Final Design Report

NAU Steel Bridge Capstone 2025-2026

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List of Abbreviations

AISC ....American Institute of Steel Construction

ASCE ....American Society of Civil Engineers

AutoCAD ....Automatic Computer-Aided Design

CECMEE ....Civil Engineering, Construction Management, & Environmental Engineering

HSS ....Hollow Structural Sections

ISWS ....Intermountain Southwest Student Symposium

MathCAD..Math Computer-Aided Design

NAU ....Northern Arizona University

SI ....Sustainability Index

STAAD...Structural Analysis and Design Software

2-D ....Two Dimensional

3-D ....Three Dimensional

Acknowledgements

The 2025-2026 Student Steel Bridge Team would like to thank all our sponsors who contributed time or materials to this project. This project would not have been possible without Sabrina Gibson, Dr. Robin Tuchscherer, Mark Lamer, Page Steel, Copper State, and the ASCE Student Chapter at NAU.

The Steel Bridge Team would like to acknowledge Sabrina Gibson, the project's technical advisor. Sabrina met with the team and provided advice about modeling the bridge using software and connection calculations. Thank you, Sabrina!

Dr. Tuchscherer was our grading instructor; he was very active when it came to responding to emails about any issues or questions that the team had. He was also open to meetings whenever clarification was needed on certain aspects of the project proposal and report. Thank you, Dr Tuchscherer!

Mark Lamer was the project's client and put us in contact with people who would help us fabricate the bridge. He also set us up with companies that were willing to donate materials for the bridge. Additionally, Mark gave us example plan sets to help us derive our own for the project. Mark also gave us tips during our first meeting on some of the issues to look out for when working on this project. Thank you, Mark!

Page Steel was generous enough to help us by donating the steel members that were needed. Without Page Steel's donation, the team would not have been able to complete this project. Thank you, Page Steel!

Copper State Bolt & Nut was a great help to make this project possible. They donated all of the bolts and nuts that were needed to construct the bridge. Without their generous donations, this project would not have been completed. Thank you, Copper State!

The NAU ASCE club was a tremendous help with this project as they made it possible for us to participate in the competition; they planned the trip to Salt Lake City, Utah and provided information, food, and transportation. Thank you, ASCE!

1.0 Project Introduction

1.1 Project Overview

The goal of the NAU 2026 Steel Bridge Project is to create a 1:10 scale prototype for a 240-foot pedestrian bridge. This prototype will be tested at the ISWS competition in Salt Lake City, Utah. The results of this competition revealed the prototype's engineering efficiency, aesthetics, cost, and constructability. This model will be utilized to create the full-scale bridge in El Paso, Texas, which is a separate event from the NAU Steel Bridge Project.

1.2 Project Location

The model's 240-foot counterpart will be located in the northern region of El Paso, Texas. The exact location can be seen below in Figure 1-1. The bridge spans over the Rio Grande River, a little less than six miles north of the US-Mexico border, as a way of connecting the two communities currently separated by the river.

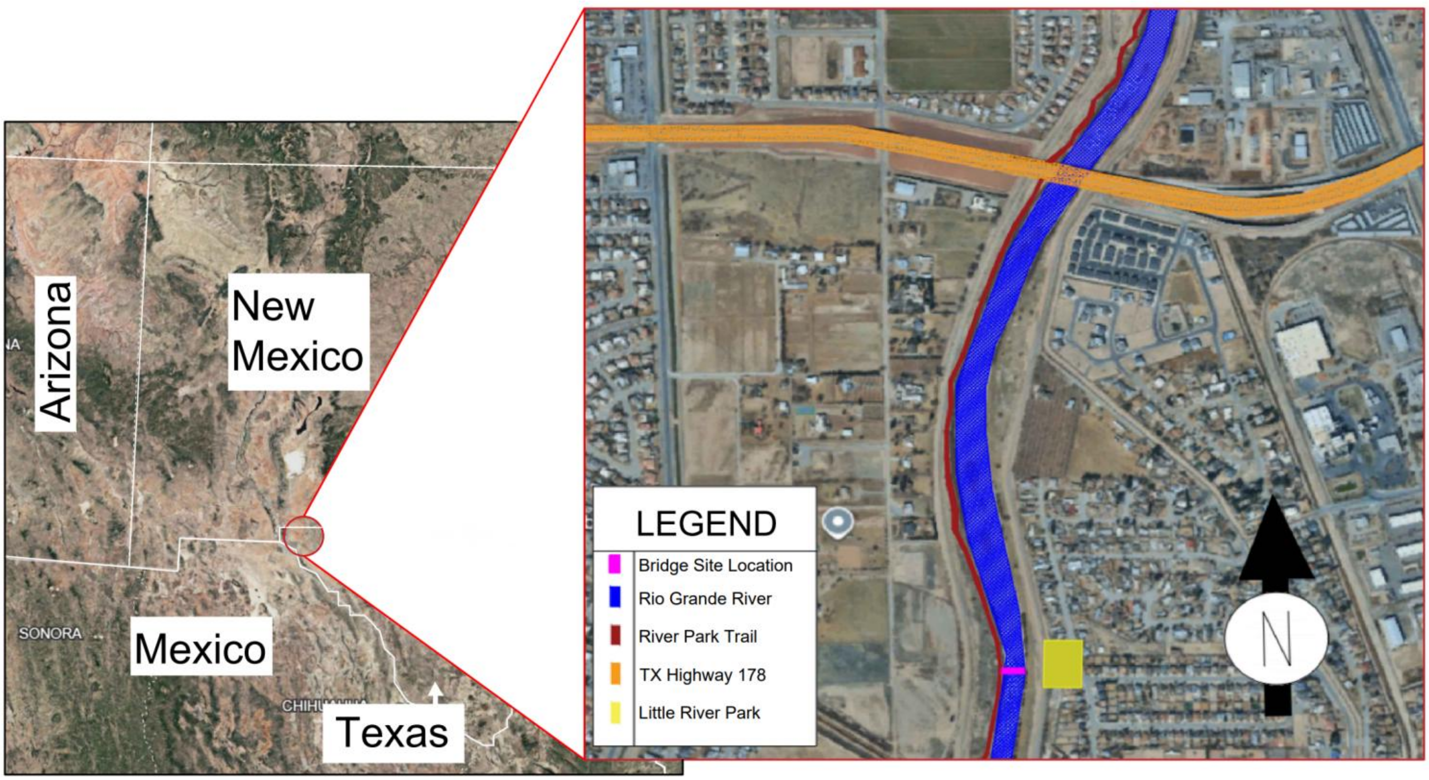

Figure 1-1: Vicinity Map

The River Park Trail to the West of the Rio Grande has a high volume of pedestrian traffic, which can only cross the river through the Texas Highway 178 overpass. This pedestrian bridge provides a safer option for the public while also avoiding conflict with those who kayak in the Rio Grande due to the dimensional constraints. The bridge cannot have footing in the riverbed and must be within a certain envelope, which allows for kayaking to continue. This is further described in the project constraints.

1.3 Project Constraints

Project constraints included specific requirements for the bridge dimensions and construction restraints. Among the many dimensional specifications, the clearance of the bridge was a controlling restraint. The Rio Grande River allows for kayaking and other recreational activities, so the bridge was required to be high enough to not interfere with these activities [1]. Another challenge was the construction parameters. To prevent erosion or disturbances to the area, construction could not take place around the riverbanks and temporary piers could not be used due to soil conditions [1]. In addition to this, construction would also be timed for the purpose of testing construction efficiency. The speed at which the bridge could be constructed ultimately affected how many steel members could be used and how many bolted connections would be in the final design.

2.0 Background Research

2.1 Truss Type Research

Due to strict dimension requirements, Beyond Bridges determined that the best bridge type for this project was an underslung truss bridge. The truss types that were found to be the most applicable to the project include: Warren, Pratt, and Howe.

The Warren truss has a repetitive triangular shape which spans across the side panels of a bridge, an example of a Warren truss can be seen in Figure 2-1. This truss type accommodates many spans and can incorporate vertical members depending on the loads placed on the bridge. Along with this, the diagonal members allow dynamic loads to be easily supported while providing buckling resistance. This is due to the alternating tension-compression forces created from the repeated diagonal members, as well as the relatively short compression members when concerned with buckling [2].

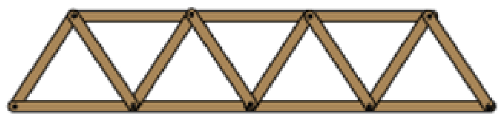

Figure 2-1: Warren Truss [3]

Howe and Pratt trusses are similar as they are both composed of vertical and diagonal members, as shown in Figure 2-2. However, they differ in the direction their diagonal member faces; this affects the internal forces within each member. Howe trusses have diagonal members in compression while the vertical members are in tension. Similarly, Pratt trusses have diagonal members in tension, and the vertical members are in compression within the truss system [4]. Both the Howe and Pratt trusses do not support lateral loads well but are reliable with vertical loads [5]. These trusses are cost efficient in terms of manufacturing and constructability in comparison to the Warren truss.

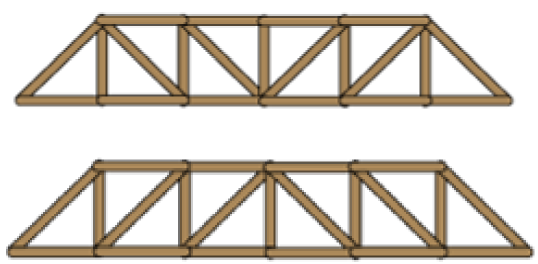

Figure 2-2: Pratt and Howe Trusses [3]

2.2 Material and Member Research

The material research conducted by Beyond Bridges resulted in a debate between carbon steel A500 and A36 for the members and plates. These two types are the most applicable within steel design, as displayed in Tables 2-1 and 2-2 in the AISC Steel Manual [6]. Steel type A500 Grades B and C would be preferred for HSS members, according to the previously stated tables, while plates are typically A36. Along with the plates, M-, S-, HP-, C-, MC-, and L-members are preferably A36 steel. W-members are typically A992 steel, but W-members are not optimal for small scale projects. It must also be stated that Page Steel, the company which is supplying steel for the project, did not have W-members small enough for the prototype bridge. Page Steel has a wide selection of steel members, but the prototype bridge requires steel to be small enough to decrease the choices significantly.

Beyond Bridges found that Page Steel could readily provide HSS members small enough to be relevant to the project. HSS members have a high strength to weight ratio with the ability to span longer lengths while also being common in smaller projects [7]. The closed sections that these members provide resist torsional loads while also improving aesthetics. When aesthetics are included in the consideration of members, the limited surface area found in these members doubles to reduce dust collection over time. This information led to a decision between a rectangular HSS member and a circular HSS member for the framework of this project. The research done in relation to these two member types included analyzing previous NAU Steel Bridge Projects and finding documents which described the best uses for each HSS type.

Previous years used rectangular HSS members with a select few utilizing circular members. A rectangular shape allows for flat surfaces, which increases the constructability due to an easier alignment while simultaneously making connections simpler and stronger. Cuts become easier when rectangular members are used because measurements and securing of the member are more reliable. These members are used when resisting bending because of their higher moment of inertia. In comparison, circular members excel when applied to cases where torsional forces are concerned, such as trusses and frameworks, due to the uniform stress distribution. This uniform distribution also increases the quality of thermal and transportation pipe systems [8].

2.3 Connection Research

The two connections that Beyond Bridges considered were welded and bolted connections. Welding fuses steel members together through a method of melting and cooling. The benefits of this fusion result in greater strength, aesthetics, and stiffness. These benefits are accompanied by the disadvantage of a higher cost from skilled labor and specific equipment, as well as a higher possibility of cracking and fatigue failure when exposed to weather.

