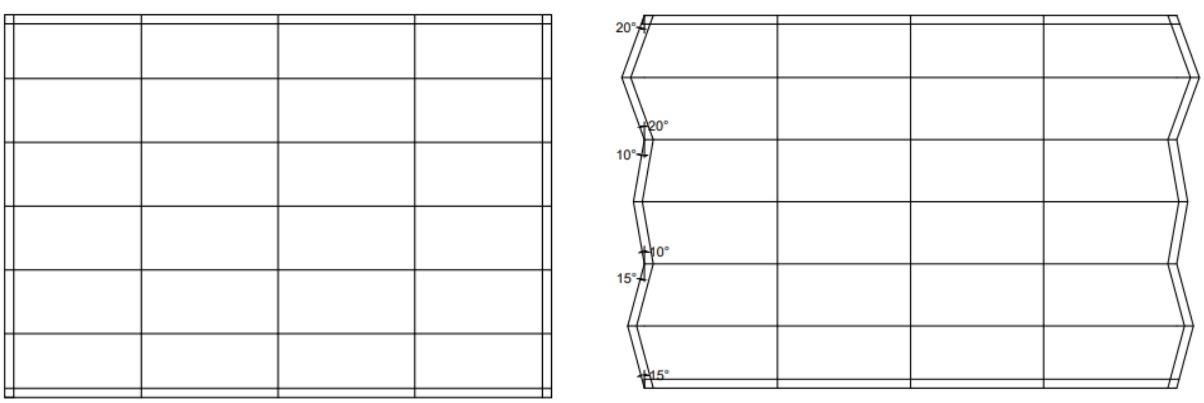


Figure 7. (a) Orthogonal connections, (b) Out-of-plane skewed connections.

RBS moves plastic hinge zones away from the column face. Its geometry may vary to straight, angularly tapered, and circular. The circular segment shape is utilized since it is widely used in the industry thanks to its great ductility level. The rounded cut absorbs energy in the column flange connection that deforms inelastically once it goes beyond the yield point. AISC Seismic Provisions (Chapter 5) [1] establishes the RBS design requirements. Dimensions are determined by three variables: cut-to-connection distance, *a*, the length and depth of the cut, *b*, and *c* respectively. Figure 8 compares an unskewed and skewed RBS.

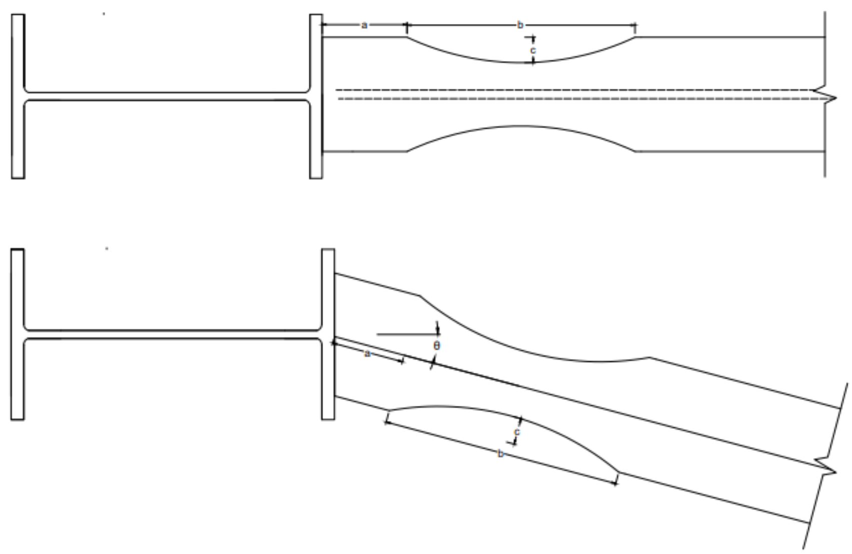


Figure 8. (a) Orthogonal reduced beam section, (b) Skewed reduced beam section.

The length between the cut and connection, *a*, must be large enough to spread stress within the beam flange at the column face. The length of the cut, *b*, must be long enough to prevent large inelastic strains in the RBS. The depth, *c*, determines the maximum moment that will develop within the column face and its section [1]. The RBS geometry must be adjusted if it is oriented. The cut length and depth remain unchanged. However, the length between the end cut and the connection changes: *a* is determined by basic trigonometry**.** Table 2 presents the RBS dimensions. Figure 9 is a flowchart of the RBS design procedure.

Table 2. RBS properties.

a **(in)**	***b*** **(in)**	***c*** **(in)**	***θ*** **(degree)**
5.5	18	2	15

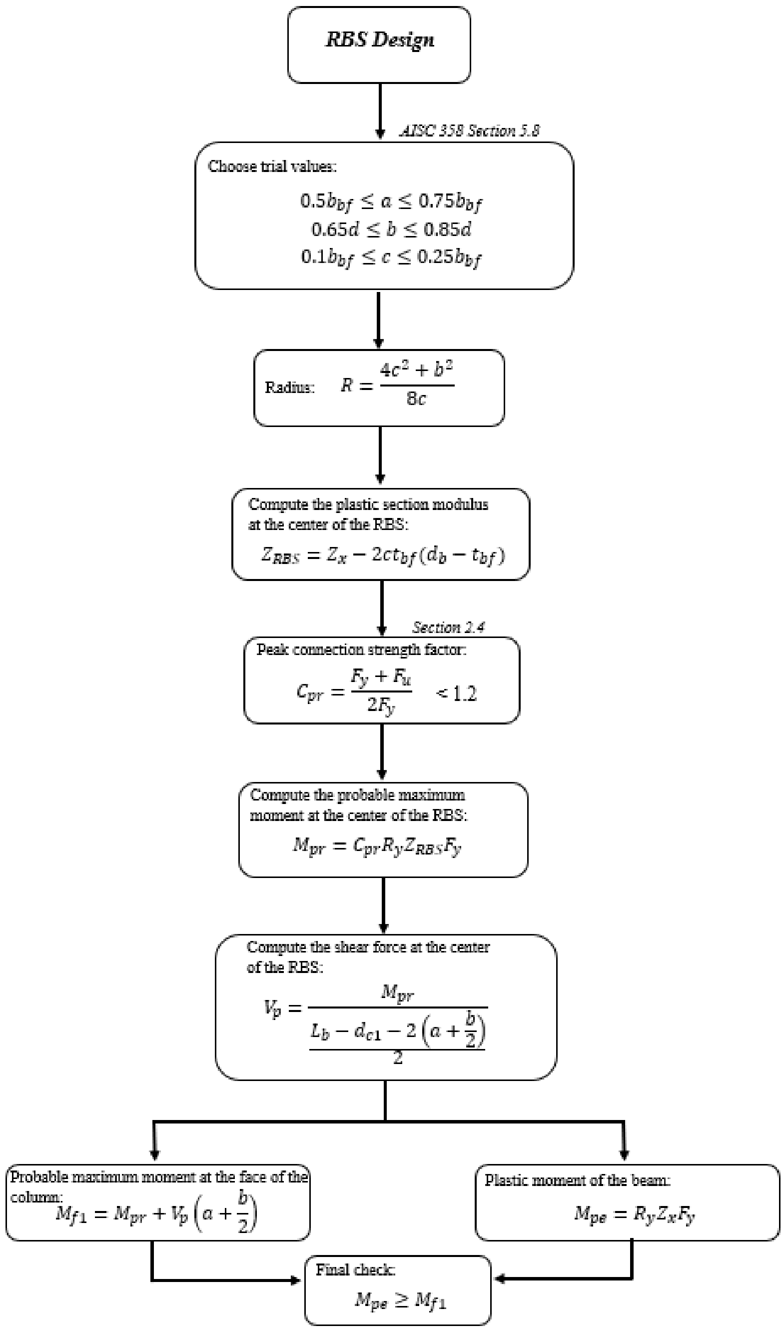


Figure 9. RBS design flowchart

AISC Seismic Provisions [1] specifies beam-to-column moment connections. The combination of bolts and welds provides extra toughness and strength to encounter inelastic behavior in zones adjacent to the column. The web-plate attachment, bolts, and any factors that weaken connection such as welds and access holes follow AISC [7] guidelines. Moment connections consist of a full depth erection plate bolted using three 1 in diameter A325X bolts and welded using CJP. The plate is used to hold the beam in place during welding. Beam flanges are also welded to the column using CJP.

Throughout the experiment, panel zone behavior becomes unpredictable. Its strength is designated as “strong”, “weak”, or “balanced”. These strength levels determine the ability of the panel zone and the framing members to deform inelastically. For instance, if the panel zone is defined as “balanced”, inelastic deformation will be equally spread within the panel zone and members adjacent. If it is “strong” or “weak”, it will tend to mainly deform near the connections or at the panel zone, respectively. A ½ in thick doubler plate is welded to one side of the column web to reinforce its strength. The plate fits in between the column flanges and extends an additional 6 1/8 in beyond the continuity plates [7]. To avoid any interference between *k* and CJP weld, a 1/8 in gap is left intentionally. Finally, the plate is welded within 1/16 in to the web. All doubler plate design requirements are established by AISC 341-16 [7]. Figure 10 details the beam-to-column connection.

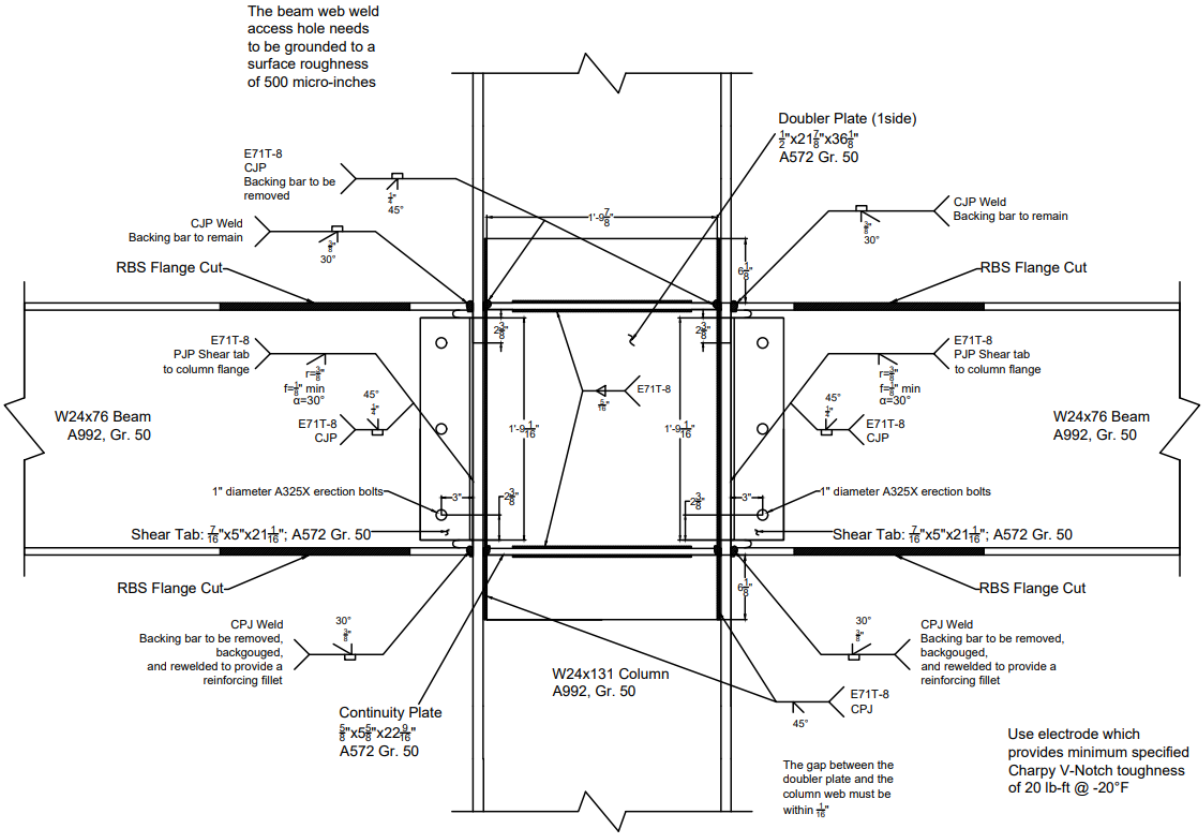


Figure 10. Detailing of the beam-to-column moment connection.

The beam-to-column connection needs continuity plates on both sides of the column to increase stiffness and moment strength. Continuity plates are considered extensions of the framing members through the panel zone. Their dimensions correspond to the beam flange thickness and the column depth. Each contact area of the plate with the column is welded using fillet weld. The plates are cut in their inside corners to avoid interactions with the column flange toe, *k*. Figure 11 details continuity plate welding. Figure 12 is a flowchart describing the panel zone design steps for the doubler plate and continuity plates.

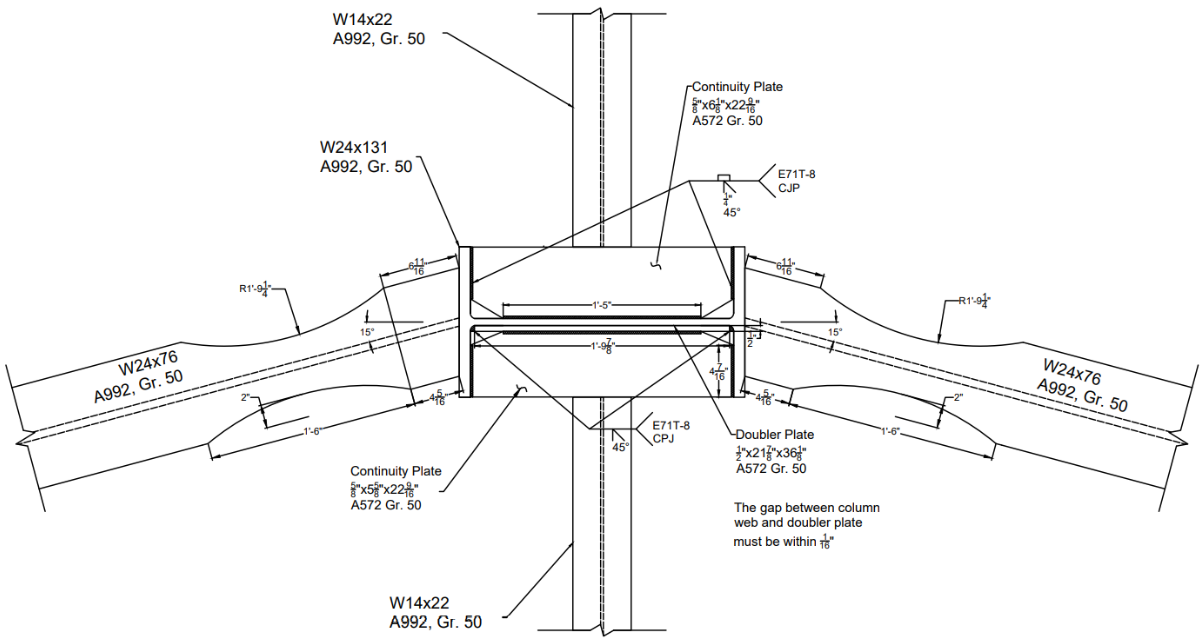


Figure 11. Continuity plates detail.

Floor beams are attached to medium-depth beams (W24x76) to support the composite concrete slab. In total, six floor beams are added to the prototype. Four beams are located 8 ft away from the RBS connections and two beams are at the center on each side of the column web. The four floor beams are coped to ease the beam-to-web connection. Coping consists of trimming a small portion of the flange and web to connect two elements together. A shear tab is welded along the medium-depth beam web and bolted to the coped beam web. A line of three 7/8 in A325X bolts is used. Gravity beams at the column panel zone are not coped. Instead, an extended shear tab covering the full depth of the panel zone is welded using fillet weld. The tab extends beyond the continuity plates to connect to a regular W14x22 beam using one line of three 7/8 in A325X erection bolts. Figure 13 shows coped and regular gravity beam connections. Figure 14 is a flowchart detailing the coped beam design.