Bolted connections involve connecting members through nuts and bolts by plated connections or directly bolting members together. The advantages of using bolts include faster installment, lower costs, accommodation for deformations, easier inspections, higher field flexibility, and connection flexibility. Bolted connections also introduce weaknesses from the bolt holes, have limited member application, and are less efficient in transferring loads in comparison to welded connections [9]. When considering bolts, it was found that A325 and A307 carbon steel are most common [6]. The AISC Steel Manual states that bolts with steel A325 are only available for diameters at half an inch and up. A307 bolts have a wider range of diameters available [6].

The AISC failure modes affiliated with bolts and welds also differ. Welding members together requires minimum steel thickness, weld sizes, and weld lengths to be calculated to prevent the members from failing due to heat deformation. The most common welded connection, a fillet weld, has the limit states of fillet weld rupture, base material shear, tension member limits, and weld connection capacity.

Bolted connections have clearance and spacing requirements for the connection to be considered for further calculations. The limit states that bolted connections have are as follows: bolt tension failure, bolt shear failure, bearing and tearout failure, and base material failure in terms of tension, shear, and block shear.

When bolting or welding HSS members together, Section-K in the AISC Steel Manual includes more calculations that must be performed to confirm the structural stability of the prototype. The new limitations include end lengths and more welding minimums as found in Table K5.1 [6].

Shop drawings from previous steel bridge projects at NAU were utilized to further understand the applicability of each connection. HSS members were bolted together through plates rather than directly bolting members together, as found in the 2017-2018 shop drawings. Certain plates were welded to the HSS members, which would ultimately increase constructability on site. The thickness of the plates was observed to range between 1/8 inches to 5/16 inches [10]. Meanwhile, welding members to each other was limited to smaller connections due to dimensional constraints from competition rules. Welds permanently fuse members, so they create a larger member than the original components.

3.0 Initial Design

3.1 Preliminary Sketches

The first step of the design process was to sketch different ideas for the prototype based on the cumulation of knowledge found during background research and dimensional restrictions provided by the competition rules.

The two figures below show different versions of Pratt trusses; they both have the same height and width, however, have different vertical clearances and cross-section designs. The purpose of the preliminary sketches was to test different ideas and explore a range of possible options for the design of the bridge.

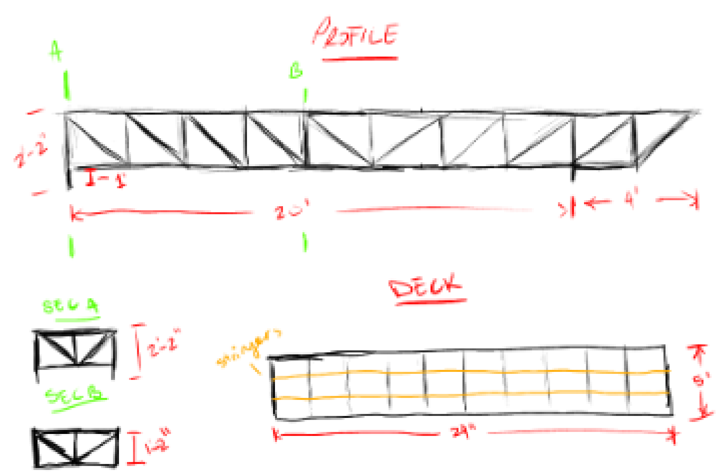

Figure 3-1: Pratt Truss Sketch #1

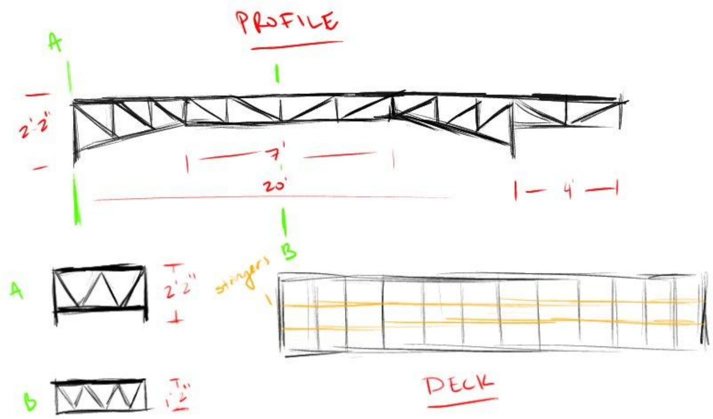

Figure 3-2: Pratt Truss Sketch #2

Throughout the preliminary sketching phase of the project, numerous ideas were considered or rejected based on how many members it would require. More members determined how complex the connection design would be to construct and analyze. During this process the decision was made to have the top chords of the bridge act as the stringers needed for competition, making the bridge narrower, and reducing the number of members needed. This change can be seen in the figure below.

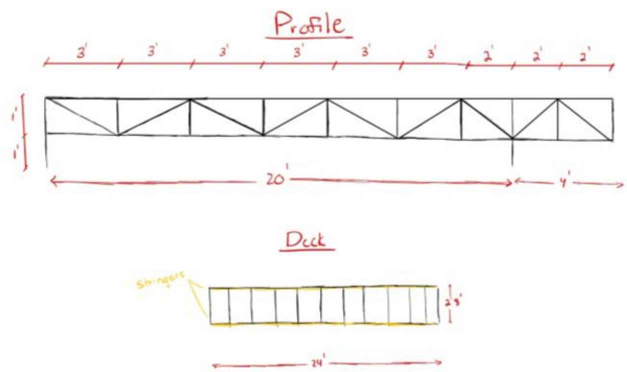

Figure 3-3: Warren Truss Sketch

3.2 Member & Connection Selection

When reviewing the research done in Section 2.0 as well as the sketches discussed in Section 2.2, it was determined that rectangular HSS members made of A500 steel would be the best fit for the prototype. Rectangular HSS members are best suited for the prototype because of their bending resistance and their flat surfaces. Beyond Bridges will be utilizing many bolted connections and will be welding plates to the members. These conditions work well with the flat surface areas of rectangular HSS members.

Due to the strict dimensional constraints for the member sizes, height, and clearance requirements, several of the bridge designs considered during preliminary sketching were ruled out. This led to a focus on bridges with similar dimensions and members, with the main difference being the type of truss. This would impact how each design would handle the different load cases or distributed loads within each unique system.

On each bridge design, most of the connections consisted of bolting members together with plates on both sides of the HSS members. Bolt connections were required for each bridge due to the requirement that all members must fit in a 3'6" x 6" x 4" box, which means that the members are short and cannot be welded together [1]. While members cannot be welded together, plates can be welded to members to limit the number of bolts required as well as increase construction speed by reducing the number of individual parts of the bridge.

3.3 Modeling & Analysis

After considering the feasibility of the preliminary sketches based on the member and connection selection information, four of the preliminary sketches were created in Risa-2D. The Risa-2D models used pinned connections for the legs and made members pinned on each end to simulate bolt connections. The legs of the bridge will not be tested as fixed during the competition, thus making the ground connection pinned.

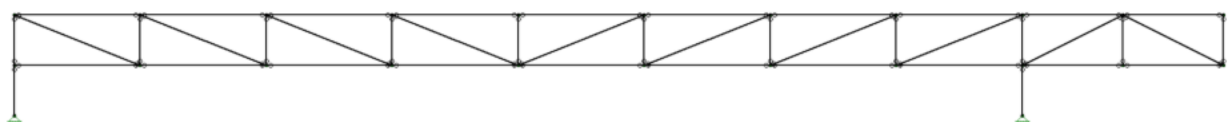

Figure 3-4: Pratt Truss Model #1

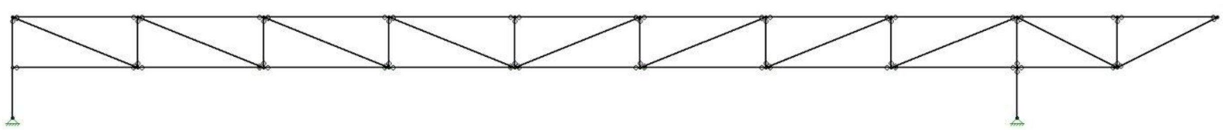

Figure 3-5: Pratt Truss Model #2

Using Risa-2D provided the opportunity to make small changes to the models to see how they would affect deflection. For example, Figure 3-4 and Figure 3-5 are very similar; both are Pratt trusses with the same size members for the main span of the bridge. The differences between the designs are found in the cantilevers, with an alternative orientation for the diagonal members. This allows for Figure 3-5 to reduce the total number of members by two in this side panel's cantilever.

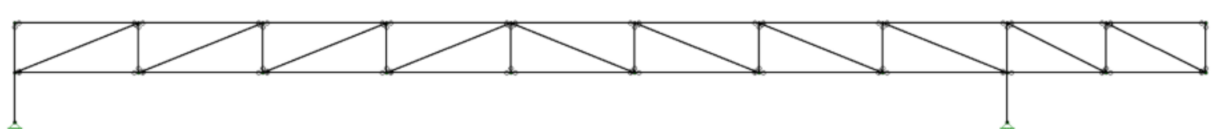

Figure 3-6: Howe Truss Model

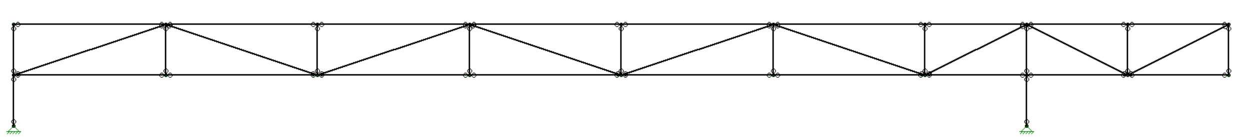

Figure 3-7: Warren Truss Model

Each design handled the loading differently based on the orientation of the diagonal members. When comparing the various 2-D models, the Warren truss reported to have the least vertical deflection, whereas the Howe truss had the most vertical deflection. The maximum vertical deflection in the ISWS competition is 3 inches before disqualification of the bridge, so all values calculated by the Risa 2-D model were ensured to not exceed this maximum.

3.4 Design Selection

Beyond Bridges utilized information gathered from the background research, the Risa-2D results, and the competition parameters to consolidate the prototype side panel options. This left three remaining possibilities which would be further reviewed through their constructability, deflection, and aesthetics. A decision matrix was created with the previously listed criteria in which the highest scoring of the three designs would determine the best model to move forward with. The results of this can be seen in the table below, with the warren truss evaluated to be the best design.

Table 3-1: Decision Matrix

	Weights	Option 1 – Warren Truss	Option 2 – Pratt Truss 1	Option 3 – Pratt Truss 2
Constructability	40%	10	9.62	9.77
Deflection	45%	10	8	9
Aesthetics	15%	9	10	8
Total Weighted Score	100%	9.9	8.9	9.2

The weight for each criterion was decided based on the estimated importance of each competition scoring category. The deflection weight was decided to be 45%, as it is fundamental to the purpose of a bridge. It consists of how well the bridge will perform under the vertical loading test conducted on Competition Day; these scores were based on deflection readings from the Risa-2D models.

Constructability was an important criterion as it has a direct connection to the competition's final scores. The bridge needs to be relatively easy to assemble during the competition; therefore, the scores were determined by the number of members and connections. The calculations done to evaluate the constructability score can be seen in the table below.

Table 3-2 –Connection & Member Scores

	Option 1 – Warren Truss	Option 2 – Pratt Truss 1	Option 3 – Pratt Truss 2
Number of Connections	22	24	24
Equation	$\frac{100-22}{100-22}*10$	$\frac{100-24}{100-22}*10$	$\frac{100-24}{100-22}*10$
Score	10	9.74	9.74
Number of Members	39	43	40
Equation	$\frac{100-39}{100-39}*10$	$\frac{100-43}{100-39}*10$	$\frac{100-40}{100-39}*10$
Score	10	9.34	9.84

Table 3-3 - Constructability Calculations

	Weights	Option 1 – Warren Truss	Option 2 – Pratt Truss 1	Option 3 – Pratt Truss 2
Connections	70%	10	9.74	9.74

Members	30%	10	9.34	9.84
Score	100%	10	9.62	9.77

Deflection was determined based on how each model performed in Risa-2D under the vertical loads. Therefore, the Warren Truss received the best score because it had the least amount of deflection, followed closely by Pratt Truss 2, with Pratt Truss 1 having the most deflection.

Lastly, aesthetics had a weight of 15%, as the full-scale bridge would be in the middle of two communities and well-traveled by pedestrians. The competition and the public consider aesthetics an important criterion when contemplating potential designs. The aesthetics scores were decided based upon a team evaluation of how each design looked.

4.0 Final Analysis and Design

4.1 *Final Analysis*

The final analysis process involved developing the selected Warren truss design from the decision matrix within STAAD, a 3D structural analysis software. A boundary condition of a pin and a roller was the primary condition, as it models realistic behavior and makes calculations simpler due to its statical determinacy. The team also modeled a pin-pin boundary condition to assess the influence it would have on force distribution and displacement for the truss. As for the member connections, most of them were assumed to behave as a pinned connection to reflect realistic bolted connections. This assumption allows for easier analysis and realistic fabrication.

Once the model was completed in STAAD, loading scenarios in accordance with the ISWS competition were modeled within the software. There were two types of loading categories that were tested at the ISWS competition; these include lateral and vertical loading.

The lateral loading evaluates the prototype's resistance to sway. For this, two loading scenarios were implemented and analyzed: One load at midspan and another load at the cantilever end of the bridge. Each of those scenarios includes 50 pounds of horizontal force being applied either in the North or South direction being determined by a dice roll. A 75-pound vertical point load will also be applied at the midspan of the bridge during either scenario; this load type can be seen in Figure 4-2. The second load type, vertical load, was applied to test the bridges' resistance to deflection. The test includes 1600 pounds of weight being vertically applied to the main midsection of the bridge and a 900-pound vertical load applied to the cantilever end of the bridge, as shown in Figure 4-1. The loading applied to the main portion of the bridge would be decided by a dice roll. Stated in the rules, the dice roll could have 11 potential locations where the loading could be applied. All possible load combinations were modeled into STAAD to ensure that the bridge can withstand every load case. This was also important to ensure the bridge meets all the deflection and sway parameters. If vertical deflection exceeds 3 inches or sway exceeds ¾ inches, then the team would be penalized in the competition [1].

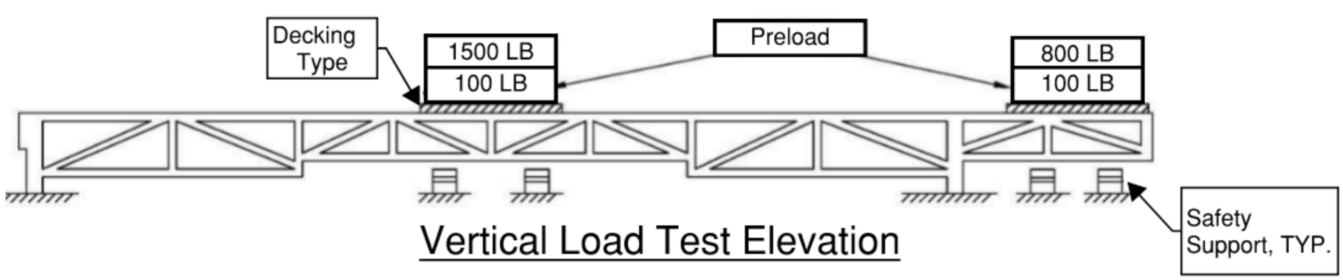

Figure 4-1 - Vertical Load Test [1]

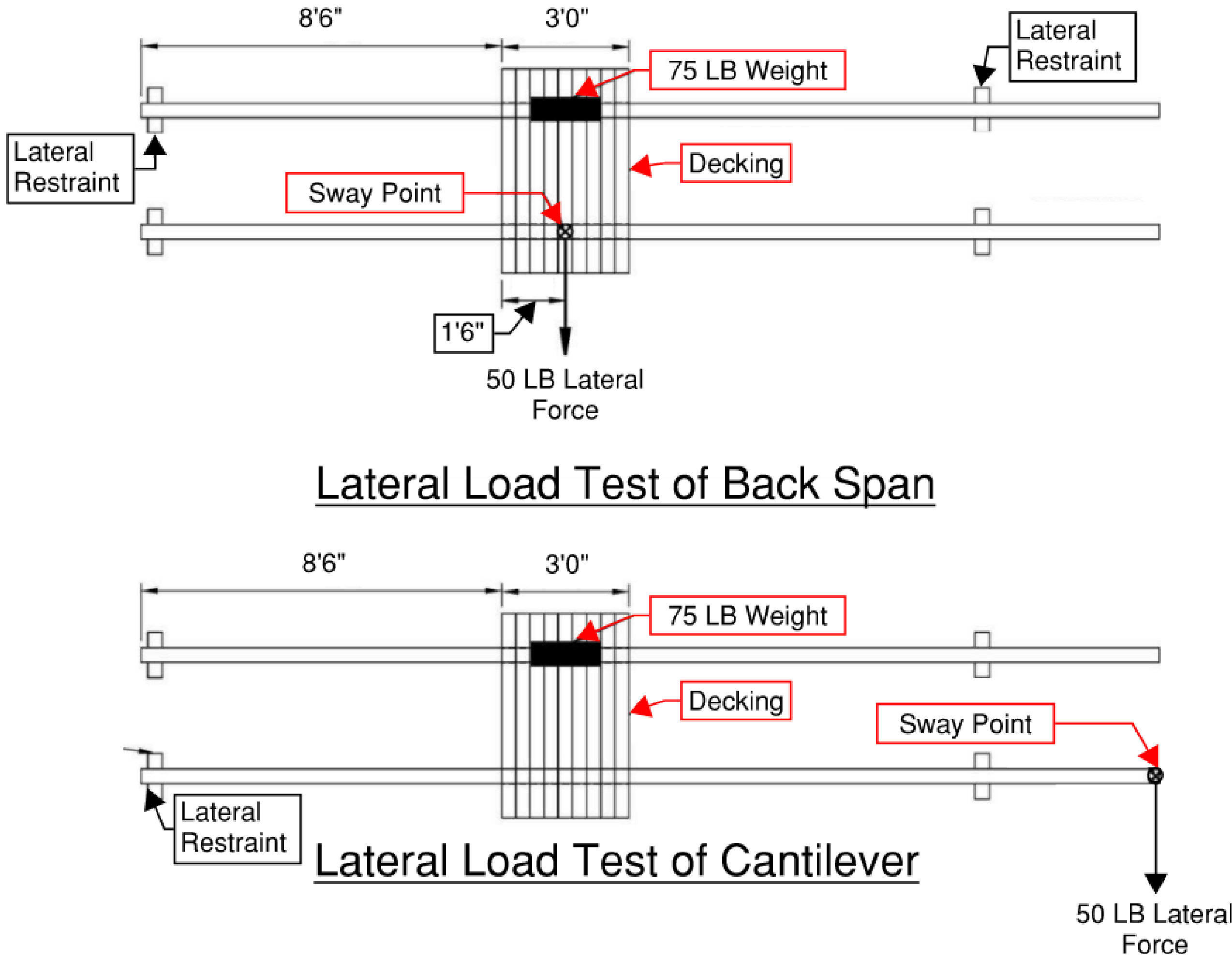

Figure 4-2- Lateral Load Test [1]

Defining the members and their properties was the final step to ensure that all software results were accurate. STAAD has a software database with various member types and properties which can be easily integrated into the model. After setting up the properties, the model could then be simulated. After running the simulation, the model met all requirements in the deflection category as there were no deflections over 3/8ths of an inch. However, the sway loads were found to have instability issues. The team determined that the bridge would need lateral bracing. The team decided against the Z shape cross frame and instead added diagonal members to brace the bridge better. After rerunning the simulation, the bridge had no issues in either test. The final design of the Warren truss for the bridge can be seen in the figure below; while the figure does not display the boundary conditions, the model was designed as pin-roller with the roller being on the cantilever side.

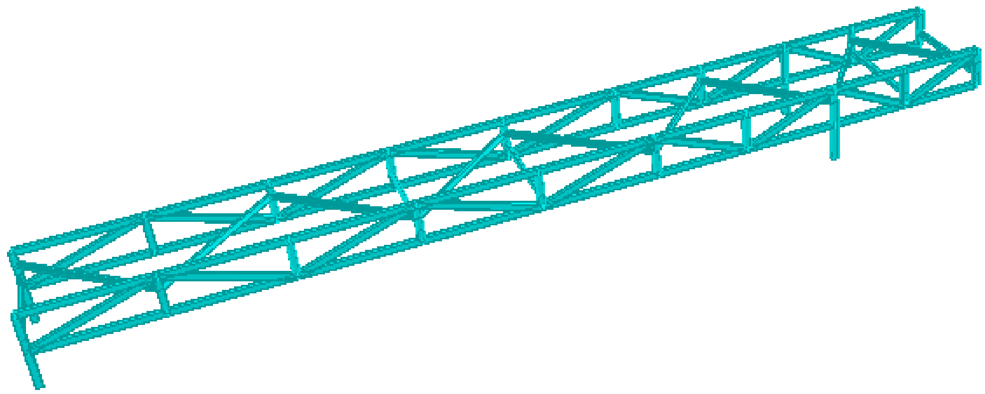

Figure 4-3: STAAD Model Final Design

The deflected shape of the model was found using the STAAD program. In the figure below, the green lines represent the deflected shape. The largest two values from the deflection and sway tests are also located in Figure 4-4.

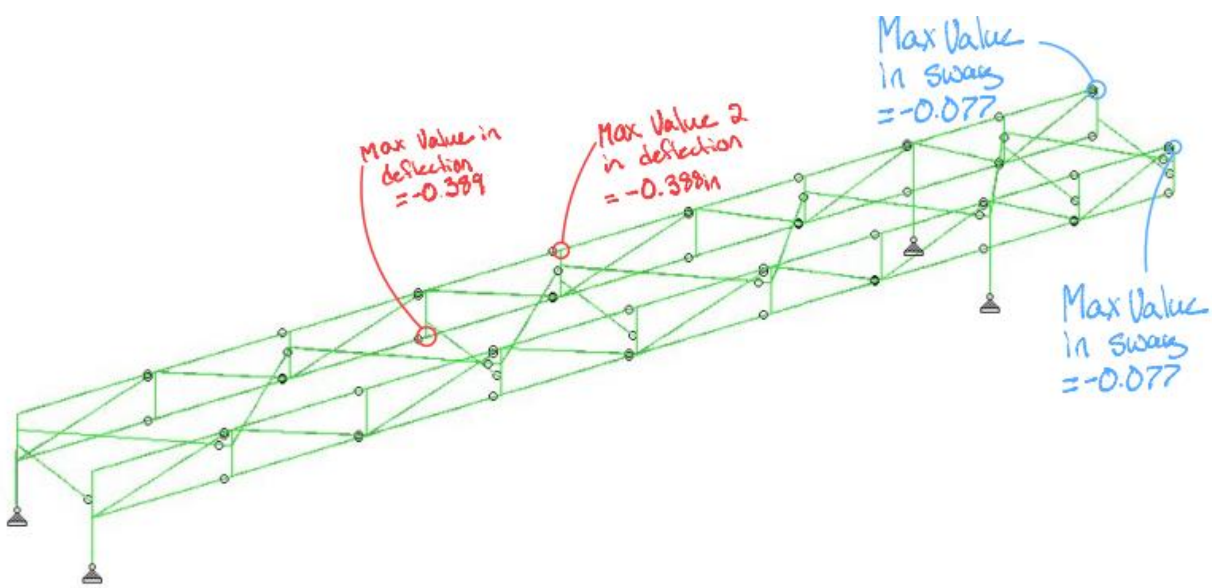

Figure 4-4: STAAD Model-Deflection and Sway

The utilization ratio figure was given within the STAAD software. This figure can be used to see where the stresses are the highest within the model. It uses a “code check” command with each member associated with the model. This check calculated the analysis capacity divided by the code capacity, and those are the numbers that are given to each member in the figure. An example of this is if a number is 0.626, at the bottom right leg. That means that the member is at 62.6% till it hits capacity based off the code it checks with. This can be seen in Figure 4-5.