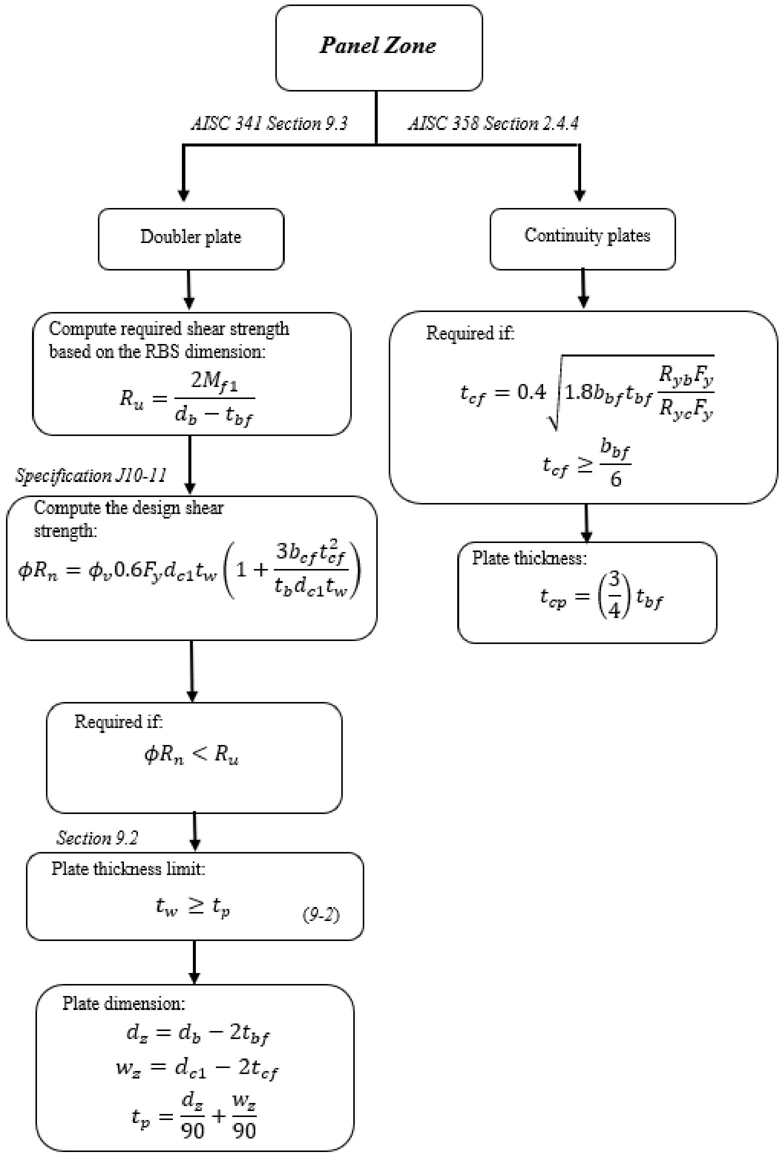


Figure 12. Panel zone reinforcement steps

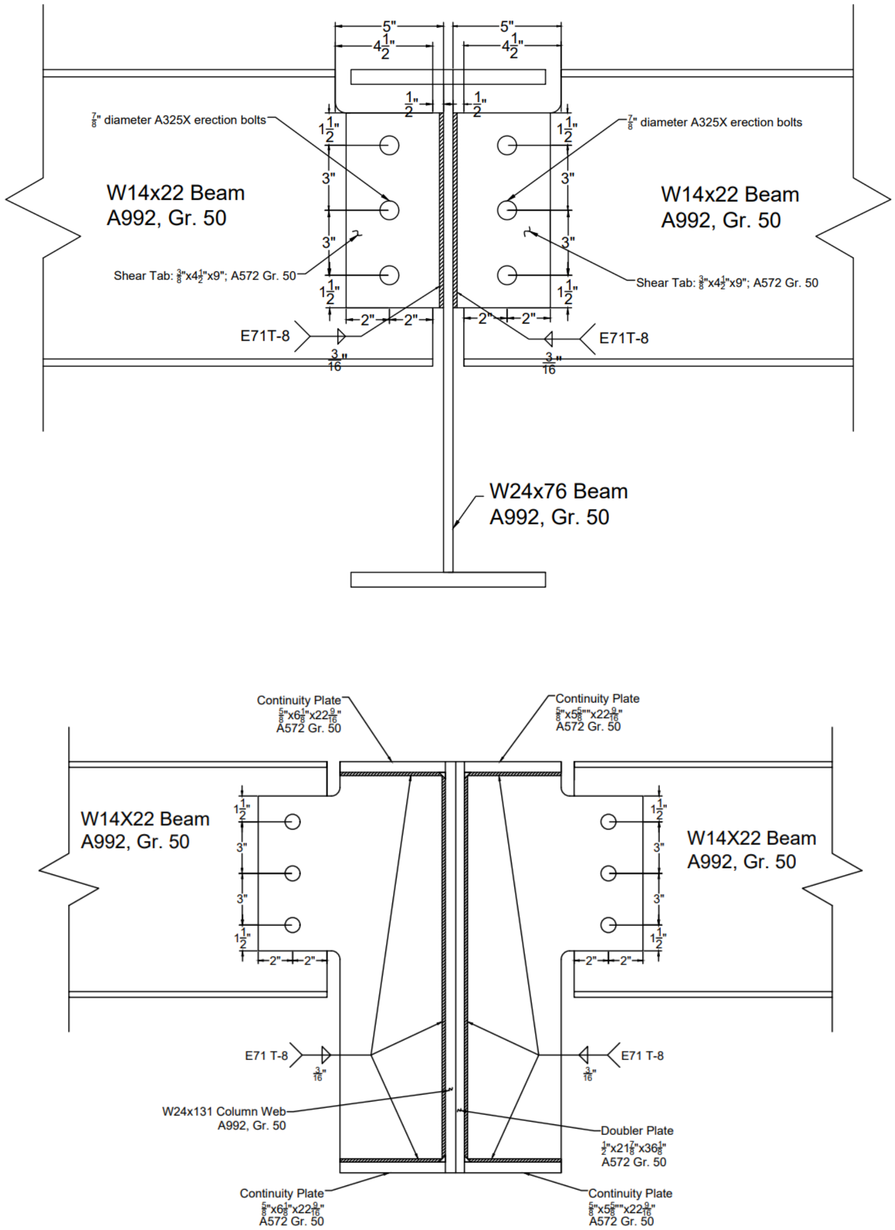


Figure 13. (a) Coped beam connections, (b) Extended shear tab connections.

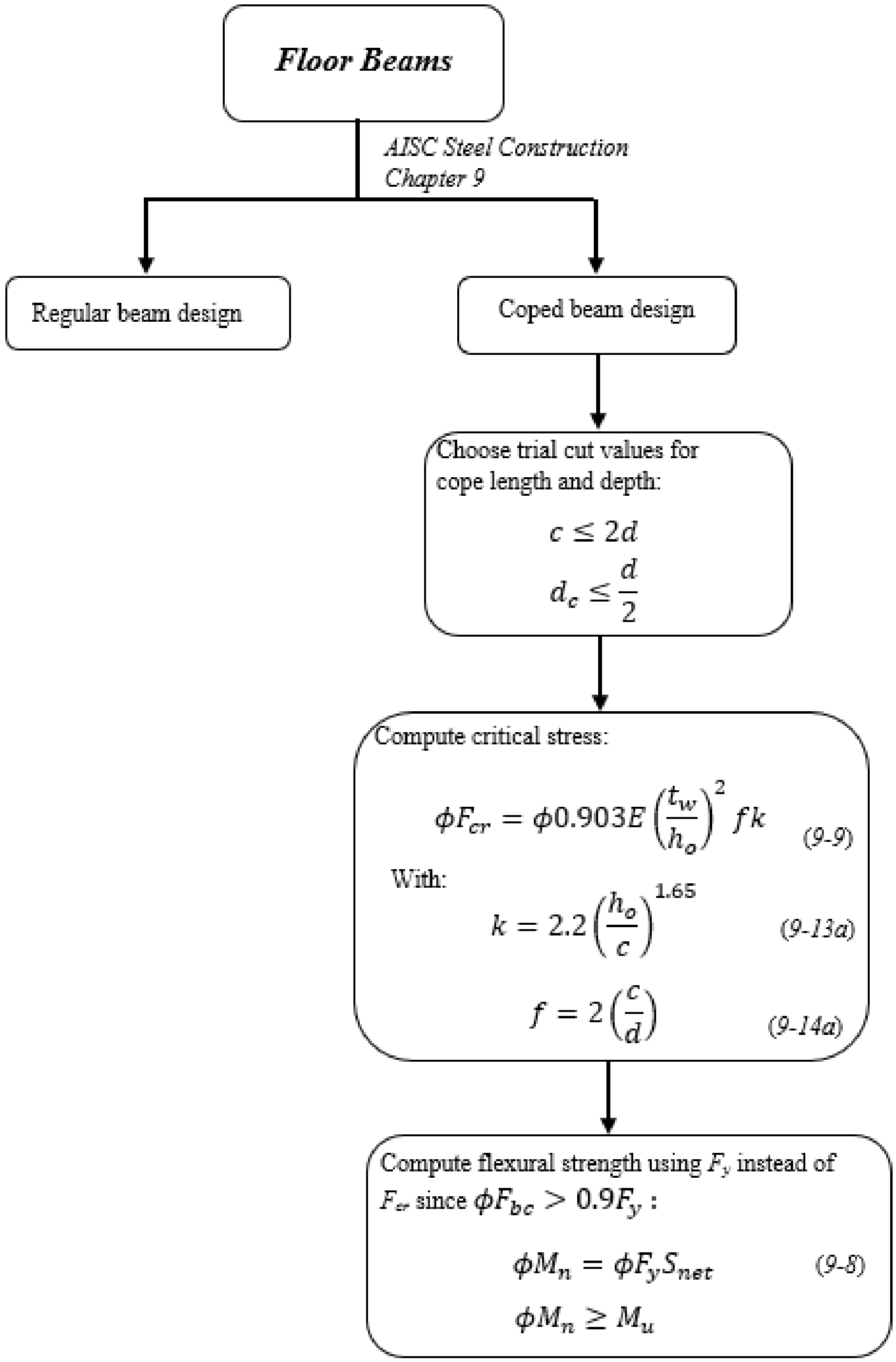


Figure 14. Coped beam design.

2.2 Test Configuration Design

The experiment will be conducted at the University of Arkansas at Fayetteville (AR). The experimental setup shown in Figure 15 consists of several columns, beams, and lateral bracing elements to allow large lateral forces to be applied to the specimen column top. Each octagonal shape refers to the strong floor location and capacity. They are all 4 ft apart center-to-center. The test configuration follows [3] that suggests setting the loading ram at an equal vertical distance *d* to the moment connection than the moment connection to the column foot connection. Here, the actuator is set 14 ft high, 7 ft away from the RBS connections. Figure 16 displays a side view of the setup.

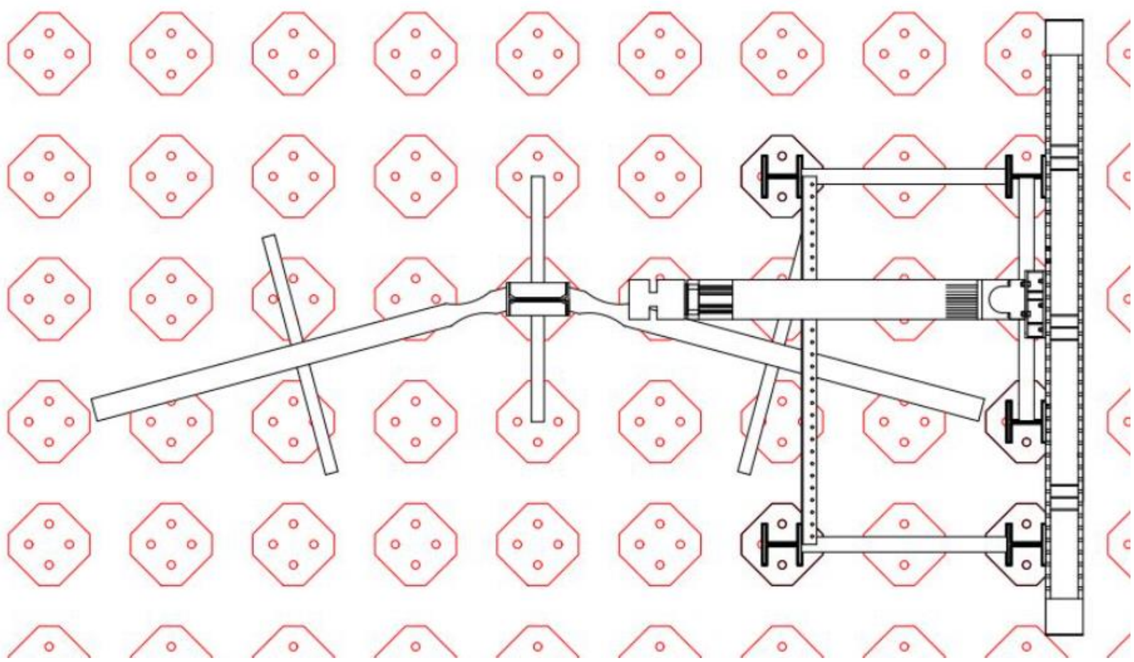


Figure 15. Test setup configuration.

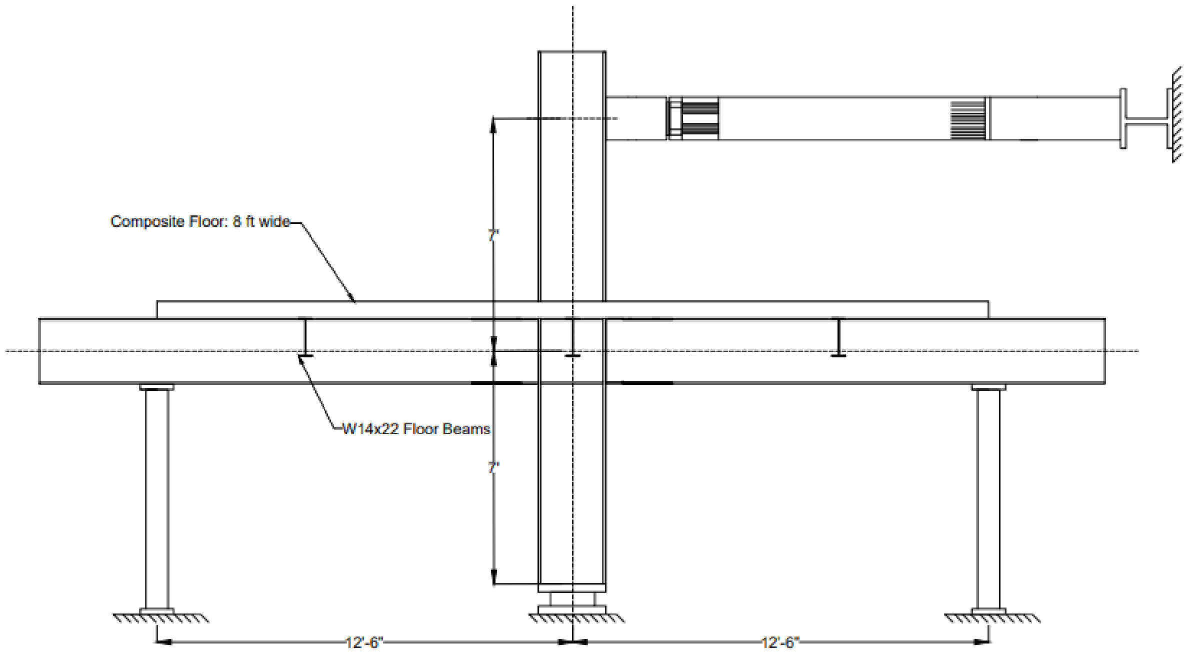


Figure 16. Side view of the test setup.

The test configuration complies with AISC Seismic Provisions [7] requirements. The code limits the subassembly to a single and double-sided connection for testing. Lateral bracing is only allowed near the loading zone. The points of inflection must be anticipated before the experiment meaning that any changes of curvature are predicted.

At the end of the experiment, the total story drift and amount of inelastic rotation determine the degree of performance of the prototype. A total story drift of 0.04 rad must be reached before a 75% deformation in the beams [7]. The prototype performance is not acceptable if this criterion is not respected.

2.2.1 Lateral Bracing System Design

A bracing system is supplied to ensure the proper loading distribution and add lateral stability to the experimental test fixtures. The system is not expected to endure over 220 kips;

however, it is overdesigned to accommodate nearly twice its capacity for future MF tests considering larger beam and column geometries. Figure 17 is a picture of a hydraulic loading ram utilized at the Research Center (CEREC).

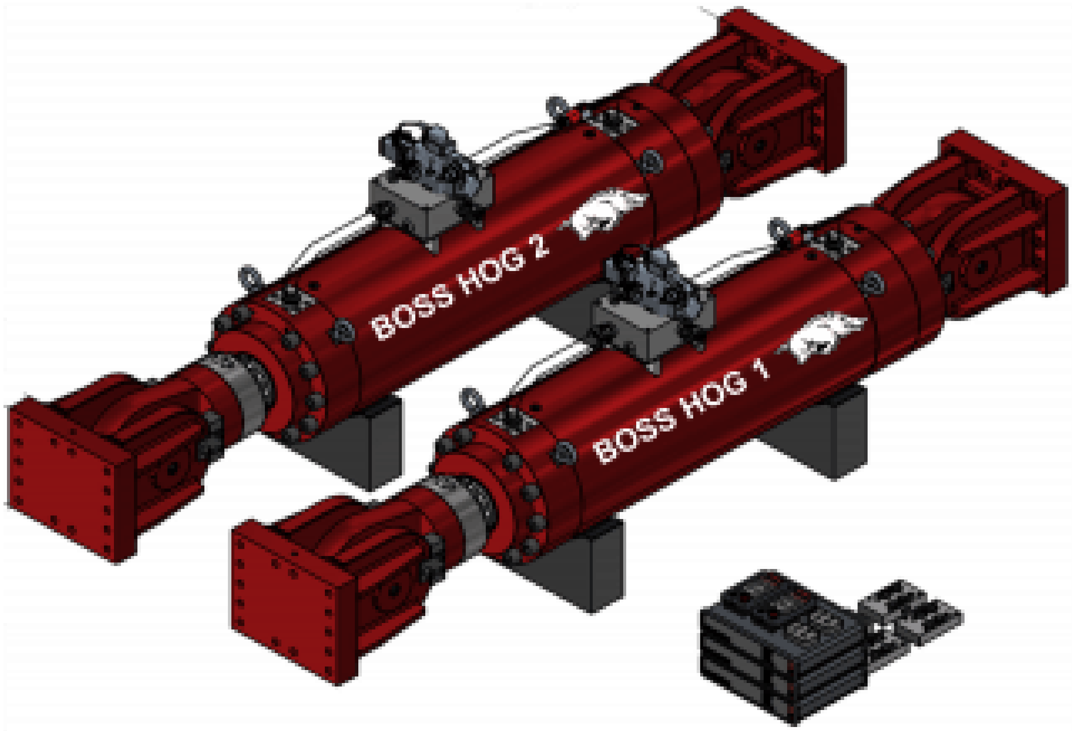


Figure 17. Hydraulic loading rams.