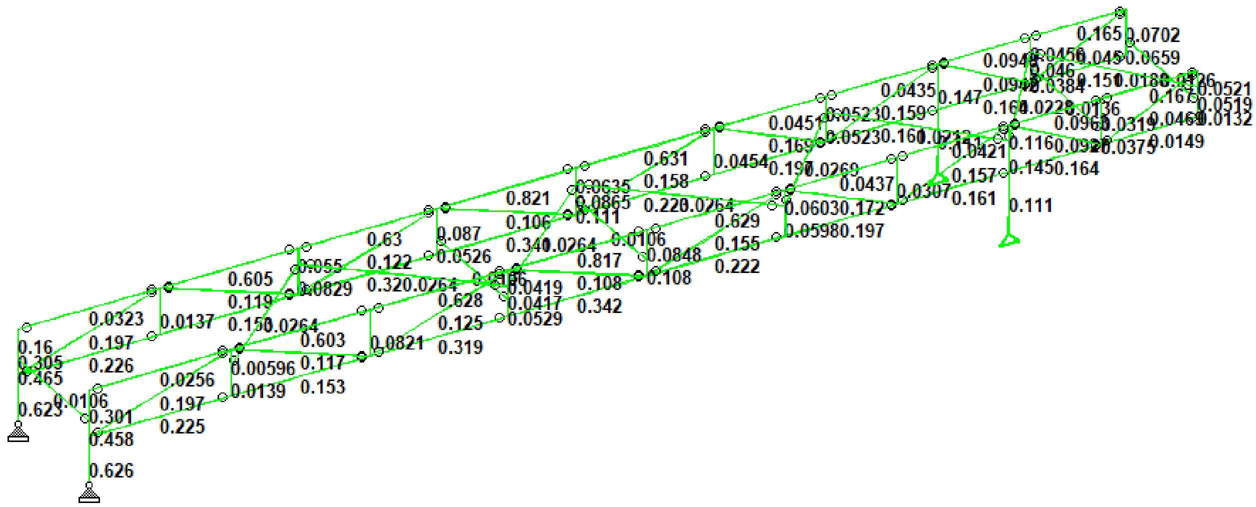

Figure 4-5: Detailed STAAD Deflections

Once the Warren truss model was completed, the team modeled a Pratt truss into the STAAD software to compare the two. This was done to ensure that the prototype chosen was the best option. The Pratt truss was modeled with the same cross section bracing as the Warren truss. The same load cases were applied to each model, and they were both simulated. The stress, sway, and deflection of each model were compared. The Warren truss was determined to have less sway compared to the Pratt model. However, the Pratt truss had slightly better results in the defection values. Neither model exceeded the maximum values for deflection or sway. After the comparison, the team decided to move forward with the Warren truss as it did better in sway and constructability, which were higher concerns. The decision matrix for the final comparison can be seen below.

Table 4-1: Final Decision Matrix

	Weights	**Option 1 – Warren Truss**		**Option 2 – Pratt Truss**	
		Actual Value	Score	Actual Value	Score
Constructability	45%	61	10	65	9
Vertical Deflection	30%	0.15 in	9.5	0.12 in	10
Lateral Deflection	20%	0.078 in	10	0.12 in	9.5
Aesthetics	5%	-	9	-	10
Total Weighted Score	100%		9.80		9.45

4.2 Final Member & Connection Design

Members and connections were decided alongside the creation of the STAAD model. Due to the rules published by AISC, the bridge was required to have stringers at a maximum height of 2 feet and 2 inches and a minimum height of 1 foot and 10 inches. As stated in the initial design, the top cords of the bridge would act as the stringers. This led the team to model the prototype with a deck height of 2 feet. The total width would then have to be within 2 feet and 10 inches due to the stringer dimensional requirements. The finalized width was decided to be 2 feet and 9 ¼ inches to assure that the bridge would meet the competition criteria. The final connection design can be seen in Figure 4-7.

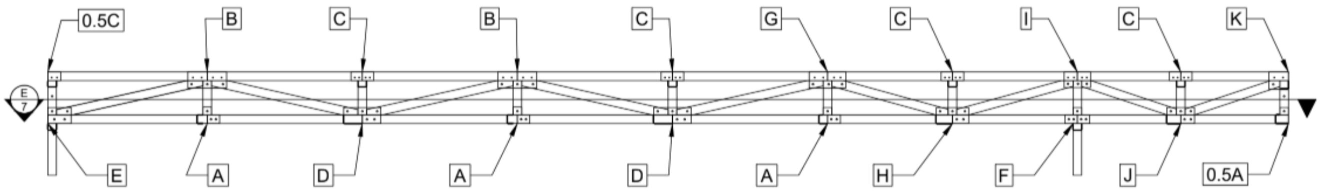

Figure 4-7: Connection Key

Figure 4-8 and 4-9 are detailed drawings of Connection A and Connection B depicted in Figure 4-5. All other connection details can be found in the shop drawings attached in A-4 of the Appendices. Connections 0.5A, C, 0.5C, D, E, F, H, J, and K are all welded to a member in the same fashion as Connection A to decrease construction speed and increase stiffness. Connections I and G are free-floating plates reliant on bolts, similar to Connection B.

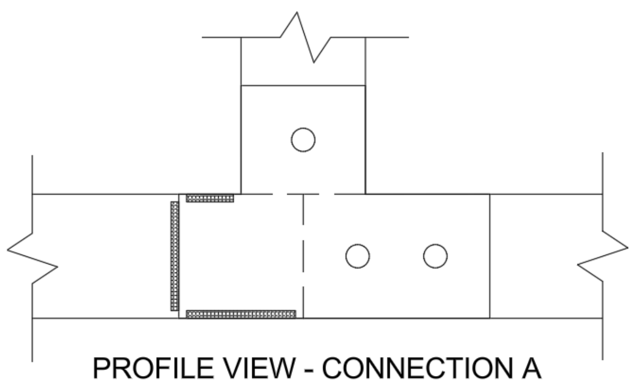

Figure 4-8: Connection A

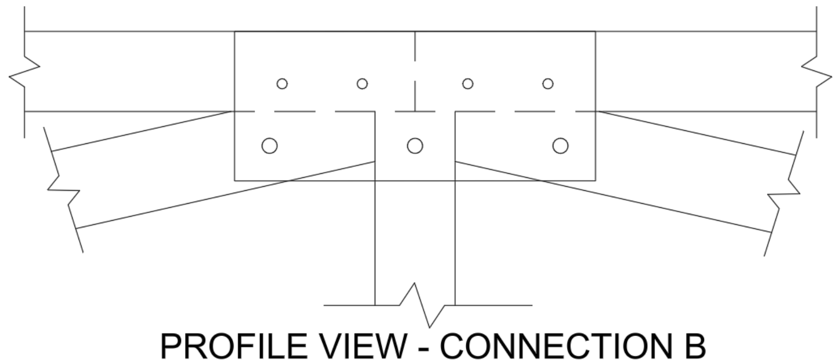

Figure 4-9: Connection B

For connections A, 0.5A, C, 0.5C, D, E, F, H, J, and K, weld failure modes were calculated. The base material weld connection capacity, longitudinal, and transverse weld group capacities were calculated. The weld size maximum, minimum weld length, maximum weld length, and weld terminations were found to ensure that the design capacity for each connection was accurate. The hand calculation example for welded connections can be found in A-2 of the Appendices.

Since all connections included bolts, bolted connection failure states were applied to all plates. The dimension calculations are the following: the bolt hole edge distance minimum, the bolt hole edge distance maximum, the bolt spacing minimum, and the bolt spacing maximum. The failure states calculated include bolt tension failure, bolt shear failure, bolt bearing and tearout per bolt, gross section yielding in tension, gross section yielding in shear, net section rupture in tension, net section rupture in shear, and block shear. The formulas associated with welded and bolted connections can be seen in A-1 of the Appendices while the MathCAD example for bolted connections is located in A-3. The worst case for each connection is summarized in Table 4-2.

Table 4-2: Worst Case Connection Failure States

Connection	Demand (lb)	Capacity (lb)
Connection A	2020	2971
Connection 0.5A	36	2971
Connection B	2950	2971
Connection C	2260	4799
Connection 0.5C	280	4799
Connection D	1600	2971
Connection E	320	2971
Connection F	1180	2971
Connection G	2770	2971
Connection H	1480	2971
Connection I	611	2971
Connection J	582	2971
Connection K	51	2971

Each bolt would penetrate the HSS member and the plate completely, resulting in a through bolt connection. The HSS members have their own failure states for through bolt connections, including end distance and through bolt bearing and tearout. These equations can be seen in A-1 of the Appendices.

Table J2.4 of the ASCE Steel Construction Manual 16th edition states that the minimum weld size is 1/8th of an inch to allow for a proper connection. The width of the weld must be equal or less than the thickness of the steel, so the thickness of the HSS members was decided to be 1/8th of an inch to keep the weight of the bridge to a minimum. Page Steel had members with this thickness at the smallest available size of 2x1x0.125 HSS members. Considering the weight and dimensional constraints, Beyond Bridges moved forward with lateral bracing utilizing 2x1x0.125 HSS members and vertical bracing utilizing 2x1.5x0.125 HSS members.

With the capacities, minimums, and maximum values found from calculations, the dimensions of members and plates were solidified. The total length of the 2x1.5x0.125 HSS members was 159.54 feet while the 2x1x0.125 HSS members totaled 43.15 feet. The plate areas summed to 20.17 square feet. A total of 202 bolts are included in the design, with 130 at a 3/8th inch diameter and 72 at a 1/4th inch diameter. These components result in 607 pounds of self-weight.

5.0 Shop Drawings

The shop drawings were created in AutoCAD, to easily communicate how the bridge should be fabricated. This required the shop drawings to be very thorough, detailing the size of each member and plate, as well as the location and size of all bolts and welds. The shop drawings were 34 pages in total; Table 5-1 shown below, provides a table of contents for the shop drawings. The full shop drawings can be found in Appendix A-3.

Table 5-1: Shop Drawing Summary

Page Number	Page Title
1	Cover Sheet
2	Project Notes
3	Profile & Elevation View
4-5	Section Cuts
6-7	Connection Keys
8	Member Key
9	Top Chord Details
10-11	Bottom Chord Details
12	Diagonal Member Details
13-14	Vertical Member Details
15	Footing Details
16-17	Lateral Bracing Member Details
18-34	Plate Details

6.0 Coordination & Fabrication

6.1 Page Steel

Beyond Bridges used information from last year's team to contact Page Steel and ask if they would be willing to donate material. After the base model of the design was complete, the team had reached out and provided a list of the amount of each HSS member that would be needed. The steel was delivered to Buddy's Welding & RV, where the team picked them up to take to the CECMEE Field Station. The team had then moved forward with gathering information about the plate sizes, and the angle members that would be needed for the bridge. The plates and angles were picked out and a list was sent to Page Steel. Once all members, plates, and angles were received, the team moved forward with fabrication.

6.2 Copper State

At the start of the design work, Copper State was contacted to ensure they would be willing to donate material as they have in previous years. After completing the calculations, Copper State was contacted to place an order for the necessary bolts and nuts for the project. Obtaining these materials allowed the team to begin fabrication.

6.3 Bridge Fabrication

Bridge fabrication consisted of three main parts: cutting and grinding members and plates, welding plates to members, and drilling holes in plates and members. The first part took place at the NAU CECMEE Field Station and consisted of using a table saw to cut the HSS2.0x1.5x0.125 members and the HSS2.0x1.0x0.125 members to the required lengths in accordance with the shop drawing schedules, seen in A-4 of the Appendices. Some members needed to be cut diagonally, which was done with a hand grinder due to the unique cuts. The hand grinder was also used to cut plates to the required sizes, and to grind rust or imperfections off the steel members and the plates in preparation for welding.