The brace shape is determined by the loading ram capacity, the strong floor, column spacing, and height. A HSS 10x5/8 round shape is selected as a cross member. Hollow sections provide a torsional resistance structural capacity. Each brace is 7.86 ft long, has a 10 in outer diameter, and is 5/8 in thick. Figure 18 is a side view of a brace connecting two columns. The brace connection combines four steel plates, split into three steps.

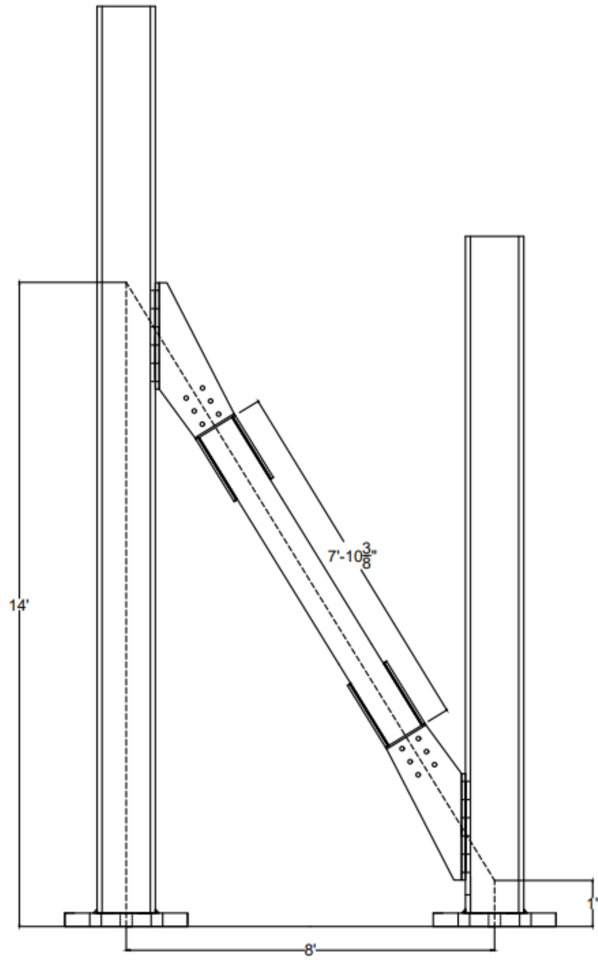


Figure 18. Side view of brace connections.

First, a cut within the hollow section is made to slide a 32 in long, 12 in wide, and 1 1/8 in thick steel plate in. The steel plate is welded using four 19 in longitudinal fillet welds. The other steel plate end is “sandwiched” between two 1 in thick gusset plates. Figure 19 is a cross section view of the HSS 10x5/8 tube.

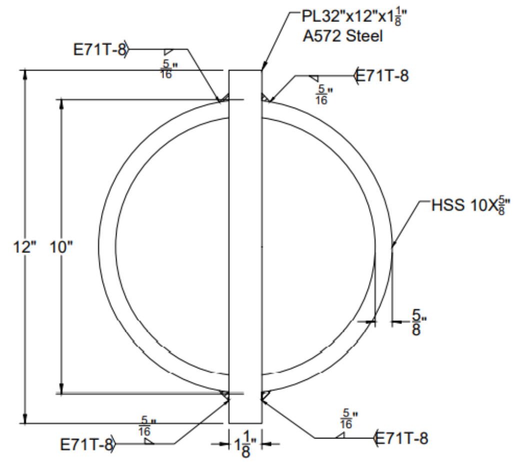


Figure 19. Cross section view of the HSS connection.

Next, the steel-to-gusset plate connection consists of two lines of three 1 1/8 in A490N erection bolts. Both gusset plates are oriented at a 58.4 angle downward or upward, matching the brace orientation. Both gusset plates are welded to a 1 in thick column flange plate using CJP weld. Figure 20 details the flange plate welds. Figure 21 is a flowchart of bolted connection design.

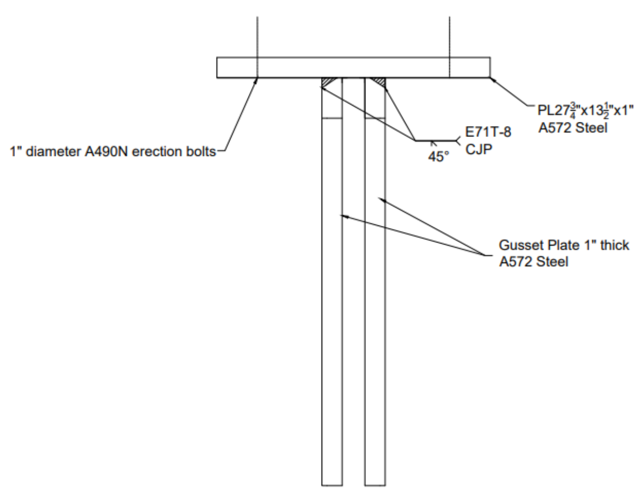


Figure 20. Top view of the column flange connection.

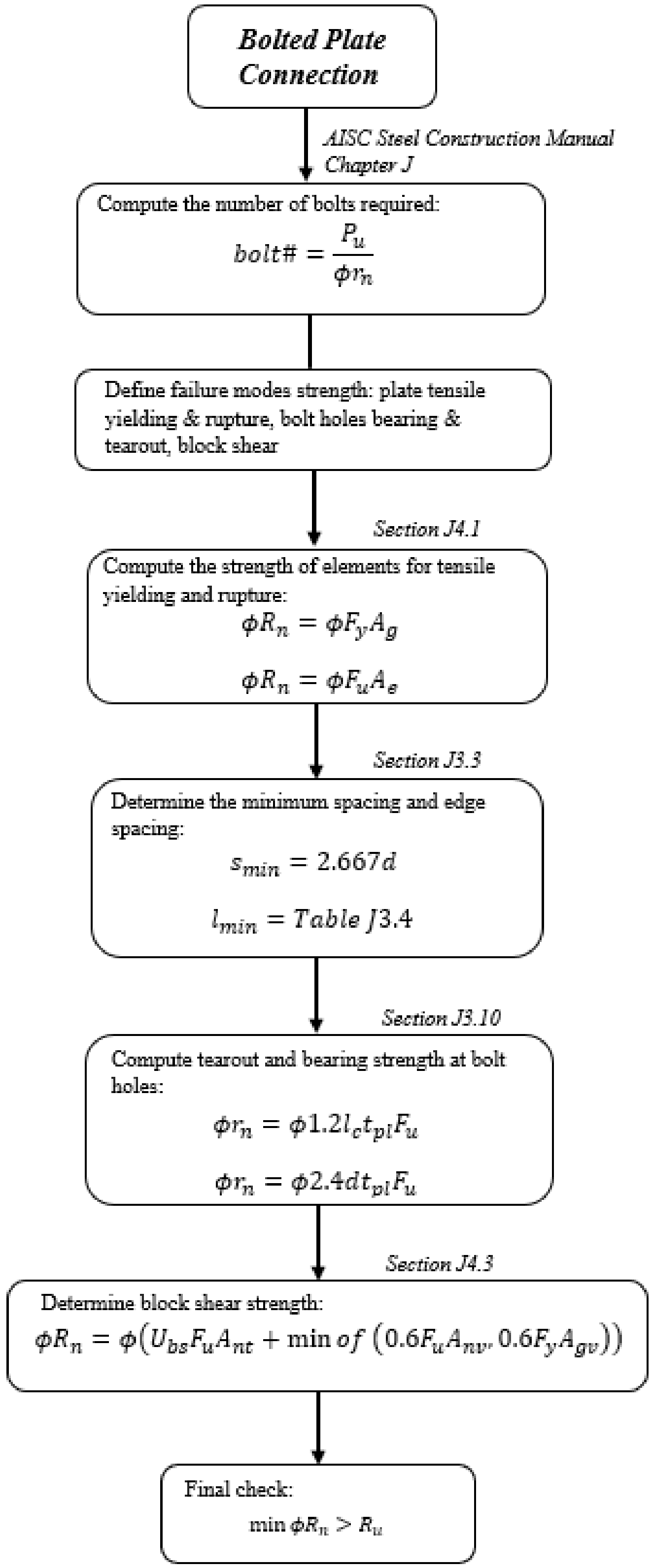


Figure 21. Bolted connection design steps.

Braces experience concentric forces. These forces do not transfer through the center of gravity of the flange connection, which produces a moment due to a 1 in eccentricity. In addition to shear and bending, bolts are in tension and compression above and below the neutral axis of the bracket. The plate design lies on predrilled bolt holes in the W14x176 columns. The 1 in diameter holes are spaced 4 ¾ in vertically and 9 ½ in horizontally. Two lines of six bolts are required. Figure 22 details the top brace connection. Appendix B provides brace fabrication drawings.

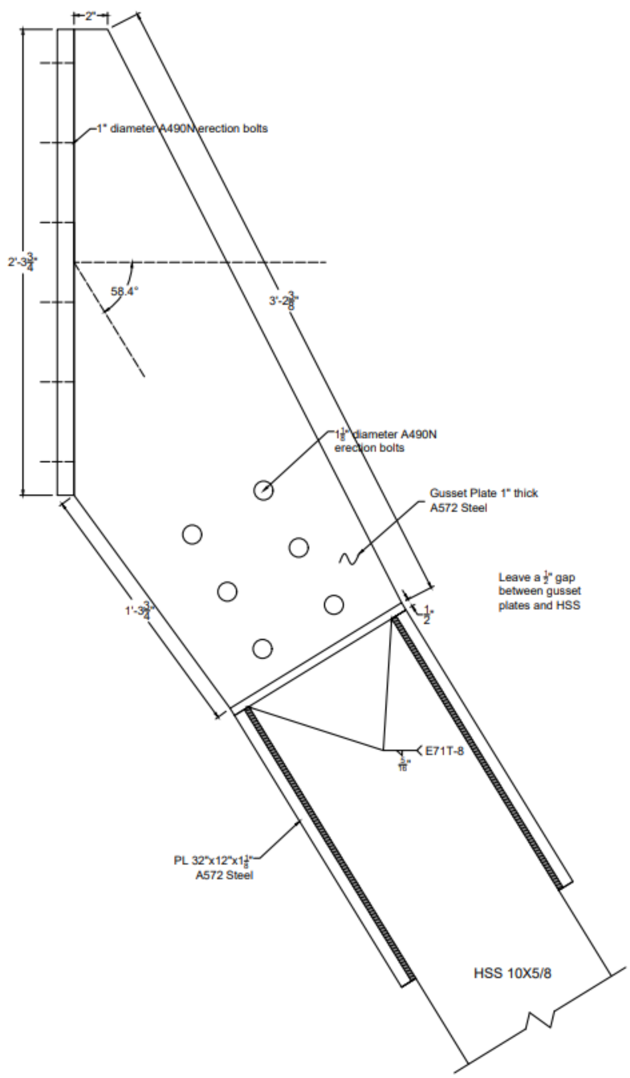


Figure 22. Detailed brace connection.

2.3 Instrumentation

Prior to the test, any pertinent information is provided. AISC Seismic Provisions [7] lists the requirements: drawings of the specimen, connections, bracing system, and details on the boundary conditions must be collected. In addition, the specimen needs to be instrumented to measure the local and global response. Linear Variable Displacement Transducers (LVDTs) are used to measure displacement. Figure 23 displays the transducer locations. Two LVDTs are supplied at each location, on the front and back of the structure. Two extra LVDTs are added to the panel zone as shown in Figure 24. Strain gauges are attached at the top and bottom of the RBS, and above and below the skewed connections. Their ability of mounting steel strain completes LVDTs. Figure 25 shows the strain gauge locations. Finally, whitewash will flake to evidence buckling.

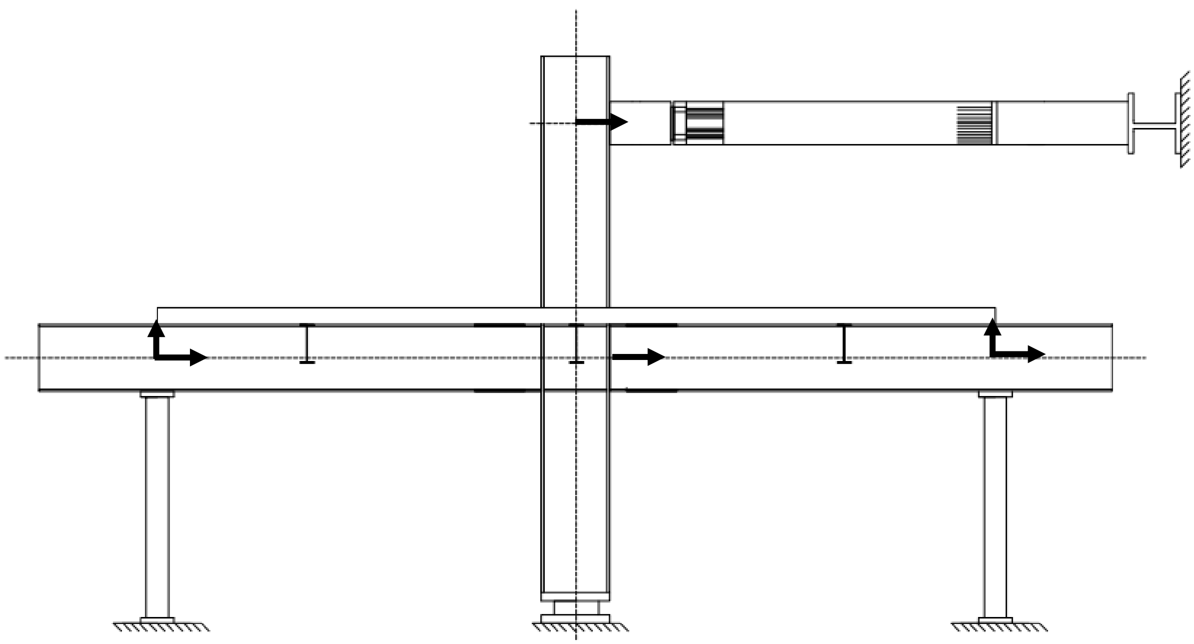


Figure 23. Prototype side view displaying LVDTs placement.

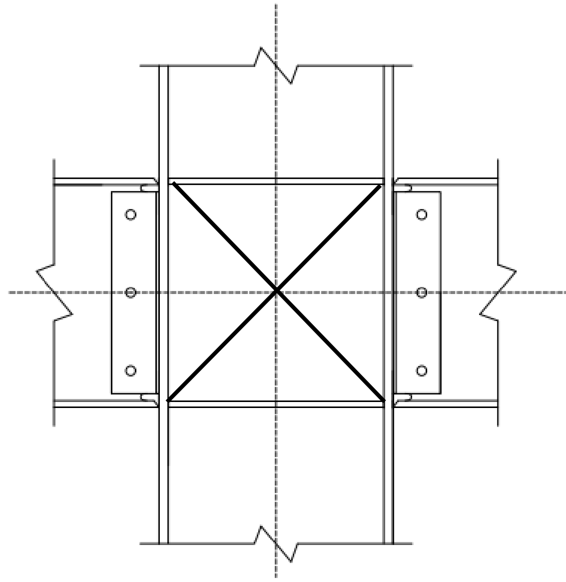


Figure 24. Location of displacement transducers.

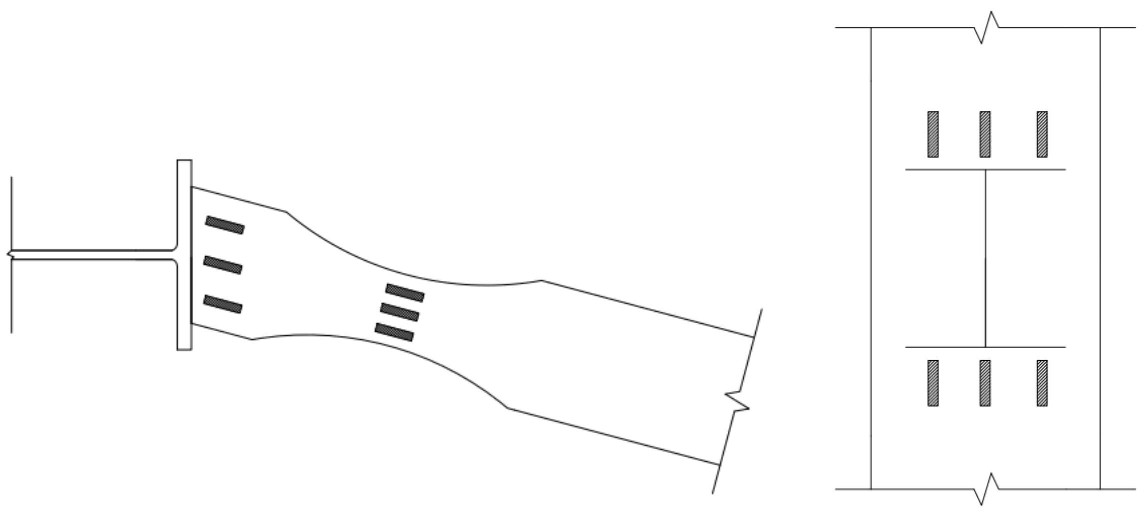


Figure 25. Strain gauge locations.