The next step to fabricating the bridge was to weld the plates onto the corresponding members. Before the plates were welded, the bolt holes were drilled to reduce the difficulty of applying welded plates to a drill press. The welds included utilizing fillet welds and J-Groove welds in all areas that were specified in the shop drawings.

The final step was to finish drilling the rest of the bolt holes in the plates that did not have any welds, and the members of the bridge. The bolt holes in the plates had to be done first so they could be used to place the holes on the members in the right places. The plates were drilled with a drill press using a 1/4 and a 3/8-inch drill bit. Once the plates were all complete, the members were lined up in the drill press, with the corresponding plate, to drill the holes in the member. This was a long process, and the team was running out of time to finish drilling the holes. Once a portion of the bridge was fabricated, and pieces could start to be put together, a hand drill was also used to drill holes for the members.

7.0 Competition

7.1 Practice Construction

The bridge had multiple delays in fabrication. This caused the team to not have a chance to practice assembling the bridge in the CECMEE Field House. Practicing construction took place in the parking garage of the team's hotel in Salt Lake City, Utah. Once the bridge was fully complete, the team used tape and a marker to label each member, plate, and connection. The completed bridge can be seen in Figure 7-1 below.

Figure 7-1: Completed Practice Build of Bridge

The team ran into issues with the self-weight deflection which made the bolt holes for certain members not line up with the corresponding plates. The self-weight deflection caused the team to pivot and drill the holes slightly bigger to make sure that every bolt would fit into the desired slot. This was a long process which meant that the team did not have the chance to practice construction within the construction parameters described in the competition rules. This construction area is shown in Figure 7-2 below.

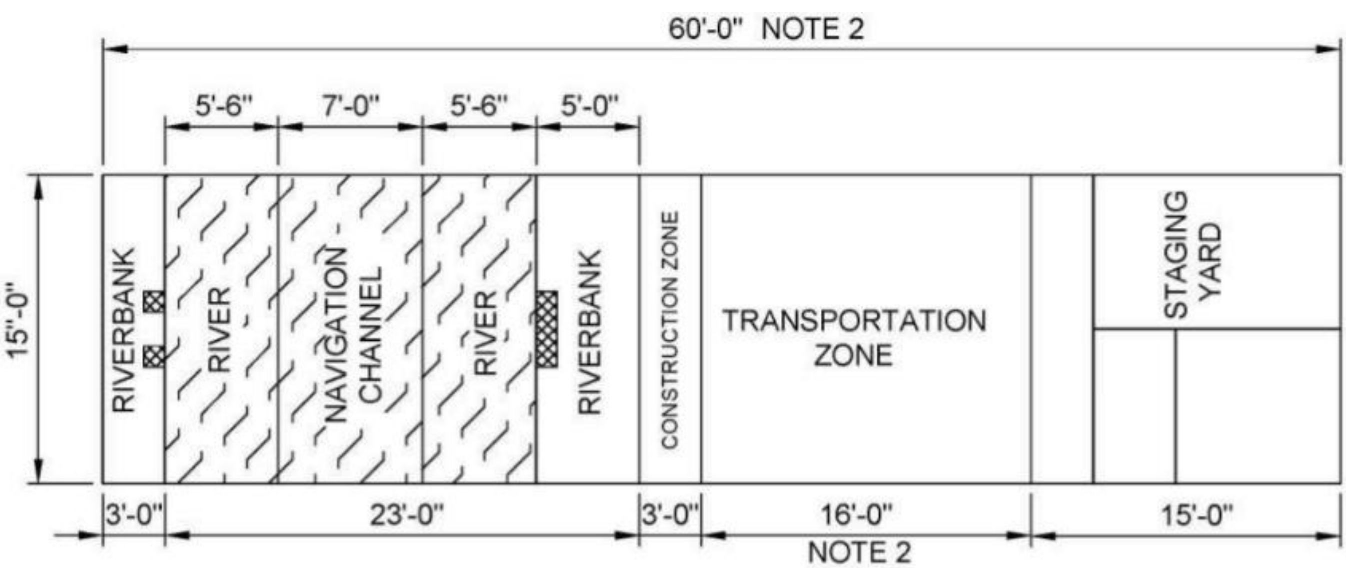

Figure 7-2: Construction Site Plan

7.2 Poster Design

The poster’s required elements were found in the ISWS competition rules. It was to be displayed beside the bridge on Display Day, where it would be included in the bridge’s aesthetics score. The poster was required to have a shear and moment diagram where the bridge is considered a simply supported beam. A scaled and dimensioned side view was also to be included with brief descriptions of the bridge configuration, structural analysis, and connection types. A member considered to be the limit state of the bridge had to be identified on the poster as well, with associated limit state calculations. Beyond Bridge’s poster can be seen in A-5 of the Appendices.

7.3 Competition Day

The first part of the competition was the display day. This is the day that each team competing displays their bridge with their posters for the judges. The judges would evaluate each bridge and poster to score the aesthetics. When the judging time commenced, the team needed to stand around their bridge and answer any questions that might come up with the bridge. Unfortunately, the team ran into issues involving the bolt holes and the bridge's self-weight, which made it extremely difficult to put together. The team spent three hours trying to get the bridge built, but the self-weight deflection caused the bolt holes to not line up properly. The team had run out of time and were disqualified from the aesthetics portion of the competition.

The competition timed construction took place the following day. The team had labeled all members and decided on a strategy to try and get the bridge built within the 45-minute time limit. The first task of the day was to get everything placed within the staging area. This area had a box where the nuts, bolts, members, and tools would be placed before starting the competition. There were multiple rules such as: no members were allowed to touch each other, and all the members and tools must fit within the 3 ½' x 6”x 4” box.

After placing everything down in their respective boxes within the staging area, the competition would begin. The team had four team members inside of the river areas; these members were labeled as barges. The barges would be constructing most of the bridge as they had the most access. There were two team members outside the river that were labeled as regular builders. The builders ran all the members, bolts, and tools to the barges. The builders were also in charge of building the cantilever due to their closer proximity. The competition timer started when everyone was in their positions, and the team started building. The team had completed the bottom cords for both sides and some of the bracing in the middle before running out of time and thus getting disqualified. The judges gave the team the option to keep building it off to the side, and they would still allow for a loading test to occur if they deemed it safe enough. After pulling the bridge to the side, the team tried their best to finish construction so the bridge could still get tested. Ultimately, the team came to the decision that the members would not properly fit together without restarting construction, which was not an option.

7.4 Final Results

The team decided to test the bridge to obtain deflection results once the competition was completed. The worst load case according to the STAAD model and hand calculations was Case 9 from the competition rules, located 9 feet and 6 inches from the West end of the bridge. A total of 1600 pounds was placed onto its respective decking, while 900 pounds was placed onto decking an inch from the cantilever end of the bridge. The results can be seen below in Table 7-1.

Table 7-1: Final Results

Midspan Vertical Deflection	1 3/8"
Cantilever Vertical Deflection	1/16"

8.0 Project Impacts

For the impact analysis, the team considered the impacts of having the ASCE/AISC Student Steel Bridge Competition with the construction requirements versus only having the competition design a bridge without constructing. The team used the triple bottom line method, looking into the social, environmental, and economic impacts for each alternative, a summary of these impacts can be seen in the table below.

Table 8-1: Impact Analysis

		People (Social)	Planet (Environmental)	Price (Economic)
Steel Bridge Competition with Bridge Construction	**Positive Impacts**	Students gain practical experience designing bridges. Networking between schools and professionals across multiple states. (Cultural Impact)	Project teaches students about the environmental impacts of building a bridge. (Cultural Impact)	Multiple prototypes are built and judged to determine the best pedestrian bridge for a given location. (Global Impact)
	Negative Impacts	Professional engineers donate time to provide advice to teams and judge competition (Cultural Impact)	Air pollution caused by manufacturing steel used and welding members. (Global Impact)	Supply companies lose money as they donate materials to teams every year. (Cultural Impact)
Steel Bridge Competition without Construction	**Positive Impacts**	Professional engineers would not need to take time away from work to attend or judge the competition. (Cultural Impact)	No air pollution caused by large groups traveling to the competition location. (Global Impact)	Schools save money by not paying for competition or travel. Supply & Engineering companies save money by not donating to the competition. (Cultural Impact)
	Negative Impacts	Students don't get the same design and construction experience that this competition provides. (Cultural Impact)	Long term sustainability could be weaker because less resources are devoted to improving the design. (Global Impact)	Pedestrian bridge projects would not have the comparison of multiple prototype test results to decide the best designs for cost and construction. (Cultural Impact)

The team then discussed the impacts of each alternative and assigned scores for each category to determine the sustainability index for each option. These scores can be seen in the table below.

Table 8-2: Triple Bottom Line

	People (Social)	Planet (Environmental)	Price (Economic)	Sum	Max-Min	SI
Competition with Construction	7	6	4	17	3	14
Competition without Construction	3	5	7	15	4	11

The results of the triple bottom line analysis show that having a Steel Bridge Competition with the construction aspect of the project provides more benefit in social and environmental categories based on the hands-on experience, educational value, and attention to awareness of sustainable engineering design. However, the economic aspect of having the bridge construction was lower based on the costs associated with materials used and donations from sponsors. These impacts were overall outweighed by the social and environmental benefits. On the other hand, the No Competition alternative had some benefits, such as eliminating extra costs for schools and supporting companies. However, it scored lower in a social category based on real-world experiences and networking between universities, students, and industry professionals. Based on the Sustainability Index, which is calculated by subtracting the Max-Min value from the total score to account for balance and the performance, having the Steel Bridge competition with bridge construction results in a more sustainable option than the no construction alternative. Both options have tradeoffs, but having the competition provides a slightly more beneficial outcome when considering all the impacts.

9.0 Summary of Engineering Work

The engineering work required for this project involved designing, analyzing, and optimizing the members and connections of a steel bridge to meet the requirements set by ASCE for the ISWS competition. Extensive background research was required to understand dimension constraints, connection requirements, and rules of construction. Along with this, the grade of steel used for members and fasteners needed to be researched to ensure they had the capacity and strength based on the loading requirements. The team selected an underslung truss configuration testing basic truss formations such as Warren, Pratt, and Howe. Rectangular HSS members made from A500 steel were selected based on their high strength to weight ratio and ability to be bolted with plate connections.

The initial bridge configurations were developed in the software RISA-2D to give an estimation of the strengths and weaknesses of the three truss types. For these models, performance criteria included vertical deflection and constructability (i.e. number of members and nodes). A decision matrix was created to assess the constructability, deflection results, and aesthetics weighing the least amount. The Warren truss scored the highest, which meant that was the initial design the team moved forward with.

The final design was analyzed in STAAD which is a more accurate and powerful 3D modeling software. Realistic boundary connections were modeled along with various vertical and horizontal loading scenarios based on where the mid-span 1600 pounds, cantilever 900 pounds, and 50-pound lateral force would be applied. The output results from the model were verified against the deflection and sway limits in which additional lateral bracing was incorporated in improving stability. Along with this, the internal force results of the members and connection nodes were evaluated to ensure all components of the bridge met the strength requirements per competition loading.

Lastly, the connection design included both bolted and welded connections. Detailed calculations were performed by and excel to obtain the weld capacity, bearing, tearout, and bolt shear. These were all in accordance with AISC specifications. Some member sizes were optimized to balance strength and constructability. The complete final design satisfied all the competition constraints, giving the client a constructable solution.

Table 9-1, located below, identifies the initial estimation of hours required to complete the project; summarizing the distribution of hours across major tasks and team roles, including the Senior Engineer, Engineer, Engineer-in-training, and the Drafter. Each task and role were assigned estimated hours based on how complex they were. Table 9-2 lists the actual hours the project required, based on team members tracking the time spent on every task for the project.