2.4 Prequalification Loading Protocol

AISC 341-16 [7] established a loading sequence for testing beam-to-column moment connections in SMF. The loading protocol is conducted and controlled by the inter-story drift angle, θ, assigned to the prototype. Loads gradually increase causing rotation at the connections until failure. Table 3 lists the specifications of the loading sequence computing the number of cycles according to the amount of story drift. The load sequence continues at increments of 0.01 radian with two cycles of loading at each step if a drift angle of 0.04 radian is reached [7]. Figure 26 is a graphical representation of the cyclic sequence.

Table 3. Loading sequence.

Number of loading cycles	Total story drift angle (rad)
6	0.00375
6	0.005
6	0.0075
4	0.01
2	0.015
2	0.02
2	0.03
2	0.04

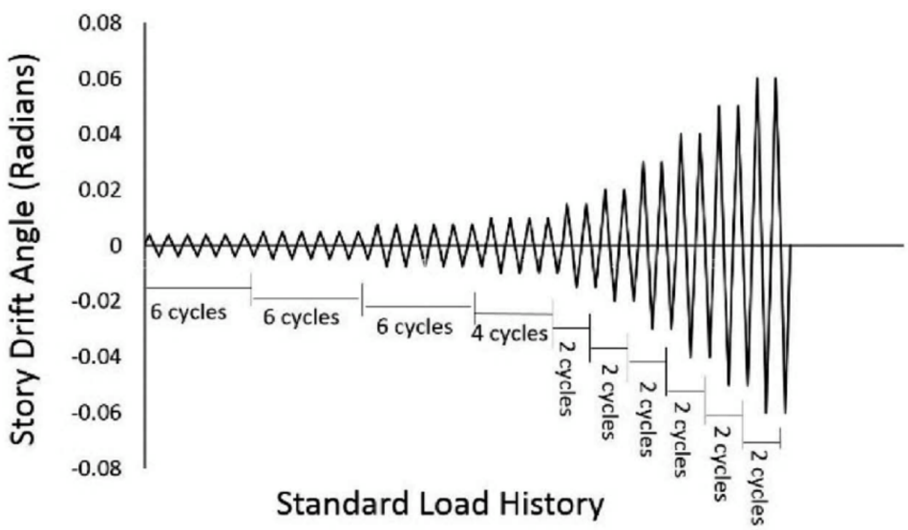


Figure 26. Cyclic loading protocol [9].

3. Conclusions

This paper is an experimental design for cyclically loaded skewed RBS moment connections having composite concrete slabs. The prototype is designed in full compliance with AISC [1,7,8]. The test has not been conducted yet; however, several assumptions may be assessed on how much the slabs will affect the connection's performance:

1) Slabs may increase connection moment capacity by providing additional stability to RBS connections.
2) Slabs may reduce twisting by increasing stiffness.
3) Slabs may decrease column twisting.
4) Slabs may increase demand at bottom flange connections.

Thus, suggestions for future research projects are made:

1) Increasing the skew level up to 20 and 30 degrees.
2) Changing the column-beam combination considering heavier members.
3) Varying the panel zone strength.
4) Trying a different RBS geometry or shape.

4. References

1. AISC (2016)."ANSI/AISC 358-16: Prequalified connections for special and intermediate steel moment frames for seismic applications." American Institute of Steel Construction, Chicago, IL.

2. Kashefizadeh, Hossein Mohammad. *Cyclic Behavior of Laterally Skewed Special Moment Frame Connections: Experimental Testing and System-Level Analysis*, May 2022.

3. Jones, Scott L., et al. "Experimental Evaluation of Cyclically Loaded Reduced Beam Section Moment Connections." *Journal of Structural Engineering*, vol. 128, no. 4, 2002, pp. 441–451.

4. Plumier, Andre. *New Idea for Safe Structure in Seismic Zones*. IABSE Reports, 1990.

5. Prinz, Gary S., and Paul W. Richards. "Demands on Reduced Beam Section Connections with out-of-Plane Skew." *Journal of Structural Engineering*, vol. 142, no. 1, 2016.

6. Dominguez, Damaso, and Gary S. Prinz. "Cyclic Behaviour of Laterally Skewed Special Moment Frame Connections Having Composite Concrete Slabs." *Steel Construction*, vol. 14, no. 2, 2021, pp. 107–116.

7. AISC (2016)."ANSI/AISC 341-16: Seismic provisions for structural steel buildings." American Institute of Steel Construction, Chicago, IL.

8. AISC (2017)."Steel Construction Manual." American Institute of Steel Construction, Chicago, IL

9. Rezaeian, Alireza, et al. "Numerical Study of Panel Zone in a Moment Connection without Continuity Plates." *Journal of Earthquake Engineering*, vol. 26, no. 2, 2020, pp. 930–948.

Appendix

A. Design Procedures

The following calculation sheets detail the design procedures of column-beam combination, RBS, moment connections, panel zone, floor beams, and braces. Each step covers design checks according to the [1,7,8].

Members selection

Beam properties (W24x76):

$I := 2100\ in^4$

$d_b := 23.9\ in$

$b_{bf} := 8.99\ in$

$t_{bf} := 0.680\ in$

$t_{wb} := 0.440\ in$

$k_{bdes} := 1.18\ in$

$h_b := d_b - 2 \cdot k_{bdes}$

$h_b = 21.54\ in$

$E := 29000\ ksi$

$F_y := 50\ ksi$

Compactness verification: Seismic Provisions Section 9.4

$\frac{b_{bf}}{2 \cdot t_{bf}} = 6.61$ (Specification B4.1)

$0.30 \cdot \left(\frac{E}{F_y}\right)^{0.5} = 7.225$ *OK*

$\frac{h_b}{t_{wb}} = 48.955$ (Specification B4.2)

$2.45 \cdot \left(\frac{E}{F_y}\right)^{0.5} = 59.004$ *OK*

Column properties (W24x131):

$I := 4020\ in^4$

$d_c := 24.5\ in$

$b_{cf} := 12.9\ in$

$t_{cf} := 0.96\ in$

$t_{wc} := 0.605\ in$

$k_{cdes} := 1.46\ in$

$h_c := d_c - 2 \cdot k_{cdes}$

$h_c = 21.58\ in$

$E := 29000\ ksi$

$F_y := 50\ ksi$

Compactness verification:

<u>Seismic Provisions Section 9.4a</u>

$\frac{b_{cf}}{2 \cdot t_{cf}} = 6.719$

(Specification B4.1)

$0.30 \cdot \left(\frac{E}{F_y}\right)^{0.5} = 7.225 \qquad OK$

$\frac{h_c}{t_{wc}} = 35.669$

(Specification B4.1)

$1.49 \cdot \left(\frac{E}{F_y}\right)^{0.5} = 35.884 \qquad OK$

<u>Seismic Design Manual</u>

(Table 1-2, Local Buckling Requirements)

RBS design

General properties:

$F_y := 50\ ksi$

$R_y := 1.1$

$F_u := 65\ ksi$

$E := 29000\ ksi$

Beam properties (W24 x 76):

$Z_x := 176\ in^3$

$L := 360\ in$

$b_{bf} := 8.99\ in$

$d_b := 23.9\ in$

$t_{bf} := 0.680\ in$

$t_{wb} := 0.440\ in$

Column properties (W24 x 131):

$d_{c1} := 24.5\ in$

$A_g := 38.6\ in^2$

$t_{cf} := 0.96\ in$

$t_{wc} := 0.605\ in$

$Z_c := 370\ in^3$

$r_y := 2.97\ in$

$k_{cdes} := 1.46\ in$

$b_{cf} := 12.9\ in$

$L_b := 360\ in$

$KL := 156\ in$

<u>Prequalified Connections for SMF and IMF for Seismic Applications (AINSI/AISC 358-05)</u>

Design procedure:

$a := 0.5 \cdot b_{bf} = 4.495\ in$	$a := 0.75 \cdot b_{bf} = 6.743\ in$	$a := 5.5\ in$
$b := 0.65 \cdot d_b = 15.535\ in$	$b := 0.85 \cdot d_b = 20.315\ in$	$b := 18\ in$
$c := 0.1 \cdot b_{bf} = 0.899\ in$	$c := 0.25 \cdot b_{bf} = 2.248\ in$	$c := 2\ in$

$$R := \frac{4 \cdot c^2 + b^2}{8 \cdot c} = 21.25\ in$$

$$Z_{RBS} := Z_x - 2 \cdot c \cdot t_{bf} \cdot (d_b - t_{bf}) = 112.842\ in^3$$

Plastic section modulus at the center of RBS

$$C_{pr} := \frac{F_y + F_u}{2 \cdot F_y} = 1.15$$

$1.15 < 1.2$ OK

$$M_{pr} := C_{pr} \cdot R_y \cdot Z_{RBS} \cdot F_y = 7137.231\ kip \cdot in$$

Probable maximum moment at the center of RBS

$$V_p := \frac{M_{pr}}{\frac{\left(L_b - d_{c1} - 2 \cdot \left(a + \frac{b}{2}\right)\right)}{2}} = 46.572\ kip$$

$$M_{f1} := M_{pr} + V_p \cdot \left(a + \frac{b}{2}\right) = 7812.532\ kip \cdot in$$

Probable maximum moment at the face of the column

$M_{pe} := R_y \cdot Z_x \cdot F_y = 9680\ kip \cdot in$

Plastic moment of the beam based on the expected yield stress

$M_{pe} \geq M_{f1}$ OK

$\frac{M_{pe}}{M_{f1}} = 1.239$ OK

Check that probable maximum moment does not exceed the factored plastic moment of beam

Column capacity:

<u>AISC Steel Construction Manual</u>

$F_e := \frac{\pi^2 \cdot E}{\left(\frac{KL}{r_y}\right)^2} = 103.744\ ksi$

(Section E3)

$\frac{F_y}{F_e} = 0.482$ $0.482 \leq 2.25$

$\phi F_{cr} := 0.9\ F_y \cdot 0.658^{\left(\frac{F_y}{F_e}\right)} = 36.779\ ksi$

$\phi P_n := \phi F_{cr} \cdot A_g = 1419.69\ kip$

$Z_{RBS} = 112.842\ in^3$

$C_{pr} = 1.15$

$P_u := 0.5 \cdot \phi P_n = 709.843\ kip$

$M_{pr} := C_{pr} \cdot R_y \cdot F_y \cdot Z_{RBS} = 7137.231\ kip \cdot in$

$$V_{RBS} := \frac{2 \cdot M_{pr}}{\left(L - d_{c1} - 2 \cdot \left(a + \frac{b}{2}\right)\right)} = 46.572\ \textbf{kip}$$

Shear force at the center of the RBS cuts (plastic hinge) at each end of beam

$$M_v := 2 \cdot V_{RBS} \cdot \left(a + \frac{b}{2} + \frac{d_{c1}}{2}\right) = 2491.627\ \textbf{kip} \cdot \textbf{in}$$

Moment produced at the column centerline by the shear at the plastic hinges

$$M_{pb} := 2 \cdot M_{pr} + M_v = 16766.09\ \textbf{kip} \cdot \textbf{in}$$

$$M_{pc} := 2 \cdot Z_c \cdot \left(F_y - \frac{P_u}{A_g}\right) = 23391.606\ \textbf{kip} \cdot \textbf{in}$$

$$\frac{M_{pc}}{M_{pb}} = 1.395$$ OK (Seismic Provisions 9.6)

Column-beam moment ratio

Beam shear strength: AISC Seismic Provisions

$$\phi := 1.0$$

$$A_w := d_b \cdot t_{wb} = 10.516\ \textbf{in}^2$$

$$\phi V_n := \phi \cdot 0.6 \cdot R_y \cdot F_y \cdot A_w = 347.028\ \textbf{kip}$$ (Specification G2-1)

$$\phi V_n > V_{RBS} = 1$$ OK

Panel zone design

Beam properties (W24x76):

$b_{bf} := 8.99\ in$

$d_b := 23.9\ in$

$t_{bf} := 0.680\ in$

$Z_x := 200\ in^3$

$L_b := 360\ in$

Column properties (W24x131):

$d_{c1} := 24.5\ in$

$t_w := 0.605\ in$

$b_{cf} := 12.9\ in$

$t_{cf} := 0.960\ in$

$A_c := 38.6\ in^2$

$R_y := 1.1$

$H := 156\ in$

$k_{des} := 1.46\ in$

Design procedure:

$$V_c := \frac{2 \cdot M_{f1}}{H} = 100.161 \ \boldsymbol{kip}$$

Shear force in the column above and below the connection

$$R_u := \frac{2 \cdot M_{f1}}{d_b - t_{bf}} = 672.914 \ \boldsymbol{kip}$$

Required shear strength of the panel zone based on the RBS dimensions

Panel zone capacity:

AISC Seismic Provisions

$\phi_v := 1.0$

(Specification J10-11)

$$\phi R_n := \phi_v \cdot 0.6 \cdot F_y \cdot d_{c1} \cdot t_w \cdot \left(1 + \frac{3 \cdot b_{cf} \cdot {t_{cf}}^2}{d_b \cdot d_{c1} \cdot t_w}\right) = 489.444 \ \boldsymbol{kip}$$

$\phi R_n < 0.75 \cdot F_y \cdot A_c = 1$ OK

$\phi R_n \geq R_u = 0$ NG ==> Need to design doubler plate

Size web doubler plate:

Seismic Provisions 9-2

$$t_w \geq \frac{(d_z + w_z)}{90}$$

$t_w := 0.605 \ \boldsymbol{in}$

Beam properties (W24 x 76):

$d_z := d_b - 2\ t_{bf}$ $d_z = 22.54 \ \boldsymbol{in}$

Column properties (W24 x 131):

$w_z := d_{c1} - 2 \cdot t_{cf}$ $\qquad$ $w_z = 22.58\ in$

$$t_w \geq \frac{(d_z + w_z)}{90} = 1$$

$$t_{pl} := \frac{d_z}{90} + \frac{w_z}{90} = 0.501\ in$$

Use:

$$t_{pl} := \frac{1}{2}\ in$$

Doubler plate width:

$w := d_{c1} - 2\ k_{des} + 0.25\ in$

$w = 21.83\ in$

Use:

$w := 21.875\ in$

Continuity plates: AISC 358 Section 2.4.4

$t_{cf} := 0.96\ in$

$R_{yc} := 1.1$

$R_{yb} := R_{yc} = 1.1$

$$t_{cf} \leq 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 1$$ OK

$$t_{cfi} := 0.4 \cdot \sqrt{1.8 \cdot b_{bf} \cdot t_{bf} \cdot \frac{R_{yb} \cdot F_y}{R_{yc} \cdot F_y}} = 1.327 \ in$$

$$t_{cf} \geq \frac{b_{bf}}{6} = 0$$

NG ==> Need to design continuity plates

Size continuity plates:

$$t_{cp} := \left(\frac{3}{4}\right) \cdot t_{bf} = 0.51 \ in$$

Use:

$$t_{cp} := \frac{5}{8} \ in$$

Floor beams design

Loads:

$$w_{14x22} := 0.022\ \frac{kip}{ft}$$

$$\gamma := 0.150\ \frac{kip}{ft^3}$$

$$t_{slab} := 6\ in$$

$$w_{slab} := \gamma \cdot t_{slab}\ \frac{1\ ft}{12\ in}$$

$$w_{slab} = 0.075\ \frac{kip}{ft^2}$$

$$Width := 7\ ft$$ *Tributary area*

$$W_{serv} := w_{slab} \cdot Width + w_{14x22}$$

$$W_{serv} = 0.547\ \frac{kip}{ft}$$

$$W_u := 1.2 \cdot W_{serv}$$

$$W_u = 0.656\ \frac{kip}{ft}$$

$$L := 4\ ft$$

$$V_u := W_u \cdot L = 2.626\ kip$$

$$M_u := \frac{W_u \cdot L^2}{2} = 63.014\ kip \cdot in$$

Beam properties (W24x76):

$b_{f1} := 8.99\ in$

Coped beam properties (W14x22): AISC Steel Construction Manual

$c := \frac{b_{f1}}{2}$

$c = 4.495\ in$

Use:

(Section 9)

$c := 4.5\ in$

$d := 13.7\ in$

$d_c := 2\ in$

$S_{net} := 7.97\ in^3$ (Table 9-2)

$t_w := 0.23\ in$

$S_{back} := 0.5\ in$

$e := c + S_{back} = 5\ in$

$h_o := d - d_c = 11.7\ in$

$F_y := 50\ ksi$

$F_u := 65\ ksi$

Coped beam flexural strenght:

$c \le 2\ d$ *Limitations*

$5 \le 27.4$ OK

$d_c \le \frac{d}{2}$

$2 \le 6.87$ OK

$\frac{c}{d} = 0.328$ $\qquad 0.365 \le 1$

Buckling adjustment

$f := 2 \cdot \frac{c}{d} = 0.657$

$\frac{c}{h_o} = 0.385$ $\qquad 0.427 \le 1$

Plate buckling coefficient

$k := 2.2 \cdot \left(\frac{h_o}{c}\right)^{1.65} = 10.645$

$\phi F_{bc} := 23590\ \mathbf{ksi} \cdot \left(\frac{t_w}{h_o}\right)^2 f \cdot k = 63.75\ \mathbf{ksi}$

$0.9 \cdot F_y = 45\ \mathbf{ksi}$

$\phi F_{bc} \ge 0.9 \cdot F_y = 1$

$\phi := 0.9$

$\phi M_n := \phi \cdot F_y \cdot S_{net}$

$\phi M_n = 358.65\ (\mathbf{kip} \cdot \mathbf{in})$

Flexural strength at the coped section

$M_u := V_u \cdot e = 13.128\ \mathbf{kip} \cdot \mathbf{in}$

$\phi M_n \ge M_u = 1$ $\qquad$ OK

Adequacy of connection:

AISC Steel Construction Manual Section J

Minimum length of connection is 1/2 of the reduced beam depth, ho

7/8 in diameter A325X bolts, 3 bolts

A572 Gr. A Plate

Minimum spacing: $d := \frac{7}{8}\ in$

$2.667 \cdot d = 2.334\ in$ use $s := 3\ in$

Minimum edge distance: $l_e \geq 1.125\ in$ use $l_{ev} := 1.5\ in$

Bolt shear strength: $n := 3$

$\phi r_n := 30.7\ kip$

$\phi R_n := n \cdot \phi r_n = 92.1\ kip$

Deformation at bolt holes: $t_{plate} := \frac{3}{8}\ in$

$t_{14x22} := 0.23\ in$

Web Plate:

Edge bolts:

$$l_c := l_{ev} - \left(\frac{d + \frac{1}{16}\ in}{2} \right)$$

$$l_c = 1.031\ in \qquad \frac{l_c}{d} \le 2 = 1$$

$$r_{ne} := 1.2 \cdot l_c \cdot t_{plate} \cdot F_u = 30.164\ kip$$

Remaining:

$$l_c := s - \left(d + \frac{1}{16}\ in \right)$$

$$l_c = 2.063\ in \qquad \frac{l_c}{d} \ge 2 = 1$$

$$r_{nr} := 2.4 \cdot d \cdot t_{plate} \cdot F_u = 51.188\ kip$$

$\phi := 0.75$

$$\phi R_n := \phi \cdot \left(n \cdot r_{ne} \right) = 67.869\ kip$$

Beam:

$$r_{nr} := 2.4\ d \cdot t_{14x22} \cdot F_u = 31.395\ kip$$

$$\phi R_n := \phi \cdot n \cdot r_{nr} = 70.639\ kip$$

Plate strength (shear):

yielding:

$$A_{gv} := \left(s + l_{ev} \right) \cdot t_{plate} = 1.688\ in^2$$

$\phi := 1$ $\qquad$ $\phi R_n := \phi \cdot 0.6 \cdot F_y \cdot A_{gv} = 50.625\ kip$

rupture: $\qquad$ $A_{nv} := t_{plate} \cdot \left(\left(s + l_{ev}\right) - 1.5 \cdot \left(d + \frac{1}{8}\ in\right)\right) = 1.125\ in^2$

$\phi := 0.75$ $\qquad$ $\phi R_n := \phi \cdot 0.6 \cdot F_u \cdot A_{nv} = 32.906\ kip$

Weld strength:

$F_{EXX} := 70\ ksi$

$w := \frac{3}{16}\ in$

$l := 2\ s + 2 \cdot l_{ev} = 9\ in$

$\phi R_n := \phi \cdot 0.6 \cdot F_{EXX} \cdot 0.707 \cdot w \cdot l \cdot 2 = 75.163\ kip$

Block shear plate: $\qquad$ $A_{nt} := \left(l_{ev} - 0.5 \cdot \left(d + \frac{1}{8}\ in\right)\right) \cdot t_{plate} = 0.375\ in^2$

$U_{bs} := 1$

$U_{bs} \cdot F_u \cdot A_{nt} = 24.375\ kip$

$0.6 \cdot F_u \cdot A_{nv} = 43.875\ kip$

$0.6 \cdot F_y \cdot A_{gv} = 50.625\ kip$

$\phi R_n := \phi \cdot \left(U_{bs} \cdot F_u \cdot A_{nt} + 0.6 \cdot F_u \cdot A_{nv}\right) = 51.188\ kip$

Plate adequate

Coped beam strength (shear):

Case 1 yielding: $A_{gv} := (2 \cdot s + l_{ev}) \cdot t_{14x22} = 1.725\ in^2$

$R_n := 0.6 \cdot F_y \cdot A_{gv} = 51.75\ kip$

rupture: $A_{nv} := t_{14x22} \cdot \left((2 \cdot s + l_{ev}) - 2.5 \cdot \left(d + \frac{1}{8}\ in \right) \right) = 1.15\ in^2$

$\phi R_n := \phi \cdot 0.6 \cdot F_u \cdot A_{nv} = 33.638\ kip$ OK

Block shear:

$l_{eh} := 2\ in$

$A_{nt} := \left(l_{eh} - 0.5 \cdot \left(d + \frac{1}{8}\ in \right) \right) \cdot t_{14x22} = 0.345\ in^2$

$U_{bs} := 1$

$U_{bs} \cdot F_u \cdot A_{nt} = 22.425\ kip$

$0.6 \cdot F_u \cdot A_{nv} = 44.85\ kip$

$0.6 \cdot F_y \cdot A_{gv} = 51.75\ kip$

$\phi R_n := \phi \cdot (U_{bs} \cdot F_u \cdot A_{nt} + 0.6 \cdot F_u \cdot A_{nv}) = 50.456\ kip$

OK

Case 2

$$A_{gv} := (s) \cdot t_{14x22} = 0.69\ in^2$$

$$A_{nv} := t_{14x22} \cdot \left((s) - 1 \cdot \left(d + \frac{1}{8}\ in \right) \right) = 0.46\ in^2$$

Block shear:

$$l_{eh} := 2\ in$$

$$A_{nt} := \left(l_{eh} - 1 \cdot \left(d + \frac{1}{8}\ in \right) \right) \cdot t_{14x22} = 0.23\ in^2$$

$$U_{bs} := 1$$

$$U_{bs} \cdot F_u \cdot A_{nt} = 14.95\ kip$$

$$0.6 \cdot F_u \cdot A_{nv} = 17.94\ kip$$

$$0.6 \cdot F_y \cdot A_{gv} = 20.7\ kip$$

$$\phi R_n := \phi \cdot \left(U_{bs} \cdot F_u \cdot A_{nt} + 0.6 \cdot F_u \cdot A_{nv} \right) = 24.668\ kip$$

OK

Use:

#3 7/8 in diameter A325X
E70xx Electrodes, w=3/16 both sides
Plate 3/8"x4 1/2"x9", A572 Gr. A

Bracing system design

Brace force:

$F_{BR} := 477\ kip$

$L_{BR} := 15.26\ ft$

r (req):

$$r_{req} := \frac{L_{BR} \cdot 12\ \frac{in}{ft}}{100}$$

$r_{req} = 1.831\ in$

Trial shape (HSS 10x5/8): AISC Steel Contruction Manual P4.88

$\phi_c P_n := 570\ kip$

$\phi_c \cdot P_n > F_{BR}$ $570\ kip > 477\ kip$ *Check slenderness*

$r := 3.34\ in$

$r > r_{req}$ $3.34 > 1.831$

OK

$E := 29000\ ksi$

$F_y := 46\ ksi$ *Check compactness*

$D := 10\ in$

$t := 0.581\ in$

AISC Sesimic Design Manual

$$\lambda_{ps} := 0.044 \cdot \frac{E}{F_y}$$

$$\lambda := \frac{D}{t}$$

(Table I-8-1)

$\lambda_{ps} = 27.739$

$\lambda = 17.212$ $\lambda < \lambda_{ps}$

OK

Column capacity (W14x176):

$R_y := 1.4$ *Tension*

$A_{BR} := 17.2\ in^2$

$F_{yBR} := 46\ ksi$ (Table I-6-1)

$F_{BRultT} := R_y \cdot A_{BR} \cdot F_{yBR}$

$F_{BRultT} = 1107.68\ kip$

$\phi_c := 0.9$ *Compression*

$F_{BRultC} := 1.1 \cdot R_y \cdot \frac{\phi_c P_n}{\phi_c}$

$F_{BRultC} = 975.33\ kip$

$L_c := 14\ ft$

$S_c := 8\ ft$

$L_b := 15.26\ ft$

$F_{col} := F_{BRultT} \cdot \frac{L_c}{L_b}$

$F_{col} = 1016.22\ kip$ $P_u := F_{col}$

$P_u = 1016.22\ kip$

$\phi_c P_n := 2570\ kip$

$\phi_c P_n > P_u$ $2570\ kip > 1016.22\ kip$

OK

Connection 1 (steel plate to HSS):

$F_{BR} = 477\ kip$

$P_u := F_{BR}$

$P_u = 477\ kip$

$A_g := 17.2\ in^2$

$U := 1$

$A_e := A_g \cdot U$

Equal longitudinal weld, E70,
5/16" fillet weld

$t_{pl} := 1.125\ in$

$D := 5\ in$

$w := \frac{5}{16}\ in$

$F_{EXX} := 70\ ksi$

$\phi := 0.75$

$\phi R_n := \phi \cdot 0.6 \cdot F_{EXX} \cdot \frac{\sqrt{2}}{2} \cdot \frac{D}{16}$

$\phi R_n = 6.961 \ \frac{kip}{in}$

$l_{req} := \frac{P_u}{\phi R_n} = 68.529 \ in$

$l := \frac{l_{req}}{4} = 17.132 \ in$ $\quad l_{tot} := 76 \ in$

$l_{min} := 4 \ w$

$l_{min} = 1.25 \ in$ OK

Use 4 equal longitudinal weld of 19 in each

HSS properties:

$F_y := 46 \ ksi$

Tension strength

$F_u := 58 \ ksi$

$\phi P_n := 0.9 \cdot F_y \cdot A_g = 712.08 \ kip$ *Yielding*

$\phi P_n := 0.75 \ F_u \cdot A_e = 748.2 \ kip$ *Rupture*

OK

Plate properties:

$F_y := 50 \ ksi$

Shear strength

$F_u := 65 \ ksi$

$A_{gv} := t_{pl} \cdot l_{tot} = 85.5 \ in^2$ $\quad A_{nv} := A_{gv}$

$\phi R_n := 1 \cdot 0.6 \cdot F_y \cdot A_{gv} = 2565\ \mathbf{kip}$ *Yielding*

$\phi R_n := 0.75 \cdot 0.6 \cdot F_u \cdot A_{nv} = 2500.875\ \mathbf{kip}$ *Rupture*

OK

Connection 2 (steel plate to gusset plates):

A572 steel, double shear, Gr. B bolts, threads included

$F_y := 50\ \mathbf{ksi}$

$F_u := 65\ \mathbf{ksi}$

$d := 1.125\ \mathbf{in}$

$\phi r_n := 101\ \mathbf{kip}$

$$\#bolt := \frac{P_u}{\phi r_n} = 4.723$$

Use 6 bolts, 2 lines of 3 bolts

Plate geometry:

$t_{pl} := 1.125\ \mathbf{in}$

$w_{pl} := 12\ \mathbf{in}$

$A_g := t_{pl} \cdot w_{pl}$

$A_g = 13.5\ \mathbf{in}^2$

$U := 1$

$$A_n := A_g - 2 \cdot \left(d + \frac{1}{8}\ in\right) \cdot t_{pl}$$

$$A_n = 10.688\ in^2$$

$$A_e := U \cdot A_n$$

$$A_e = 10.688\ in^2$$

Design strength

$$\phi P_{ny} := 0.9 \cdot F_y \cdot A_g$$

Tensile yielding

$$\phi P_{ny} = 607.5\ kip$$

$607.5 > 477\ kip$

OK

$$\phi P_{nr} := 0.75 \cdot F_u \cdot A_e$$

Tensile rupture

$$\phi P_{nr} = 521.016\ kip$$

$521.02 > 477\ kip$

OK

Spacing

$$s_{min} := 2.66 \cdot d$$

$$s_{min} = 2.993\ in$$

$$s_{max} := 7\ in$$

$$l_{emin} := 1.25\ in$$

$$l_{emax} := 6\ in$$

Use $s := 4\ in$

$$s_t := 5\ in$$

$$l_e := 2.5\ in$$

$$l_t := 3.5\ in$$

Bolt strength

Tearout

$l_c := l_e - \frac{1}{2} \cdot \left(d + \frac{1}{8}\ \mathbf{in}\right)$

$l_c = 1.875\ \mathbf{in}$

Edge bolts

$\frac{l_c}{d} = 1.667$ $\qquad 1.667 < 2$

$\phi r_{n1} := 0.75 \cdot 1.2 \cdot l_c \cdot t_{pl} \cdot F_u$

$\phi r_{n1} = 123.398\ \mathbf{kip}$

Remaining bolts

$l_c := s - \left(d + \frac{1}{8}\ \mathbf{in}\right)$

$l_c = 2.75\ \mathbf{in}$

$\frac{l_c}{d} = 2.444$ $\qquad 2.444 > 2$

Bearing

$\phi r_{n2} := 0.75 \cdot 2.4 \cdot d \cdot t_{pl} \cdot F_u$

$\phi r_{n2} = 148.078\ \mathbf{kip}$

$\phi R_n := 6 \cdot \phi r_n = 606\ \mathbf{kip}$ $\qquad 606\ kip > 477\ kip$

OK

Block shear (case 1):

$A_{gv} := 2 \cdot t_{pl} \cdot (2 \cdot s + l_e)$

$A_{gv} = 23.625\ in^2$

$A_{nv} := A_{gv} - t_{pl} \cdot \left(5 \cdot \left(d + \frac{1}{8}\ in\right)\right)$

$A_{nv} = 16.594\ in^2$

$A_{nt} := t_{pl} \cdot \left(s_t - \left(d + \frac{1}{8}\ in\right)\right)$

$A_{nt} = 4.219\ in^2$

$U_{bs} := 1$

$U_{bs} \cdot F_u \cdot A_{nt} = 274.219\ kip$

$0.6 \cdot F_u \cdot A_{nv} = 647.156\ kip$

$0.6 \cdot F_y \cdot A_{gv} = 708.75\ kip$

$\phi R_n := 0.75 \cdot (U_{bs} \cdot F_u \cdot A_{nt} + 0.6 \cdot F_u \cdot A_{nv})$

$\phi R_n = 691.031\ kip$ $\qquad$ $691.03\ kip > 477\ kip$

OK

Block shear (case 2):

$A_{gv} := 2 \cdot t_{pl} \cdot (2 \cdot s + l_e)$

$A_{gv} = 23.625\ in^2$

$A_{nv} := A_{gv} - t_{pl} \cdot \left(5 \cdot \left(d + \frac{1}{8}\ in\right)\right)$

$A_{nv} = 16.594\ in^2$

$$A_{nt} := 2 \cdot t_{pl} \cdot \left(l_t - \left(d + \frac{1}{8}\ in\right)\right)$$

$$A_{nt} = 5.063\ in^2$$

$$U_{bs} := 1$$

$$U_{bs} \cdot F_u \cdot A_{nt} = 329.063\ kip$$

$$0.6 \cdot F_u \cdot A_{nv} = 647.156\ kip$$

$$0.6 \cdot F_y \cdot A_{gv} = 708.75\ kip$$

$$\phi R_n := 0.75 \cdot \left(U_{bs} \cdot F_u \cdot A_{nt} + 0.6 \cdot F_u \cdot A_{nv}\right)$$

$$\phi R_n = 732.164\ kip \qquad 732.16\ kip > 477\ kip$$

OK

Block shear (case 3):

$$A_{gv} := t_{pl} \cdot (2\ s + l_e)$$

$$A_{gv} = 11.813\ in^2$$

$$A_{nv} := A_{gv} - t_{pl} \cdot \left(2.5 \cdot \left(d + \frac{1}{8}\ in\right)\right)$$

$$A_{nv} = 8.297\ in^2$$

$$A_{nt} := t_{pl} \cdot \left(s_t + l_t - 1.5\ \left(d + \frac{1}{8}\ in\right)\right)$$

$$A_{nt} = 7.453\ in^2$$

$$U_{bs} := 1$$

$$U_{bs} \cdot F_u \cdot A_{nt} = 484.453\ kip$$

$$0.6 \cdot F_u \cdot A_{nv} = 323.578\ kip$$

$$0.6 \cdot F_y \cdot A_{gv} = 354.375\ kip$$

$$\phi R_n := 0.75 \cdot \left(U_{bs} \cdot F_u \cdot A_{nt} + 0.6 \cdot F_u \cdot A_{nv}\right)$$

$\phi R_n = 606.023\ \mathbf{kip}$ $\qquad$ $606.02 > 477\ kip$

OK

Use PL 32"x12"x1 1/8"

HSS properties:

$A_g := 17.2\ \mathbf{in}^2$

$L_c := 16\ \mathbf{ft}$

$r := 3.34\ \mathbf{in}$

$E := 29000\ \mathbf{ksi}$

$F_y := 46\ \mathbf{ksi}$

$\phi := 0.9$

$$\frac{L_c \cdot 1\ \frac{\mathbf{ft}}{12\ \mathbf{in}}}{r} = 57.485$$

Flexural buckling

$$4.71 \cdot \sqrt{\frac{E}{F_y}} = 118.261 \qquad 50.30 < 118.26$$

OK

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{L_c \cdot 1\ \frac{\mathbf{ft}}{12\ \mathbf{in}}}{r}\right)^2}$$

$$F_e = 86.614\ \frac{\mathbf{kip}}{\mathbf{in}^2}$$

$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) \cdot F_y$

Critical stress

$F_{cr} = 36.831 \; \frac{\textbf{kip}}{\textbf{in}^2}$

$P_n := F_{cr} \cdot A_g$

Nomial compressive strength

$P_n = 633.501 \; \textbf{kip}$

$\phi \cdot P_n = 570.151 \; \textbf{kip}$ $\quad 570.151 > 477 \; kip$

OK

Connection 3 (column flange connection):

$\theta := 58.39 \; °$

$P_u = 477 \; \textbf{kip}$

$P_x := \cos(\theta) \cdot P_u$

Tension

$P_x = 250.012 \; \textbf{kip}$

$P_y := \sin(\theta) \cdot P_u$

Shear

$P_y = 406.23 \; \textbf{kip}$

$P := \sqrt{P_x^{\,2} + P_y^{\,2}}$

$P = 477 \; \textbf{kip}$

Bolt Gr. B, single shear, threads included

$d := 1 \; \textbf{in}$

$\phi r_n := 40 \; \textbf{kip}$

$$\#bolt := \frac{P_y}{\phi r_n} = 10.156$$

Use 12 bolts total, 2 lines of 6 bolts

Spacing

$s_{min} := 2.66\ in$

$s := 4.75\ in$

$s_s := 9.5\ in$

$l_{emin} := 1.25\ in$

$l_e := 2\ in$

$l_t := 2\ in$

$t_{pl} := 1\ in$

Bolt strength

Tearout

$$l_c := l_e - \frac{1}{2} \cdot \left(d + \frac{1}{8}\ in\right)$$

$l_c = 1.438\ in$

Edge bolts

$$\frac{l_c}{d} = 1.438$$

$1.438 < 2$

OK

$$\phi r_{n1} := 0.75 \cdot 1.2 \cdot l_c \cdot t_{pl} \cdot F_u$$

$\phi r_{n1} = 84.094\ kip$

$l_c := s - \left(d + \frac{1}{8}\ in\right)$ *Remaining bolts*

$l_c = 3.625\ in$

$\frac{l_c}{d} = 3.625$ $3.625 > 2$

OK

$\phi r_{n2} := 0.75 \cdot 2.4 \cdot d \cdot t_{pl} \cdot F_u$ *Bearing*

$\phi r_{n2} = 117\ kip$

$\phi R_n := 12 \cdot \phi r_n = 480\ kip$ $480\ kip > 406.23\ kip$

OK

Block shear (case 1):

$A_{gv} := 2 \cdot t_{pl} \cdot (5 \cdot s + l_e)$

$A_{gv} = 51.5\ in^2$

$A_{nv} := A_{gv} - 2 \cdot t_{pl} \cdot \left(5.5 \cdot \left(d + \frac{1}{8}\ in\right)\right)$

$A_{nv} = 39.125\ in^2$

$A_{nt} := 2 \cdot t_{pl} \cdot \left(l_t - \left(d + \frac{1}{8}\ in\right)\right)$

$A_{nt} = 1.75\ in^2$

$U_{bs} := 1$

$U_{bs} \cdot F_u \cdot A_{nt} = 113.75\ kip$

$0.6 \cdot F_u \cdot A_{nv} = 1525.875\ kip$

$0.6 \cdot F_y \cdot A_{gv} = 1421.4\ \textbf{kip}$

$\phi R_n := 0.75 \cdot \left(U_{bs} \cdot F_u \cdot A_{nt} + 0.6 \cdot F_y \cdot A_{gv}\right)$

$\phi R_n = 1151.363\ \textbf{kip}$ $1151.36 > 406.23\ kip$

OK

Block shear (case 2):

$A_{gv} := t_{pl} \cdot (5\ s + l_e)$

$A_{gv} = 25.75\ \textbf{in}^2$

$A_{nv} := A_{gv} - t_{pl} \cdot \left(5.5 \cdot \left(d + \frac{1}{8}\ \textbf{in}\right)\right)$

$A_{nv} = 19.563\ \textbf{in}^2$

$A_{nt} := t_{pl} \cdot \left(s_s + l_t - 1.5\ \left(d + \frac{1}{8}\ \textbf{in}\right)\right)$

$A_{nt} = 9.813\ \textbf{in}^2$

$U_{bs} := 1$

$U_{bs} \cdot F_u \cdot A_{nt} = 637.813\ \textbf{kip}$

$0.6 \cdot F_u \cdot A_{nv} = 762.938\ \textbf{kip}$

$0.6 \cdot F_y \cdot A_{gv} = 710.7\ \textbf{kip}$

$\phi R_n := 0.75 \cdot \left(U_{bs} \cdot F_u \cdot A_{nt} + 0.6 \cdot F_y \cdot A_{gv}\right)$

$\phi R_n = 1011.384\ \textbf{kip}$ $1011.38 > 406.23\ kip$

OK

Block shear (case 3):

$$A_{gv} := 2 \cdot t_{pl} \cdot (5 \cdot s + l_e)$$

$$A_{gv} = 51.5\ \mathbf{in}^2$$

$$A_{nv} := A_{gv} - 2 \cdot t_{pl} \cdot \left(5.5 \cdot \left(d + \frac{1}{8}\ \mathbf{in}\right)\right)$$

$$A_{nv} = 39.125\ \mathbf{in}^2$$

$$A_{nt} := t_{pl} \cdot \left(s_s - \left(d + \frac{1}{8}\ \mathbf{in}\right)\right)$$

$$A_{nt} = 8.375\ \mathbf{in}^2$$

$$U_{bs} := 1$$

$$U_{bs} \cdot F_u \cdot A_{nt} = 544.375\ \mathbf{kip}$$

$$0.6 \cdot F_u \cdot A_{nv} = 1525.88\ \mathbf{kip}$$

$$0.6 \cdot F_y \cdot A_{gv} = 1421.4\ \mathbf{kip}$$

$$\phi R_n := 0.75 \cdot (U_{bs} \cdot F_u \cdot A_{nt} + 0.6 \cdot F_y \cdot A_{gv})$$

$\phi R_n = 1474.33\ \mathbf{kip}$ $\quad\quad$ $1474.33\ kip > 406.23\ kip$

OK

Bearing type connection (tension & shear): AISC Steel Construction Manual Section J

$$e := 1\ \mathbf{in}$$

$F_{nt} := 113\ \mathbf{ksi}$ $\quad\quad$ (Table J3.2, Gr.B)

$$F_{nv} := 68\ \mathbf{ksi}$$

$\phi := 0.75$

$V_u := P_y$

$A_b := \frac{\pi \cdot d^2}{4}$

$A_b = 0.785 \; in^2$

$f_{rv} := \frac{V_u}{12 \cdot A_b}$

$f_{rv} = 43.102 \; ksi$

$\phi \cdot F_{nv} = 51 \; ksi$

$F_{nt'} := 1.3 \cdot F_{nt} - \frac{F_{nt}}{\phi \cdot F_{nv}} \cdot f_{rv}$

$F_{nt'} = 51.399 \; ksi$

$\phi R_n := \phi \cdot F_{nt'} \cdot A_b$

$\phi R_n = 30.276 \; kip$

$T := 6 \cdot \phi R_n$

$T = 181.658 \; kip$ $C := T$ *Tension=Compression*

$D := s + \frac{1}{2} \cdot s$

$D = 7.125 \; in$

$\phi M_n := T \cdot D + C \cdot D$

$\phi M_n = 2588.627 \; kip \cdot in$

$M_u := P_y \cdot e$

$M_u = 406.23 \; kip \cdot in$ $\phi M_n > M_u$

OK

B. Specimen Fabrication Drawings

Figures B1 and B2 detail beam-to-column connections. Figure B3 is a plan view of the RBS. Figures B4 to B7 show the coped and regular W14x22 beam connections. Figures B8 to B14 provide additional details on reinforcement and connecting elements such as doubler plates, continuity plates, shear tabs, and welding holes.

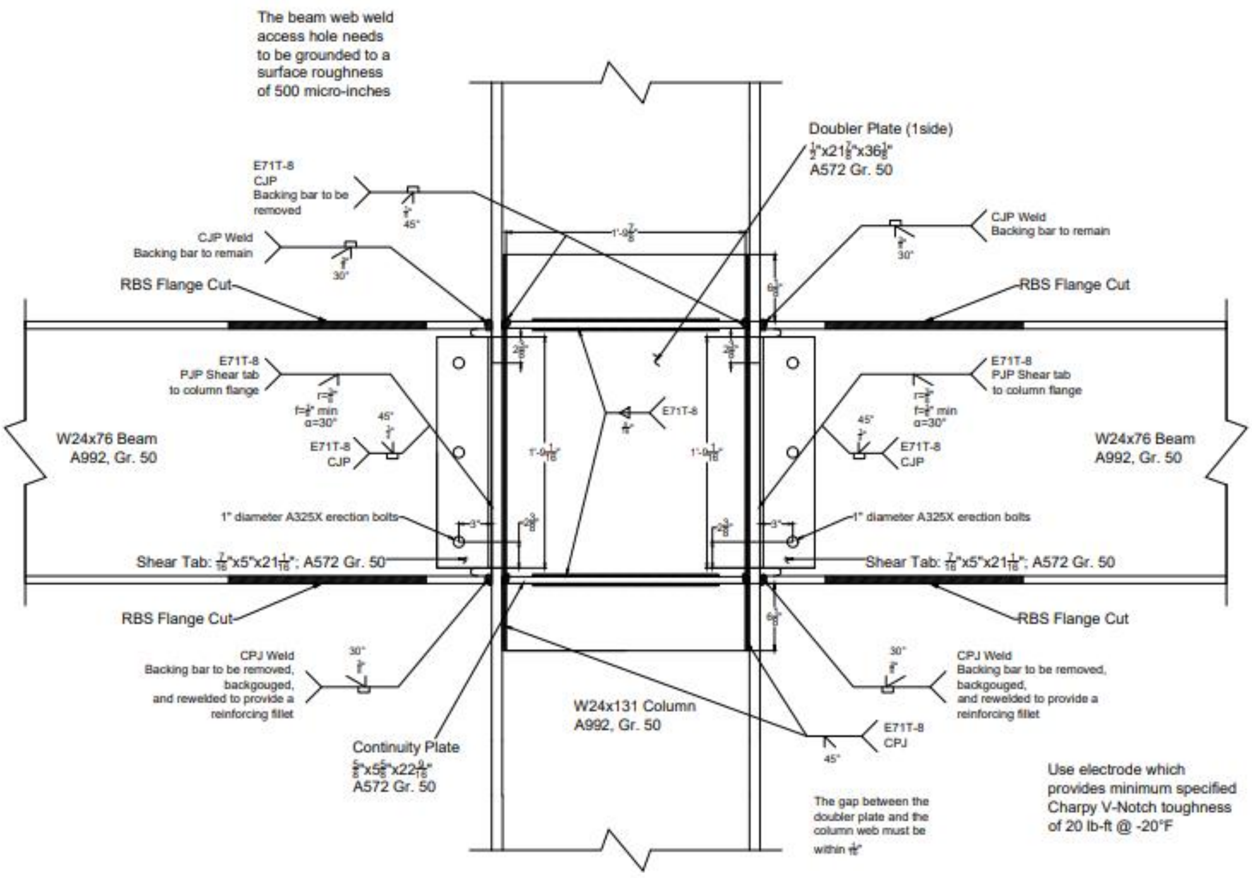


Figure B1. Detailed side view of the RBS moment connections.

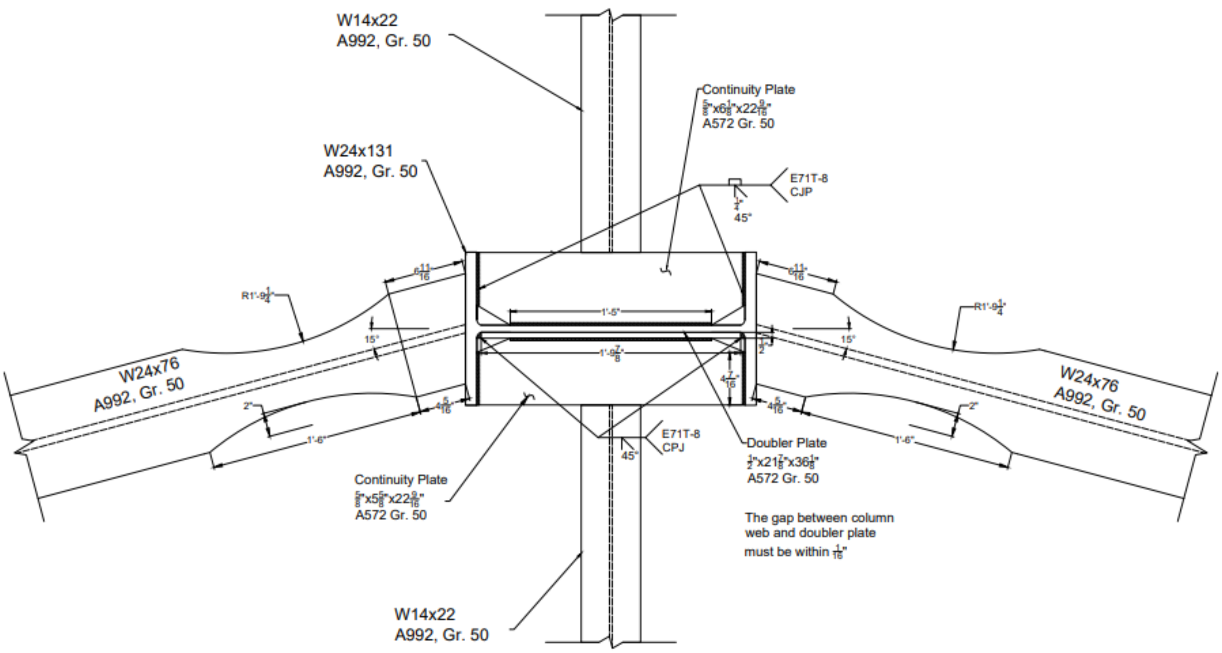


Figure B2. Detailed plan view of the RBS moment connections.

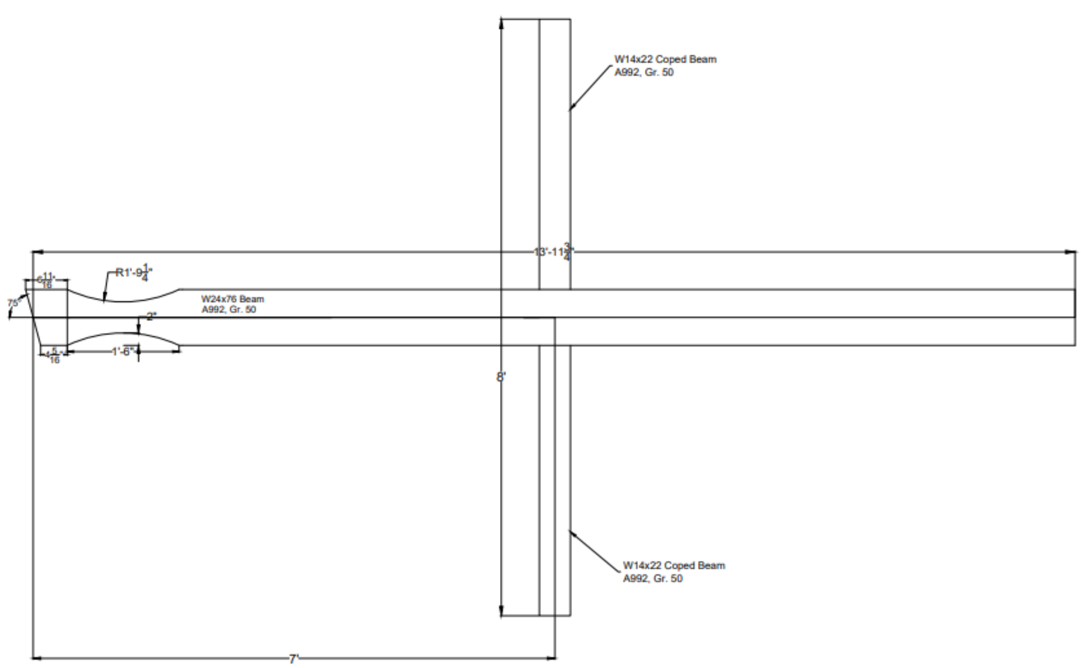


Figure B3. Plan view of the skewed W24x76 beam.

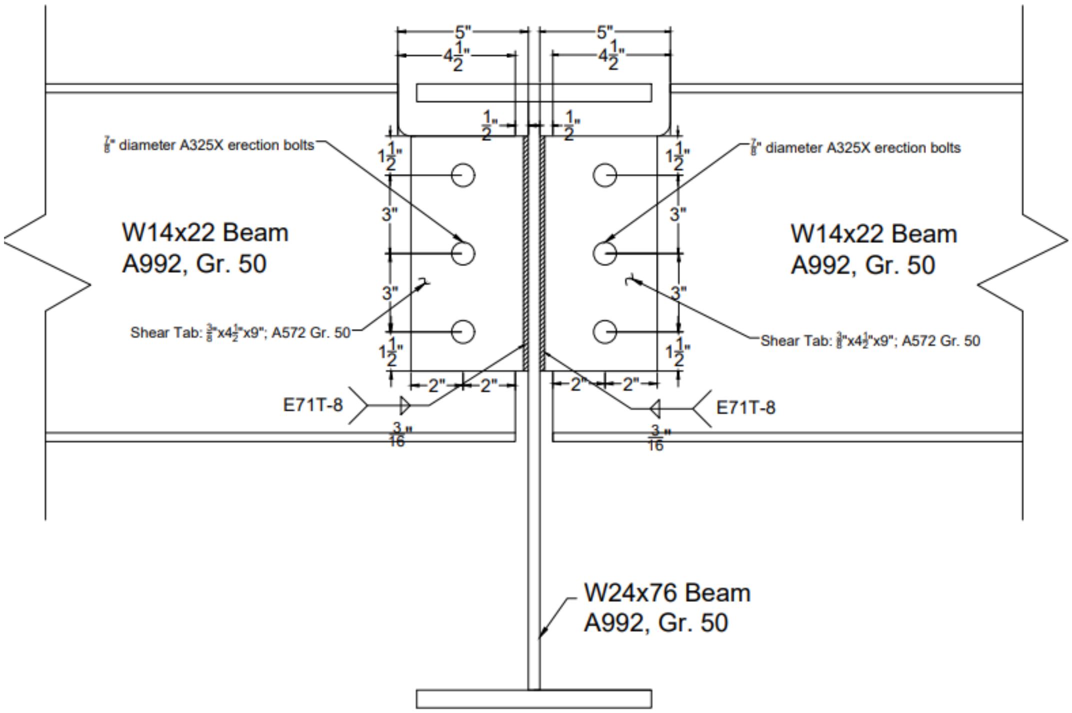


Figure B4. Side view of the double-sided W14x22 coped beam connection.