Table 9-1: Proposed Hours

Task #	Task Name	MSENG	ENG	EIT	DRFT
1.0	Background Research	-	6	15	-
2.0	Initial Design	6	33	42	10
3.0	Final Analysis & Design	15	40	40	-
4.0	Shop Drawings	5	5	10	85
5.0	Coordination & Fabrication	16	35	7	-
6.0	Competition	29	31	37	29
7.0	Project Impacts	-	3	5	-
8.0	Capstone Deliverables	4	12	48	-
9.0	Project Management	31	19	16	16
	Position Totals	106	183	217	140
				Project Total	650

Table 9-2: Actual Hours

Task #	Task Name	MSENG	ENG	EIT	DRFT
1.0	Background Research	-	9	19	-
2.0	Initial Design	7	31	45	9
3.0	Final Analysis & Design	13	43	94	0
4.0	Shop Drawings	5	1	3	127
5.0	Coordination & Fabrication	22	104	30	-
6.0	Competition	3	25	26	-
7.0	Project Impacts	1	1	4	-
8.0	Capstone Deliverables	7	11	36	-
9.0	Project Management	12	10	12	4
	Position Totals	70	235	269	140
				Project Total	714

Differences between the estimated and actual hours are mainly due to unforeseen adjustments required during the modeling, analysis, shop drawings, and fabrication processes. A couple of tasks, particularly Final Analysis & Design, and Shop Drawings, required extensive time beyond the initial estimates due to design revisions. These revisions came from member size adjustments and a re-design of the lateral bracing. Since the shop drawings reflect the design itself, those had to be updated to have a cohesive representation of the steel bridge.

Likewise, some tasks required fewer hours than anticipated as seen in Project Impacts, Capstone Deliverables, and Project Management. The reduction in time was mainly due to the improved efficiency of a team as the project progressed. The overall workflow improvement and familiarity with the design process and modeling tools were reflected in the later portion of the project. While there were differences between the estimated time and actual hours, the distribution of hours did not change significantly since the major task areas of the project that were estimated to take longer were correctly predicted.

10.0 Summary of Engineering Costs

The proposed cost for this project was $88,000. The breakdown for this cost can be seen in the table below.

Table 10-1: Proposed Cost

	Description	Quantity	Units	Unit Cost	Cost
Staffing					
	Managing Senior Engineer	106	hrs	$208	$22,048
	Project Engineer	184	hrs	$113	$20,792
	Engineer in Training	220	hrs	$68	$14,960
	Drafter	140	hrs	$54	$7,560
				Subtotal	$65,360
Supplies					
	Computer Lab	38	day	$100	$3,800
	PPE	1	LS	$560	$560
	Tools	1	LS	$120	$120
				Subtotal	$4,480
Materials					
	Steel	1	LS	$6,000	$6,000
	Bolts	1	LS	$100	$100
	Nuts	1	LS	$35	$35
				Subtotal	$6,135
Subcontracting					
	Welding	88	hrs	$100	$8,800
				Subtotal	$8,800
Travel					
	Conference Registration	1	LS	$310	$310
	Rental Vehicle	5	days	$40	$200
	Fuel	526	miles	$0	$79
	Hotel	4	nights (2 rooms)	$142	$1,136
	M&IE	5	days (4 people)	$75	$1,500
				Subtotal	$3,225
				Total	$88,000

The actual cost for the project was approximately $79,500; Table 10-2 shows the details for each cost category.

Table 10-2: Actual Cost

	Description	Quantity	Units	Unit Cost	Cost
Staffing					
	Managing Senior Engineer	70	hrs	$208	$14,560
	Project Engineer	235	hrs	$113	$26,555
	Engineer in Training	269	hrs	$68	$18,292
	Drafter	136	hrs	$54	$7,344
				Subtotal	$66,751
Supplies					
	Computer Lab	65	day	$100	$6,500
	PPE	1	LS	$560	$560
	Tools	1	LS	$700	$700
				Subtotal	$7,760
Materials					
	Steel	607	LBS	$2.70	$1,639
	Bolts	1	LS	$200	$200
	Nuts	1	LS	$40	$40
				Subtotal	$1,779
Travel					
	Conference Registration	1	LS	$310	$310
	Rental Vehicle	5	days	$40	$200
	Fuel	526	miles	$0.15	$79
	Hotel	4	nights (2 rooms)	$142	$1,136
	M&IE	5	days (4 people)	$75	$1,500
				Subtotal	$3,225
				Total	$79,615

There is approximately an $8,500 difference between the proposed cost and the actual work costs. The main difference between the proposed cost and the actual cost was caused by the team's decision to fabricate the entire bridge rather than subcontracting the welding and drilling work to Flagstaff High School. This resulted in removing all subcontracting costs, adding more hours to the staffing, and having more costs for tools. Another notable change was the cost of the steel material. After determining the weight of the bridge during final analysis, the cost of steel was reevaluated based on the price of steel per pound. Overall, the final project cost was relatively similar to the proposed cost.

11.0 Conclusion

The objective of the project was to design, fabricate, and construct a structurally efficient steel bridge that met the requirements given by ASCE and AISC in the Student Steel Bridge Competition. The design process consisted of evaluating different truss configurations, performing structural analysis based on loadings, and finally optimizing strength, stability, and constructability.

Background research combined with preliminary modeling helped narrow down the design with a Warren truss ultimately being selected as the final configuration based on its structural performance. Various structural analyses were performed including both hand calculations as well as computational modeling using RISA-2D and STAAD. Both methods of analysis verified internal member forces and predicted deflection values. The calculations by hand were similar and consistent with the modeling software used. Refinements were made throughout the process, with a major refinement being a redesign of the lateral bracing to improve stability and ensure compliance with the sway requirements of the competition.

The final steel bridge design utilized rectangular HSS members with combinations of bolts and welded connections. These were designed in accordance with AISC specifications. Overall, the bridge design met all required structural performance criteria, and the project achieved the main objective in delivering an efficient, safe, and constructible steel bridge. In addition, a triple bottom line analysis was completed showing that having the Steel Bridge Competition provides greater overall benefit as compared to not having the competition.

12.0 References

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Appendices

A-1: Formulas Used in Calculations

Equation A-1-1: Weld Size Maximum [6]

$$t \leq \frac{1}{4}" \; ; \; w \leq t$$

Variables:

t = Material Thickness
w = Weld Thickness

Equation A-1-2: Minimum Length of Welds [6]

$$L \geq 4w$$

Variables:

L = Length of Welded Segments
w = Width of the Material

Equation A-1-3: Maximum Length of Welds [6]

$$L \leq 100w$$

Variables:

L = Length of Welded Segments
w = Width of the Material

Equation A-1-4: Weld Termination [6]

$$Weld\ Termination \geq w$$

Variables:

w = Width of the Material

Equation A-1-5: Connection Capacity [6]

$$\phi R_n = min \begin{cases} 1.392 * (KD) \\ 0.6 F_y t \\ 0.45 F_u t \end{cases}$$

Variables:

ϕ = Safety Factor
R_n = Connection Capacity
K = Directional Strength Factor
D = Weld Size in 1/16ths
F_y = Yield Strength of Material
F_u = Tensile Stress of Material
t = Thickness of the Material

Equation A-1-6: Longitudinal and Transverse Weld Groups [6]

$$\phi R_n = (0.75) max \begin{cases} R_{nwL} + R_{nwT} \\ 0.85R_{nwL} + 1.5R_{nwT} \end{cases}$$

Variables:

ϕ = Safety Factor
R_n = Connection Capacity
R_{nwL} = Longitudinal Strength Increase
R_{nwT} = Transverse Strength Increase

Equation A-1-7: Edge Distance for Standard Holes [6]

$$Min\ Edge\ Distance = d_b + \frac{1}{4}$$

Variables:

d_b = Bolt Diameter

Equation A-1-8: Maximum Edge Distance [6]

$$Max\ Edge\ Distance = \min\left(12t_{min}, 6"\right)$$

Variables:

t_w = Thickness of the Smaller Material

Equation A-1-9: Bolt Spacing Minimum [6]

$$Min\ Spacing = 2\frac{2}{3}d_b\ or\ 3d_b$$

Variables:

d_b = Bolt Diameter

Equation A-1-10: Maximum Bolt Spacing [6]

$$Max\ Spacing = \min\left(24t_{min}, 12"\right)$$

Variables:

t_{min} = Thickness of the Smallest Material

Equation A-1-11: Bolt Tension Failure [6]

$$\phi R_n = \phi F_{nt} A_b n_b$$

Variables:

ϕ = Safety Factor
R_n = Bolt Tension Capacity
F_{nt} = Nominal Stress of Bolts
A_b = Area of the Bolt
n_b = Number of Bolts in Connection

Equation A-1-12: Bolt Shear Failure [6]

$$\phi R_n = \phi F_{nv} A_b n_b n_s$$

Variables:

ϕ = Safety Factor
R_n = Bolt Shear Capacity
F_{nv} = Nominal Stress of Bolts
A_b = Area of the Bolt
n_b = Number of Bolts in Connection
n_s = Number of Shear Planes

Equation A-1-13: Bolt Bearing and Tearout Per Bolt [6]

$$\phi R_n = 0.75 min \begin{cases} 2.4 d t F_u \\ 1.2 L_c t F_u \end{cases}$$

Variables:

ϕ = Safety Factor
R_n = Bolt Bearing or Tearout Capacity
d = Diameter of Bolt
t = Thickness of the Material
F_u = Tensile Stress of Material

Equation A-1-14: Base Material Failure Gross-Section Yeilding in Tension [6]

$$\phi R_n = 0.9 F_y A_{gv}$$

Variables:

ϕ = Safety Factor
R_n = Tension Member Capacity
F_y = Yield Strength of Material
A_g = Gross Area in Tension

Equation A-1-15: Base Material Failure-Net Section Rupture in Tension [6]

$$\phi R_n = 0.75 F_u U A_e$$

Variables:

ϕ = Safety Factor
R_n = Tension Member Capacity
F_u = Tensile Stress of Material
U = Shear Lag Factor
A_e = Effective Area

Equation A-1-16: Base Material Failure-Gross Section Yielding in Shear [6]

$$\phi R_n = (1.0)(0.6) F_y A_{gv}$$

Variables:

ϕ = Safety Factor

R_n = Shear Member Capacity
F_y = Yield Strength of Material
A_{gv} = Gross Area in Shear

Equation A-1-17: Base Material Failure-Net Section Rupture in Shear [6]

$$\phi R_n = (0.75)(0.6)F_u A_{nv}$$

Variables:
ϕ = Safety Factor
R_n = Shear Member Capacity
F_u = Yield Strength of Material
A_{nv} = Gross Area in Shear

Equation A-1-18: Base Material Failure-Block Shear [6]

$$\phi R_n = \phi min \begin{cases} 0.6F_u A_{nv} + U_{BS}F_u A_{nt} \\ 0.6F_y A_{gv} + U_{BS}F_u A_{nt} \end{cases}$$

Variables:
ϕ = Safety Factor
R_n = Block Shear Capacity
F_u = Yield Strength of Material
A_{nv} = Gross Area in Shear
F_y = Yield Strength of Material
A_{gv} = Gross Area in Shear
U_{BS} = Shear Lag Factor
A_{nt} = Net Area in Tension

Equation A-1-19: Effective Length of Weld for HSS Members [6]

$$l_e = \frac{2H_b}{sin\theta} + 2B_e$$

Variables:
l_e = Effective Length for Welds Along an HSS Member
H_b = Overall Height of Rectangular HSS Branch Member in Plane of Direction
θ = Acute Angle Between the Branch and Chord
B_e = Effective Width of Rectangular HSS Branch Member or Plate

Equation A-1-20: Effective Elastic Section Modulus of Welds for In-Plane Bending [6]

$$S_{ip} = \frac{t_w}{3}\left(\frac{H_b}{sin\theta}\right)^2 + H_b t_w \left(\frac{H_b}{sin\theta}\right)$$