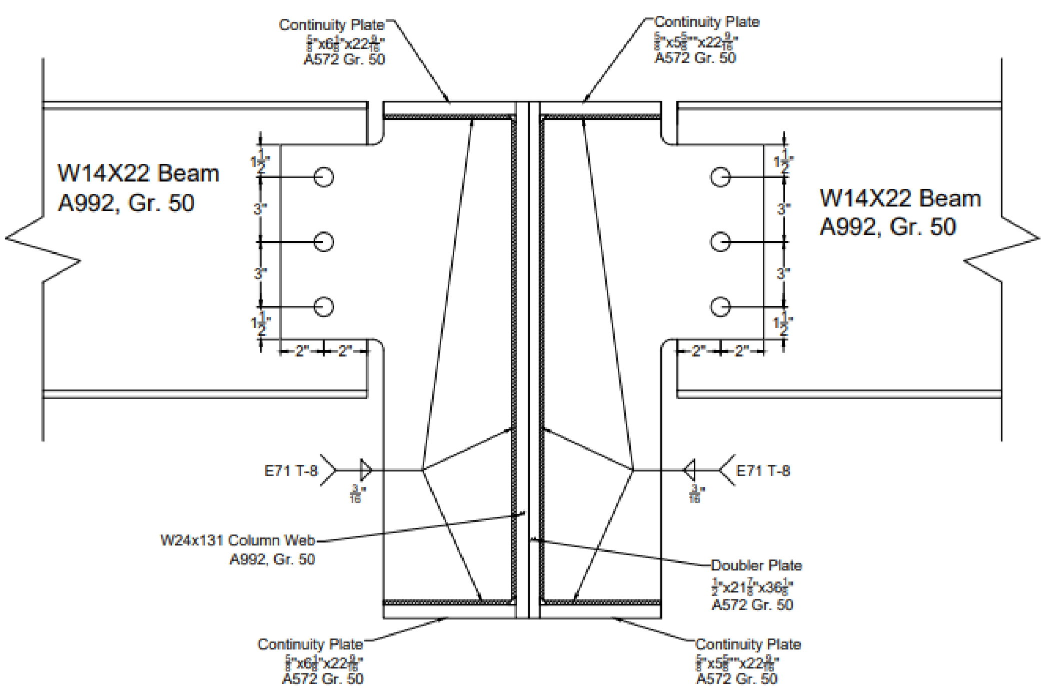


Figure B5. Side view of the double-sided extended shear tab connection.

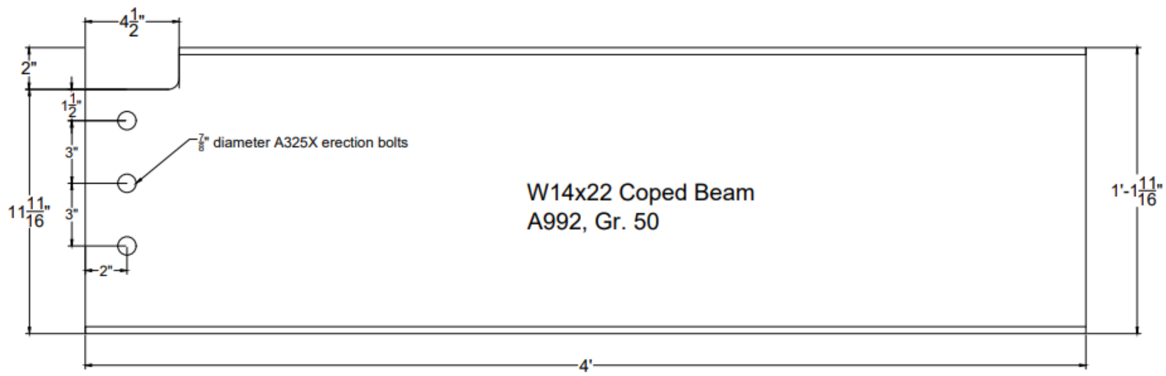


Figure B6. Side view of the W14x22 coped beam.

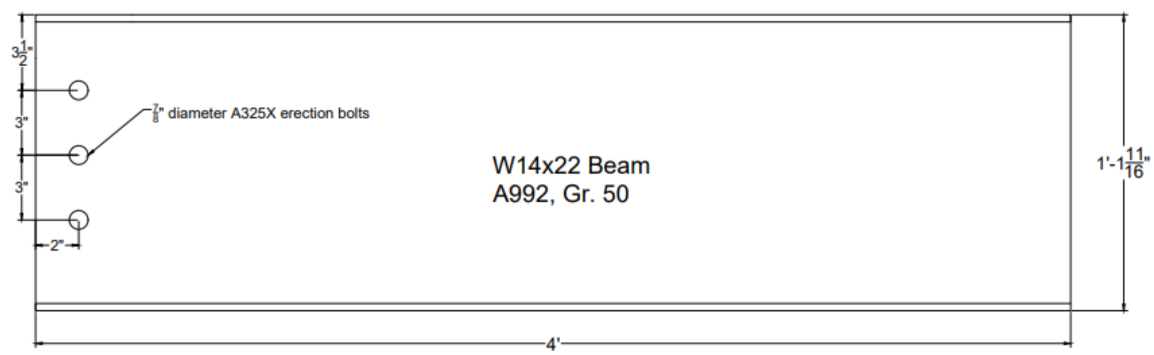


Figure B7. Side view of the W14x22 beam.

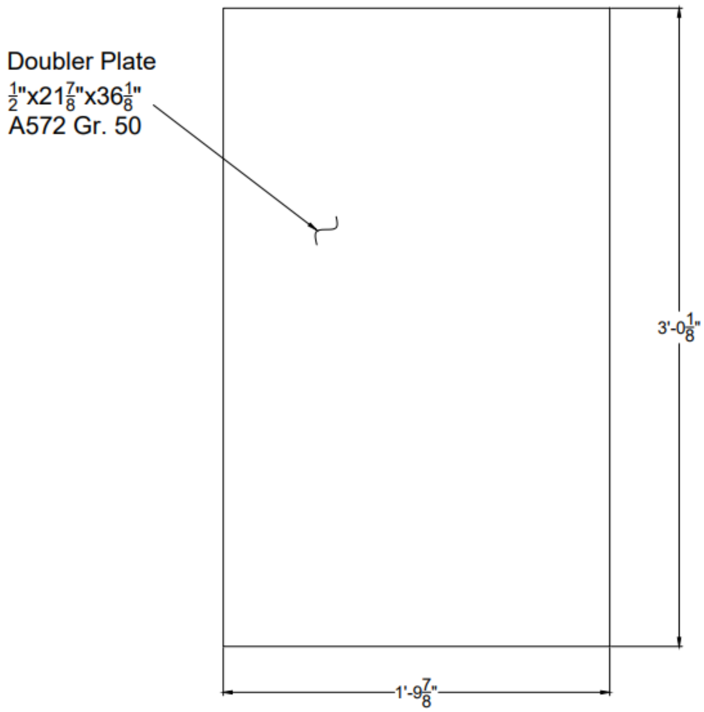


Figure B8. Column doubler plate.

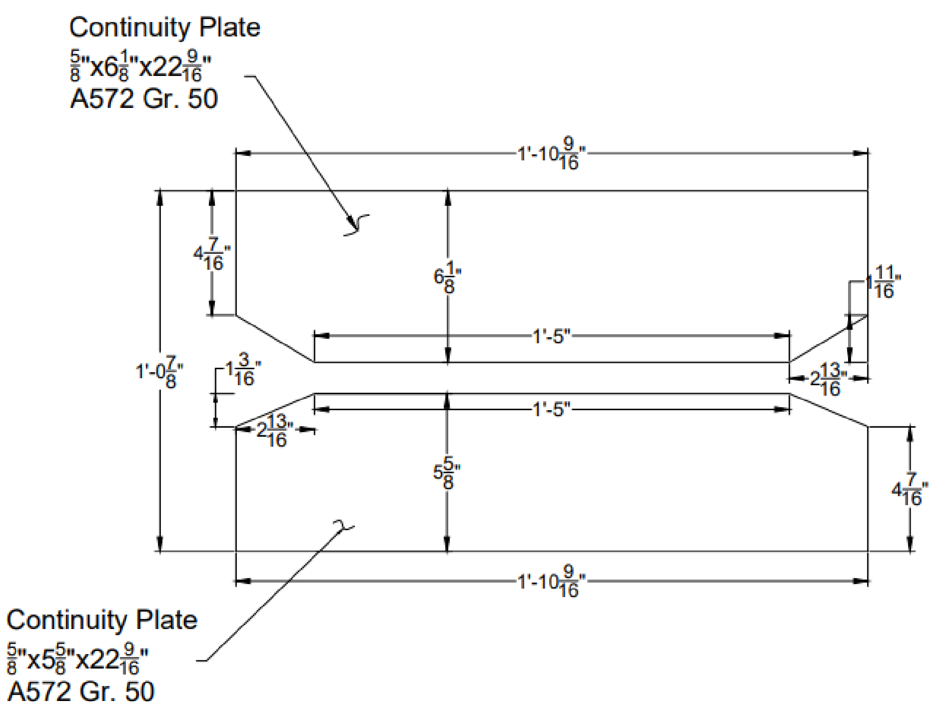


Figure B9. Top view of continuity plate configuration.

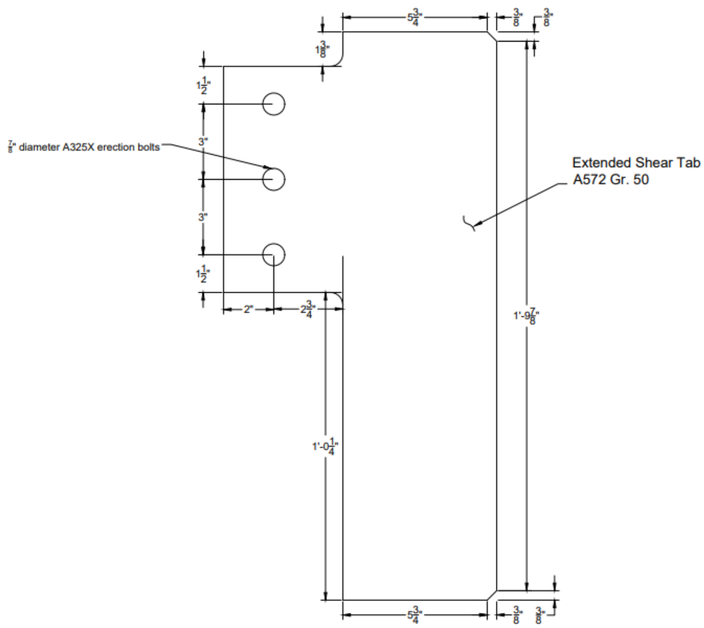


Figure B10. Extended shear tab 1.

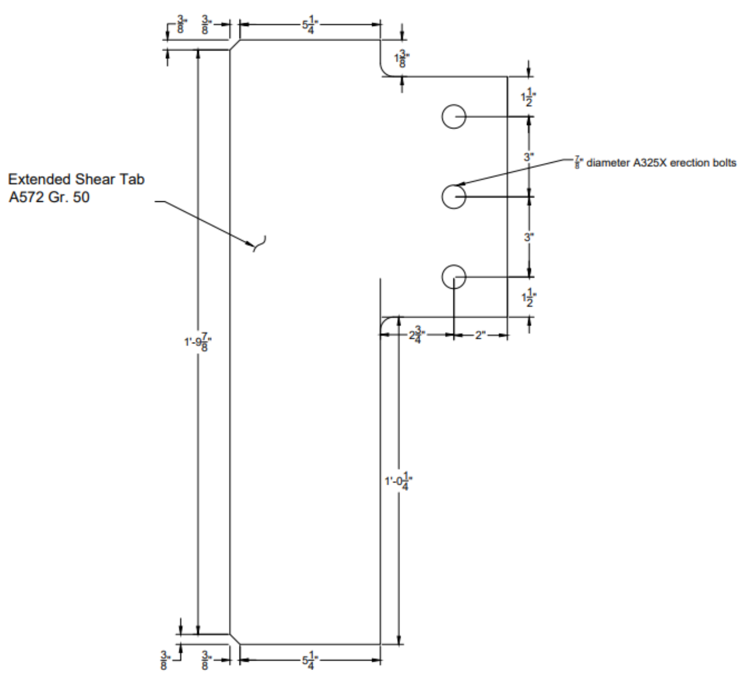


Figure B11. Extended shear tab 2.

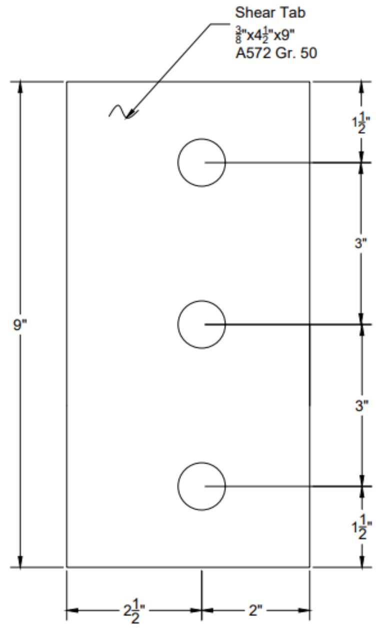


Figure B12. W14x22 beam shear tab.

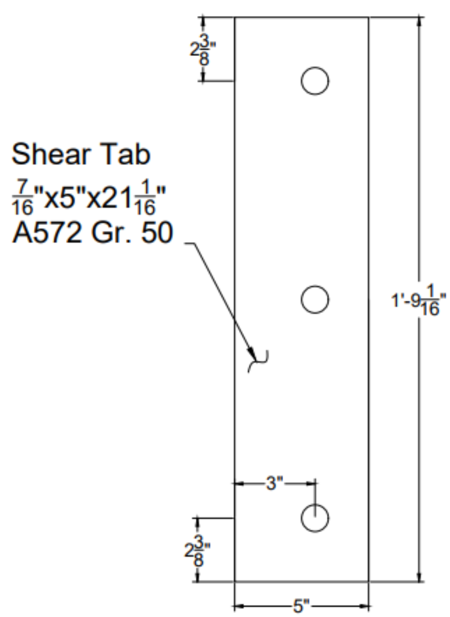
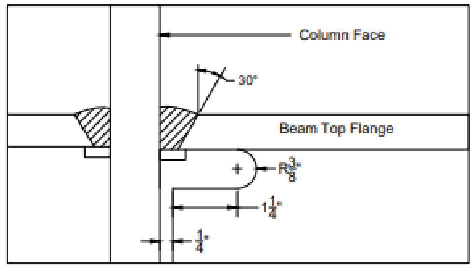
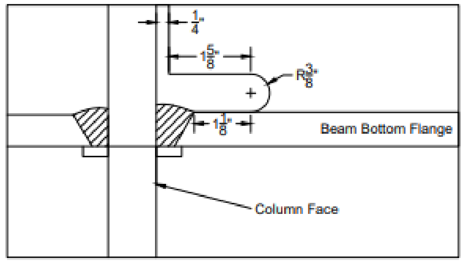


Figure B13. W24x76 beam shear tab.



Detail of Beam Top Access Hole



Detail of Beam Bottom Access Hole

Figure B14. W24x76 beam access hole details.

C. Bracing Fabrication Drawings

Figures C1 and C2 represent front views of the column flange plate with and without the gusset plates. Figures C3 and C4 show the whole brace connection. Figures C5 to C9 detail all brace connecting elements. Figures C10 and C11 display a side and top view of the HSS brace.

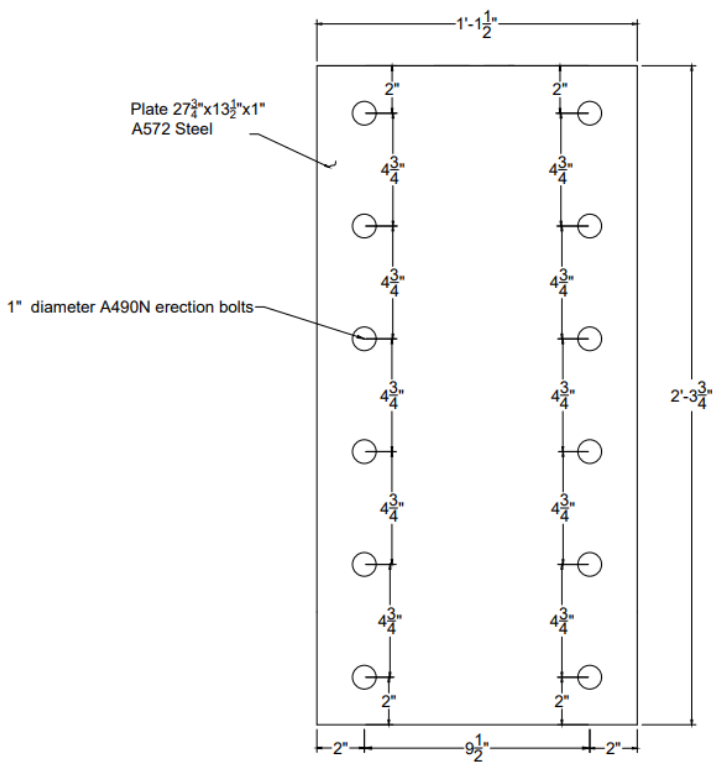


Figure C1. Column flange plate.

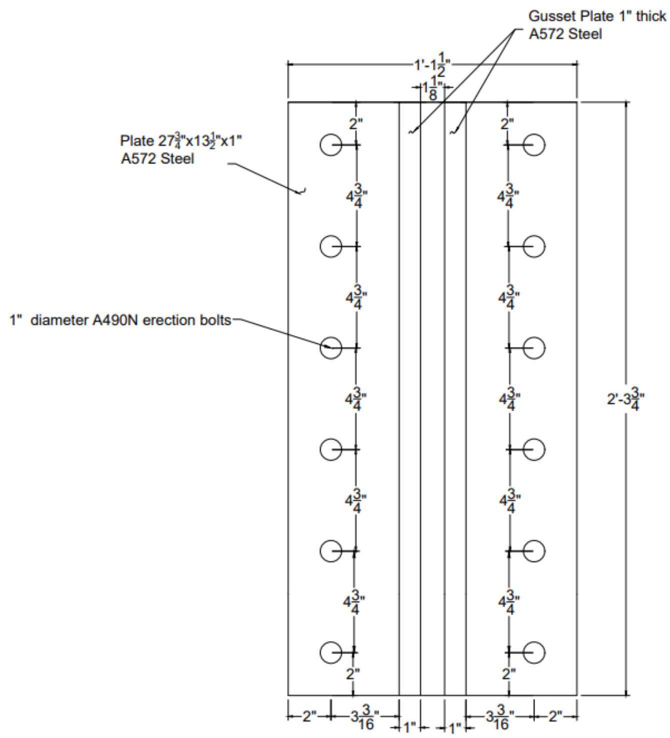


Figure C2. Front view of the column flange connection.

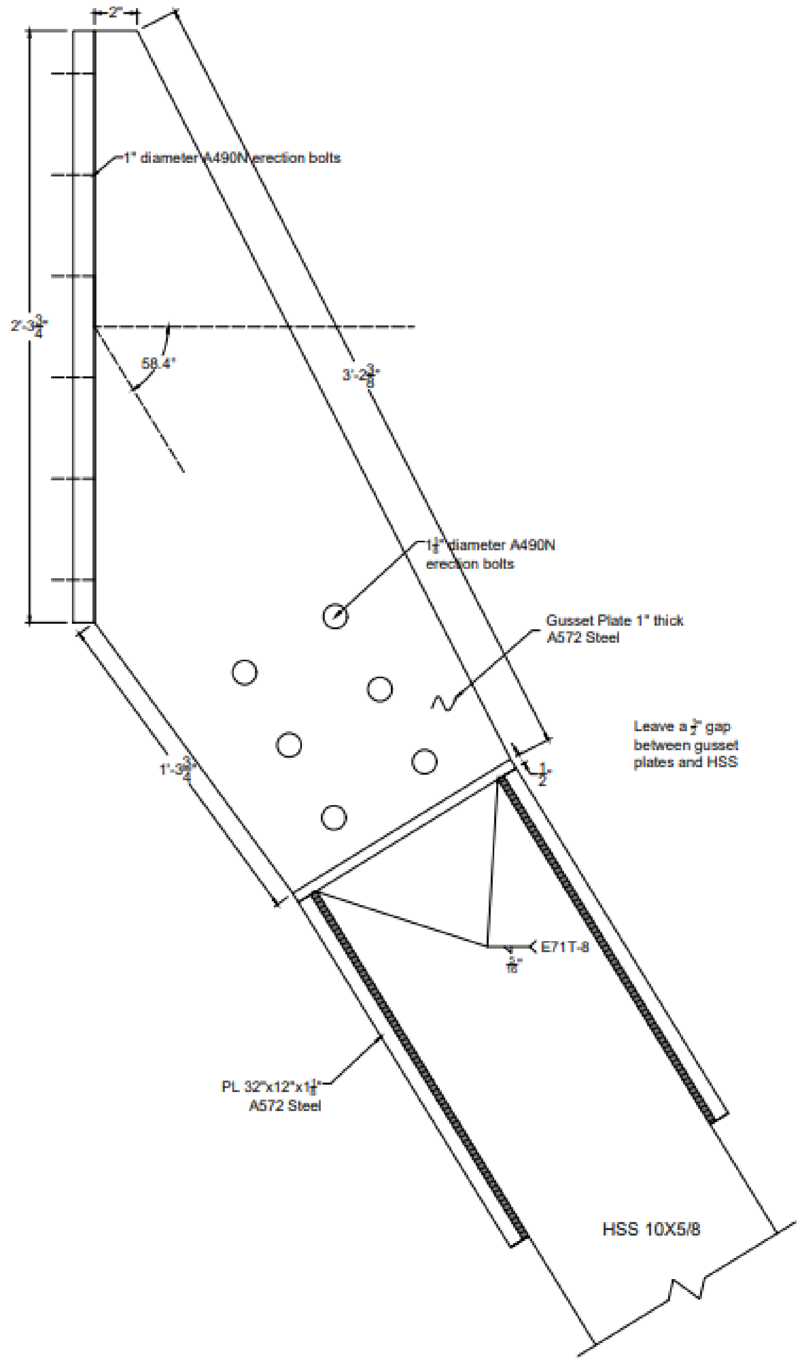


Figure C3. Side view of the top brace connection.

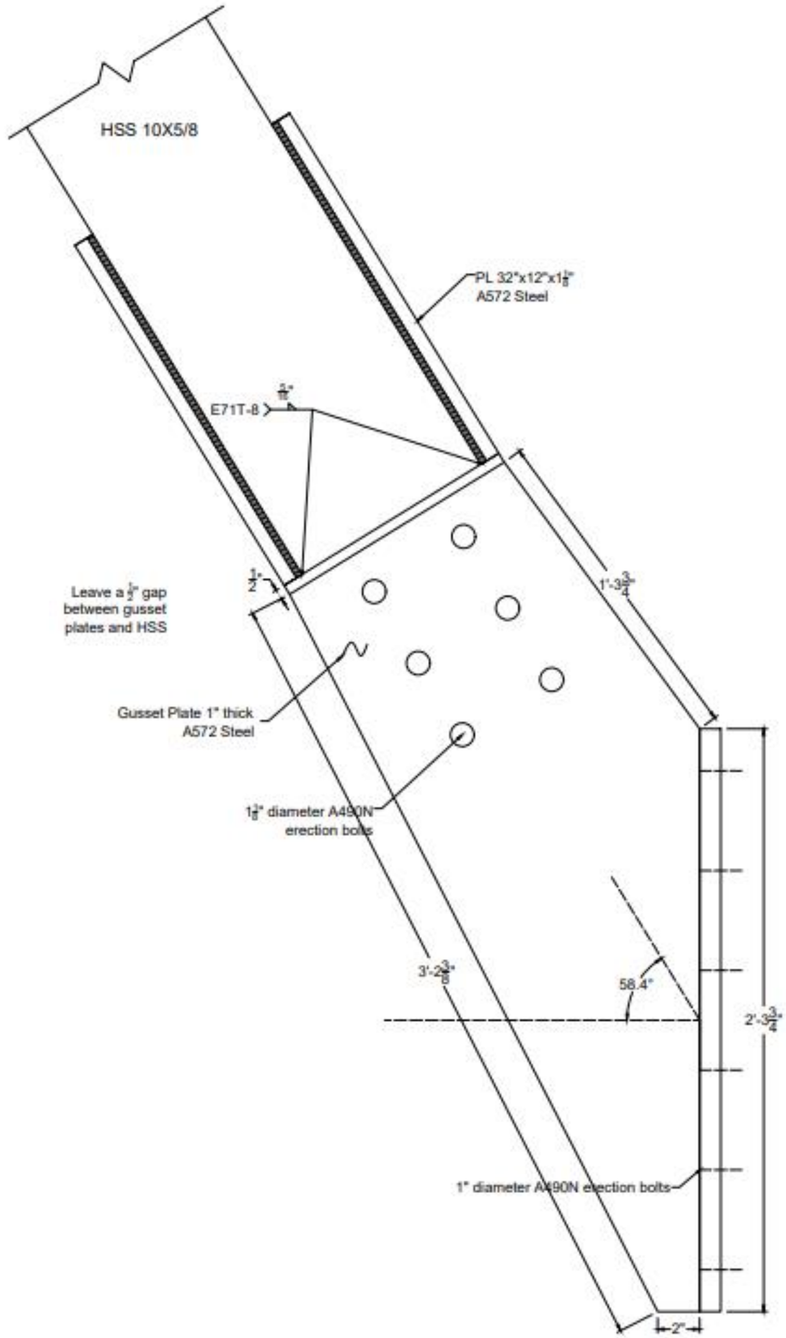


Figure C4. Side view of the bottom brace connection.

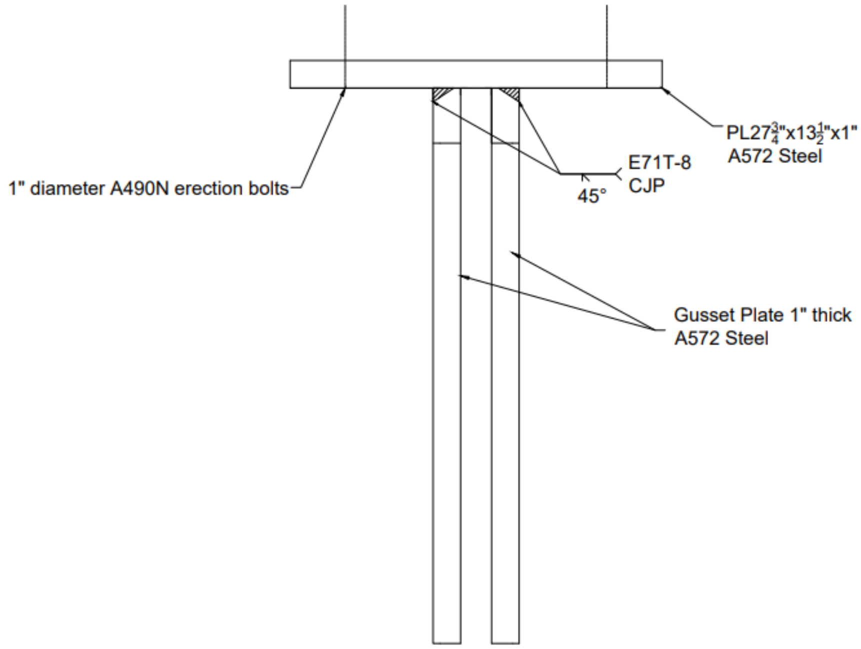


Figure C5. Top view of the column flange connection.

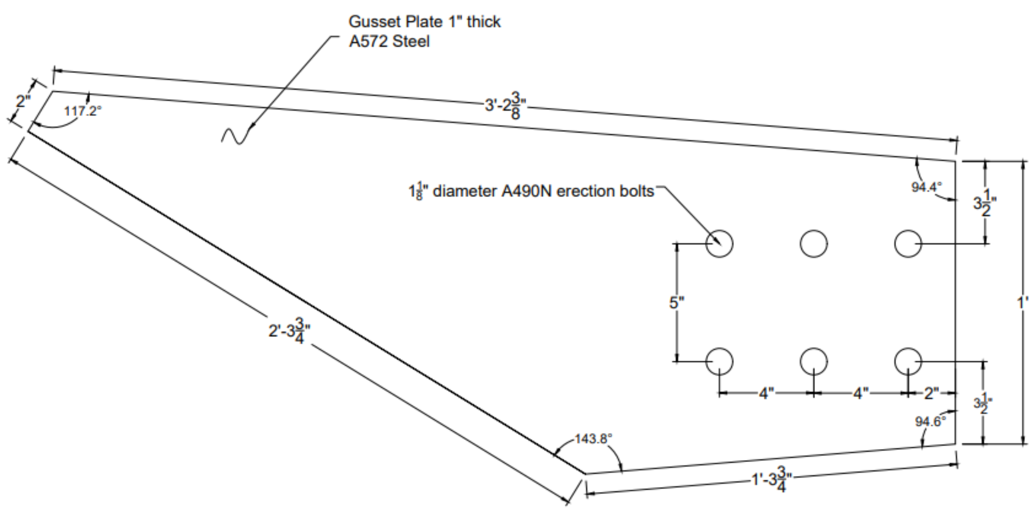


Figure C6. Gusset plate.

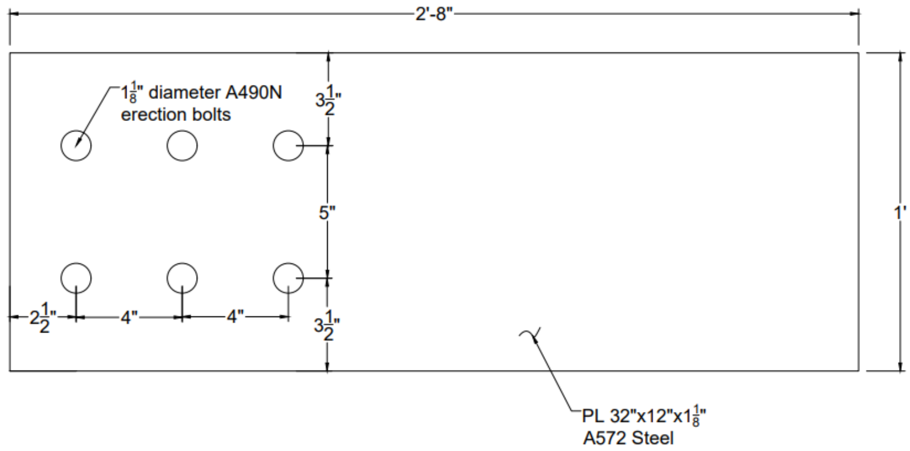


Figure C7. Steel plate.

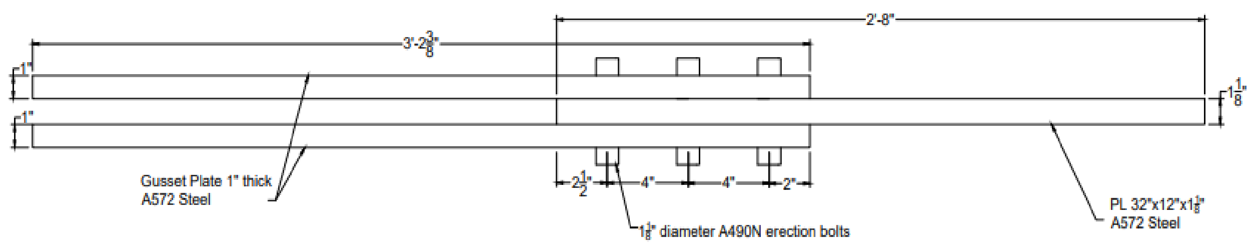


Figure C8. Top view of the steel-to-gusset plates connection.

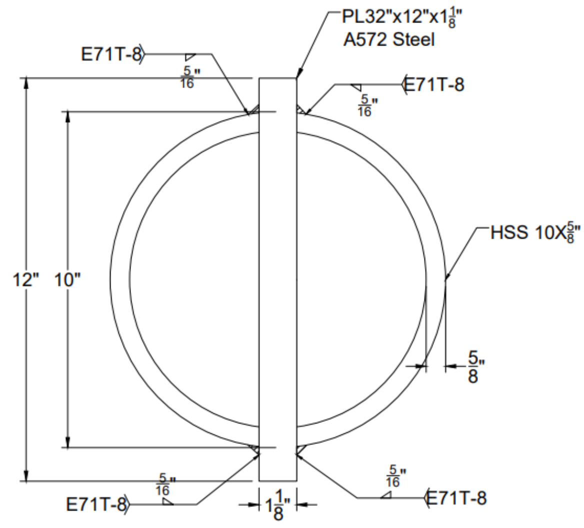


Figure C9. Cross section view of the steel plate within the HSS 10x5/8 tube.

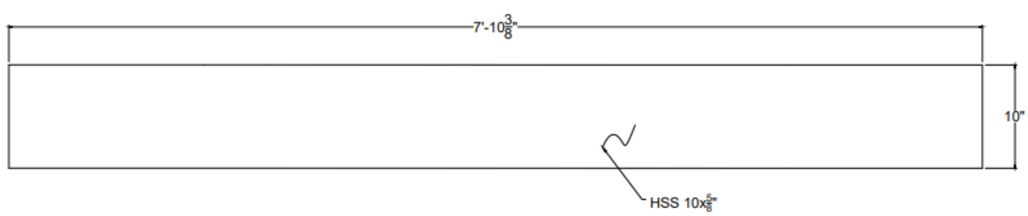


Figure C10. Side view of the HSS 10x5/8.

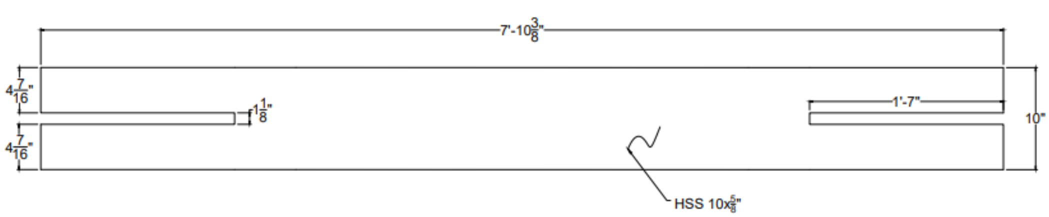


Figure C11. Top view of the HSS 10x5/8.