Variables:
t_w = Smallest Effective Weld Throat Thickness Around the Perimeter of Branch or Plate
H_b = Overall Height of Rectangular HSS Branch Member in Plane of Direction
θ = Acute Angle Between the Branch and Chord

Equation A-1-21: Effective Elastic Section Modulus of Welds for Out-of-Plane Bending [6]

$$S_{op} = t_w\left(\frac{H_b}{sin\theta}\right)B_b + \frac{t_w}{3}(B_b^2) - \frac{\left({t_w}/{3}\right)(B_b - B_e)^3}{B_b}$$

Variables:

l_e = Effective Length for Welds Along an HSS Member
H_b = Overall Height of Rectangular HSS Branch Member in Plane of Direction
θ = Acute Angle Between the Branch and Chord
B_e = Effective Width of Rectangular HSS Branch Member or Plate
B_b = Overall Width of Rectangular HSS Branch Member or Plate for Local Yielding of Transverse element

Equation A-1-22: HSS Member End Distance [6]

$$l_{end} \geq B\sqrt{1-\beta}, for\ \beta \leq 0.85$$

Variables:

l_{end} = End Distance
B = Overall Width of Rectangular HSS Cord Member or Plate 90°
β = Width Ratio

Equation A-1-23: HSS Through Bolt Bearing [6]

$$\phi R_n = 1.8 F_y A_{pb} n$$

Variables:

ϕ = Safety Factor
R_n = Through Bolt Bearing Capacity
F_y = Yield Strength of Material
A_{pb} = Gross Area in Shear
n = Shear Planes

Equation A-1-24: HSS Through Bolt Tearout [6]

$$\phi R_n = 1.2 l_c t F_u$$

Variables:

ϕ = Safety Factor
R_n = Through Bolt Tearout Capacity
F_u = Tensile Strength of Material
t = HSS Thickness
l_c = Clear Distance

A-2: Sample Weld Calculations

See hand calculations on the following page.

WELDING CALCS

Try 1/4th Plate

Max Weld Size

Matl $\leq 1/4'' \rightarrow 1/8 \leq 1/4$ $\quad w_{min} \leq t$

Min Weld Size

Tab J2.4 $\rightarrow 1/8''$ $\quad$ **weld size $w = 1/8''$**

Length min

$L \geq 4w \rightarrow$ **$L \geq 1/2''$**

Length Max

$L \leq 100w \rightarrow L \leq 100(1/8'') \rightarrow$ **$L \leq 12.5$ inches**

Weld Terminations

Term $\geq w \rightarrow$ **Term $\geq 1/8''$**

Conn. Capacity

$$\phi R_n = \min \begin{vmatrix} 1.392(kD) \\ 0.6F_y t \\ 0.45 F_u t \end{vmatrix} = \begin{matrix} 1.392(1.5)(2) = 4.176 \text{ k/in} \\ 0.6(50\text{ksi})(1/8'') = 3.75 \text{ k/in} \\ 0.45(62)(1/8'') = \boxed{3.49 \text{ k/in}} \end{matrix}$$

ALL AROUND WELDS

$$\phi R_n = \max \begin{vmatrix} R_{nwL} + R_{nwT} \\ 0.85R_{nwL} + 1.5R_{nwT} \end{vmatrix} \begin{matrix} = 3.49 \text{ k/in}(2\times2 + 2\times1) = 20.94\text{k} \\ = 0.85(3.49 \text{ k/in})(4'') + 1.5(3.49\text{k})(2'') = \boxed{22.37\text{k}} \end{matrix}$$

Termination welds option

$$\phi R_n = \max \begin{vmatrix} R_{nwL} + R_{nwT} \\ 0.85R_{nwL} + 1.5R_{nwT} \end{vmatrix} \begin{matrix} = 3.49 \text{ k/in}(3'') + 3.49 \text{ k/in}(1'') = 13.96\text{k} \\ = 0.85(3'')(3.49 \text{ k/in}) + 1.5(3.49 \text{ k/in})(1'') = \boxed{14.13\text{k}} \end{matrix}$$

A-3: MathCAD Bolt Calculations

See MathCAD sample calculations on the following pages.

Beyond Bridges Bolt Calculations

Defined Variables

Shear lag factor	$U := 1$	$S_s := 1.125\ in$
Bolt diameter	$D_b := 0.375\ in$	$\phi := 0.75$
Hole diameter	$D_h := 0.4375\ in$	$D_e := 0.8\ in$
Plate thickness	$t := 0.250\ in$	
Bolt ultimate strength	$F_{ub} := 54\ ksi$	
Plate ultimate strength	$F_{up} := 65\ ksi$	
Plate yield strength	$F_{yp} := 50\ ksi$	
Bolt Area	$A_b := \frac{\pi}{4} \cdot D_b^{\ 2} = 0.11\ in^2$	

Bolt Tension failure

$F_{nt} := 0.75\ F_{ub} = 40.5\ ksi$ $\quad n_b := 2$

$\phi R_n := \phi \cdot F_{nt} \cdot A_b \cdot n_b = 6.71\ kip$ $\quad n_b := 4$

$\phi R_n := \phi \cdot F_{nt} \cdot A_b \cdot n_b = 13.419\ kip$ $\quad n_b := 1$

$\phi R_n := \phi \cdot F_{nt} \cdot A_b \cdot n_b = 3.355\ kip$

Bolt Shear failure

$F_{nv} := 0.625 \cdot F_{ub} \cdot 0.9 \cdot 1 = 30.375\ ksi$ $\quad n_s := 2$

$n_b := 2$

$\phi R_n := \phi \cdot F_{nv} \cdot A_b \cdot n_b \cdot n_s = 10.064\ kip$ $\quad n_b := 4$

$\phi R_n := \phi \cdot F_{nv} \cdot A_b \cdot n_b \cdot n_s = 20.129\ kip$ $\quad n_b := 1$

$\phi R_n := \phi \cdot F_{nv} \cdot A_b \cdot n_b \cdot n_s = 5.032\ kip$

$F_{nv} := 0.625 \cdot F_{ub} \cdot 0.75 \cdot 1 = 25.313\ ksi$

$n_b := 2$

$\phi R_n := \phi \cdot F_{nv} \cdot A_b \cdot n_b \cdot n_s = 8.387\ kip$ $\quad n_b := 4$

$\phi R_n := \phi \cdot F_{nv} \cdot A_b \cdot n_b \cdot n_s = 16.774\ kip$ $\quad n_b := 1$

$\phi R_n := \phi \cdot F_{nv} \cdot A_b \cdot n_b \cdot n_s = 4.194\ kip$

Bearing and Tearout

Bearing $\quad B_b := .75 \cdot 2.4\ D_b \cdot t \cdot F_{up} = 10.969\ kip$ $\quad L_c := 0.8\ in$

Tearout $\quad B_t := .75 \cdot 1.2 \cdot L_c \cdot t \cdot F_{up} = 11.7\ kip$

$\phi R_{n1} := min\left(B_b, B_t\right) = 10.969\ kip$

Bolts not at edge

Bearing $\quad B_b := .75 \cdot 2.4\ D_b \cdot t \cdot F_{up} = 10.969\ kip$ $\quad L_c := 0.688\ in$

Tearout $\quad B_t := .75 \cdot 1.2 \cdot L_c \cdot t \cdot F_{up} = 10.062\ kip$

$\phi R_{n2} := min\left(B_b, B_t\right) = 10.062\ kip$

$\phi R_n := \phi R_{n1} + \phi R_{n2} = 21.031\ kip$

Tension Strength

Gross area
Effective net area
Net Area
Yield strength of tension member
Ultimate strength of tension member

$A_g := t \cdot 2\ in = 0.5\ in^2$

$A_e := A_g - D_h \cdot 1 \cdot \left(\frac{1}{4}\ in\right) = 0.391\ in^2$

$A_n := A_e$

$GYS_t := 0.9 \cdot F_{yp} \cdot A_g = 22.5\ kip$

$NSR_t := 0.75 \cdot F_{up} \cdot U \cdot A_n = 19.043\ kip$

Effective net area (2 bolts)
Net Area (2 bolts)

$A_e := A_g - D_h \cdot 2 \left(\frac{1}{4}\ in\right) = 0.281\ in^2$

$A_n := A_e$

$GYS_t := 0.9 \cdot F_{yp} \cdot A_g = 22.5\ kip$

$NSR_t := 0.75 \cdot F_{up} \cdot U \cdot A_n = 13.711\ kip$

Shear Strength

Gross area in shear
Net area in shear (after bolt holes)
Net Area in tension
Yield strength of steel in shear
Ultimate strength of steel in shear

$A_{gv} := t \cdot \left(S_s + \left(D_e + D_h \cdot 0.5\right)\right) = 0.536\ in^2$

$A_{nv} := A_{gv} - \left(D_h + D_h \cdot 0.5\right) \cdot t = 0.372\ in^2$

$A_{nt} := t \cdot D_e = 0.2\ in^2$

$GYS_v := 1.0 \cdot 0.6 \cdot F_{yp} \cdot A_{gv} = 16.078\ kip$

$NSR_v := 0.75 \cdot 0.6 \cdot F_{up} \cdot A_{nv} = 10.877\ kip$

Block Shear

$B_1 := 0.75 \cdot \left(0.6 \cdot F_{up} \cdot A_{nv} + F_{up} \cdot A_{nt}\right) = 20.627\ kip$

$B_2 := 0.75 \cdot \left(0.6 \cdot F_{yp} \cdot A_{gv} + F_{up} \cdot A_{nt}\right) = 21.809\ kip$

$\phi R_n := min\left(B_1, B_2\right) = 20.627\ kip$

Gross area in shear
Net area in shear (after bolt holes)
Net Area in tension
Yield strength of steel in shear
Ultimate strength of steel in shear

$A_{gv} := t \cdot \left(D_e + D_h \cdot 0.5\right) = 0.255\ in^2$

$A_{nv} := A_{gv} - \left(D_h \cdot 0.5\right) \cdot t = 0.2\ in^2$

$A_{nt} := t \cdot D_e = 0.2\ in^2$

$GYS_v := 1.0 \cdot 0.6 \cdot F_{yp} \cdot A_{gv} = 7.641\ kip$

$NSR_v := 0.75 \cdot 0.6 \cdot F_{up} \cdot A_{nv} = 5.85\ kip$

Block Shear

$B_1 := 0.75 \cdot \left(0.6 \cdot F_{up} \cdot A_{nv} + F_{up} \cdot A_{nt}\right) = 15.6\ kip$

$B_2 := 0.75 \cdot \left(0.6 \cdot F_{yp} \cdot A_{gv} + F_{up} \cdot A_{nt}\right) = 15.48\ kip$

$\phi R_n := min\left(B_1, B_2\right) = 15.48\ kip$

Gross area in shear
Net area in shear (after bolt holes)
Net Area in tension
Yield strength of steel in shear
Ultimate strength of steel in shear

$A_{gv} := t \cdot (D_e + D_h \cdot 0.5) = 0.255\ in^2$

$A_{nv} := A_{gv} - (D_h \cdot 0.5) \cdot t = 0.2\ in^2$

$A_{nt} := t \cdot (S_s + (D_e + D_h \cdot 0.5)) = 0.536\ in^2$

$GYS_v := 1.0 \cdot 0.6 \cdot F_{yp} \cdot A_{gv} = 7.641\ kip$

$NSR_v := 0.75 \cdot 0.6 \cdot F_{up} \cdot A_{nv} = 5.85\ kip$

Block Shear

$B_1 := 0.75 \cdot (0.6 \cdot F_{up} \cdot A_{nv} + F_{up} \cdot A_{nt}) = 31.977\ kip$

$B_2 := 0.75 \cdot (0.6 \cdot F_{yp} \cdot A_{gv} + F_{up} \cdot A_{nt}) = 31.857\ kip$

$\phi R_n := min(B_1, B_2) = 31.857\ kip$

A-4: Shop Drawings

See Shop Drawings on the following pages.

2026 NAU ASCE STEEL BRIDGE

SENIOR DESIGN PROJECT
BEYOND BRIDGES

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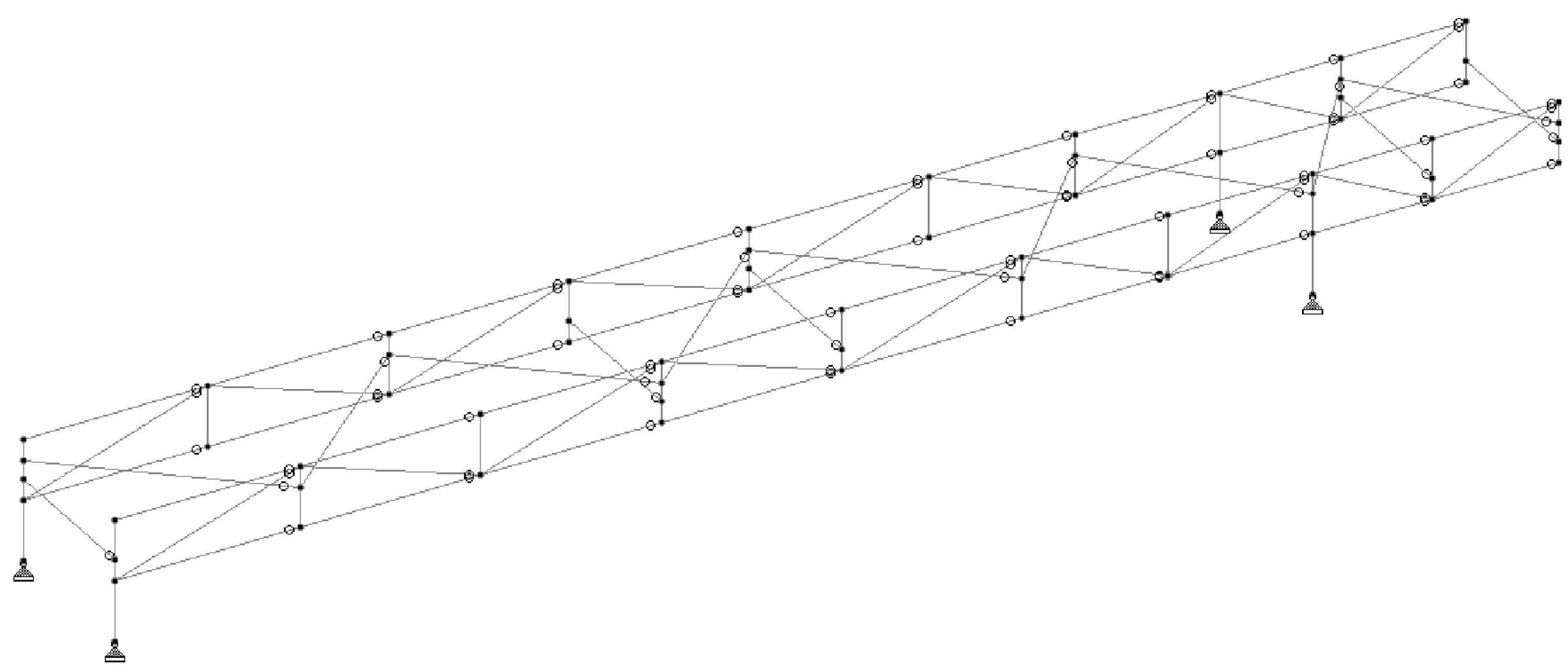

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HIDDEN LINES	– — –

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COVER PAGE	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 1 OF: 34
		DATE: MARCH 2026	

CONNECTION SCHEDULE

CONNECTION	THICKNESS	SIZE	QUANTITY
A	1/4" PLATE	5" x 3 3/4"	6
0.5A	1/4" PLATE	3" x 3 3/4"	2
B	1/4" PLATE	9" x 3 3/4"	4
C	1/4" PLATE	5 1/4 x 3 3/4"	8
0.5C	1/4" PLATE	3" x 3 3/4"	2
D	1/4" PLATE	8 5/8" x 3 3/4"	4
E	1/4" PLATE	5 3/8" x 5 1/2"	2
F	1/4" PLATE	5 3/4" x 5 1/2"	2
G	1/4" PLATE	8" x 3 3/4"	2
H	1/4" PLATE	7 1/2" x 3 3/4"	2
I	1/4" PLATE	6 3/4" x 3 7/8"	2
J	1/4" PLATE	6 3/8" x 3 3/4"	2
K	1/4" PLATE	4 5/16" x 4 1/2"	2
L	1/4" PLATE	2" x 2 5/8"	10
M	1/4" ANGLE	2" x 1 1/4" x 3 1/2"	8
0.5M	1/4" ANGLE	2" x 2" x 1 3/4"	2
N	1/4" PLATE	4" x 2"	7

LATERAL BRACING MEMBER SCHEDULE

MEMBER	MEMBER SIZE	CUT LENGTH	QUANTITY	TOTAL LENGTH
LL	HSS 2.0x1.0x0.12	2'-5 3/4"	5	12'-4 3/4"
MM1	HSS 2.0x1.0x0.12	3'-2 5/8"	1	3'-2 5/8"
MM2	HSS 2.0x1.0x0.12	3'-3 1/4"	1	3'-3 1/4"
MN1	HSS 2.0x1.0x0.12	1'-11 7/8"	1	1'-11 7/8"
MN2	HSS 2.0x1.0x0.12	1'-11 7/8"	1	1'-11 7/8"
MN3	HSS 2.0x1.0x0.12	1'-11 1/2"	4	7'-10"
MN4	HSS 2.0x1.0x0.12	1'-11 1/2"	4	7'-10"
MN5	HSS 2.0x1.0x0.12	1'-9"	2	3'-6"
MN6	HSS 2.0x1.0x0.12	1'-8 7/8"	2	3'-5 3/4"

TRUSS MEMBER SCHEDULE

MEMBER	MEMBER SIZE	CUT LENGTH	QUANTITY	TOTAL LENGTH
AA	HSS 2.0x1.5x0.12	3'-0"	4	12'-0"
AH	HSS 2.0x1.5x0.12	2'-5"	2	4'-10"
AJ	HSS 2.0x1.5x0.12	2'-1"	2	4'-2"
AK	HSS 2.0x1.5x0.12	0'-8"	1	0'-8"
AK2	HSS 2.0x1.5x0.12	0'-8"	1	0'-8"
BA	HSS 2.0x1.5x0.12	0'-8"	4	5'-4"
BA2	HSS 2.0x1.5x0.12	0'-8"	4	5'-4"
BC	HSS 2.0x1.5x0.12	3'-0"	8	24'-0"
BC2	HSS 2.0x1.5x0.12	3'-1"	2	6'-2"
BD	HSS 2.0x1.5x0.12	2'-11"	10	29'-2"
CD	HSS 2.0x1.5x0.12	0'-8"	4	5'-4"
CD2	HSS 2.0x1.5x0.12	0'-8"	4	5'-4"
CE	HSS 2.0x1.5x0.12	0'-8"	1	0'-8"
CE2	HSS 2.0x1.5x0.12	0'-8"	1	0'-8"
CI	HSS 2.0x1.5x0.12	2'-0"	2	4'-0"
CK	HSS 2.0x1.5x0.12	2'-1"	2	4'-2"
DD	HSS 2.0x1.5x0.12	3'-0"	4	12'-0"
EA	HSS 2.0x1.5x0.12	3'-1"	2	6'-2"
EE	HSS 2.0x1.5x0.12	1'-0"	2	2'-0"
FF	HSS 2.0x1.5x0.12	1'-0"	2	2'-0"
FH	HSS 2.0x1.5x0.12	2'-5"	2	4'-10"
FJ	HSS 2.0x1.5x0.12	2'-0"	2	4'-0"
GH	HSS 2.0x1.5x0.12	2'-4 1/4"	4	9'-5"
GI	HSS 2.0x1.5x0.12	2'-5"	4	9'-8"
IJ	HSS 2.0x1.5x0.12	1'-11 5/8"	4	7'-7 1/2"

Beyond Bridges

PROJECT: NAU 2025-2026 STEEL BRIDGE

PROJECT NOTES

SCALE: N/A

BY: MA

CHECKED BY: KJ, KF, & IK

DATE: MARCH 2026

REVISION: 03

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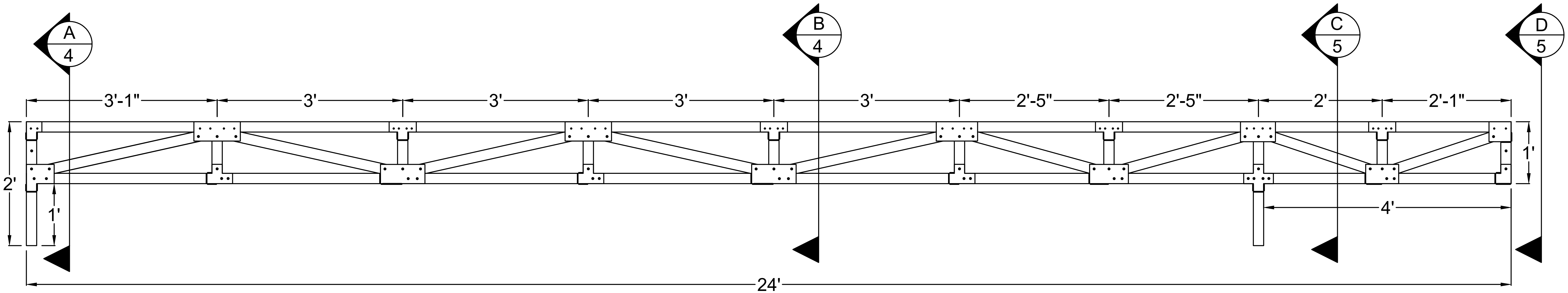
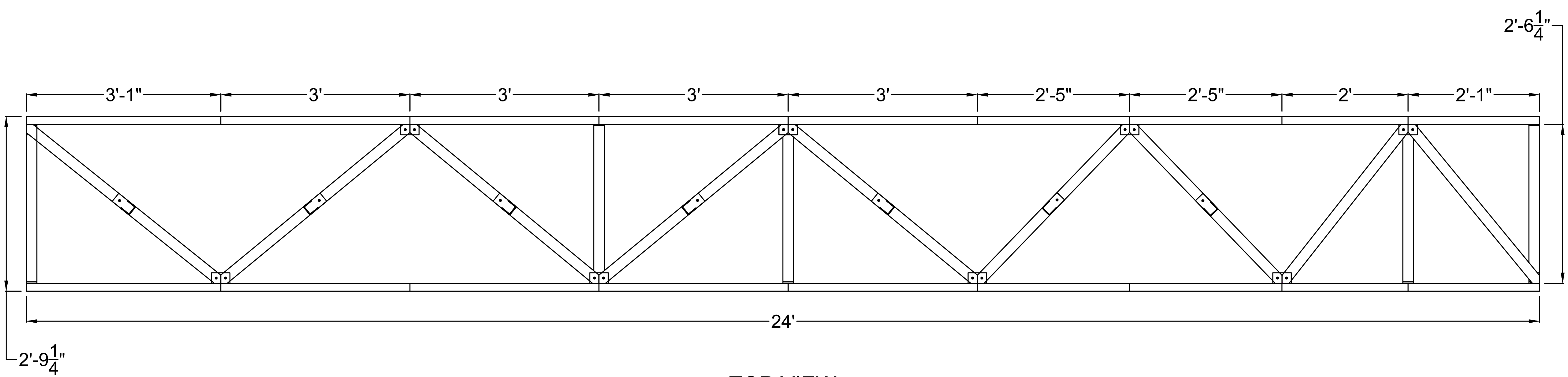


ELEVATION VIEW



TOP VIEW

PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 6" 1' 1' 6"	BY: MA	REVISION: 03	
ELEVATION AND TOP VIEW	SCALE: 5/8" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 3	OF: 34
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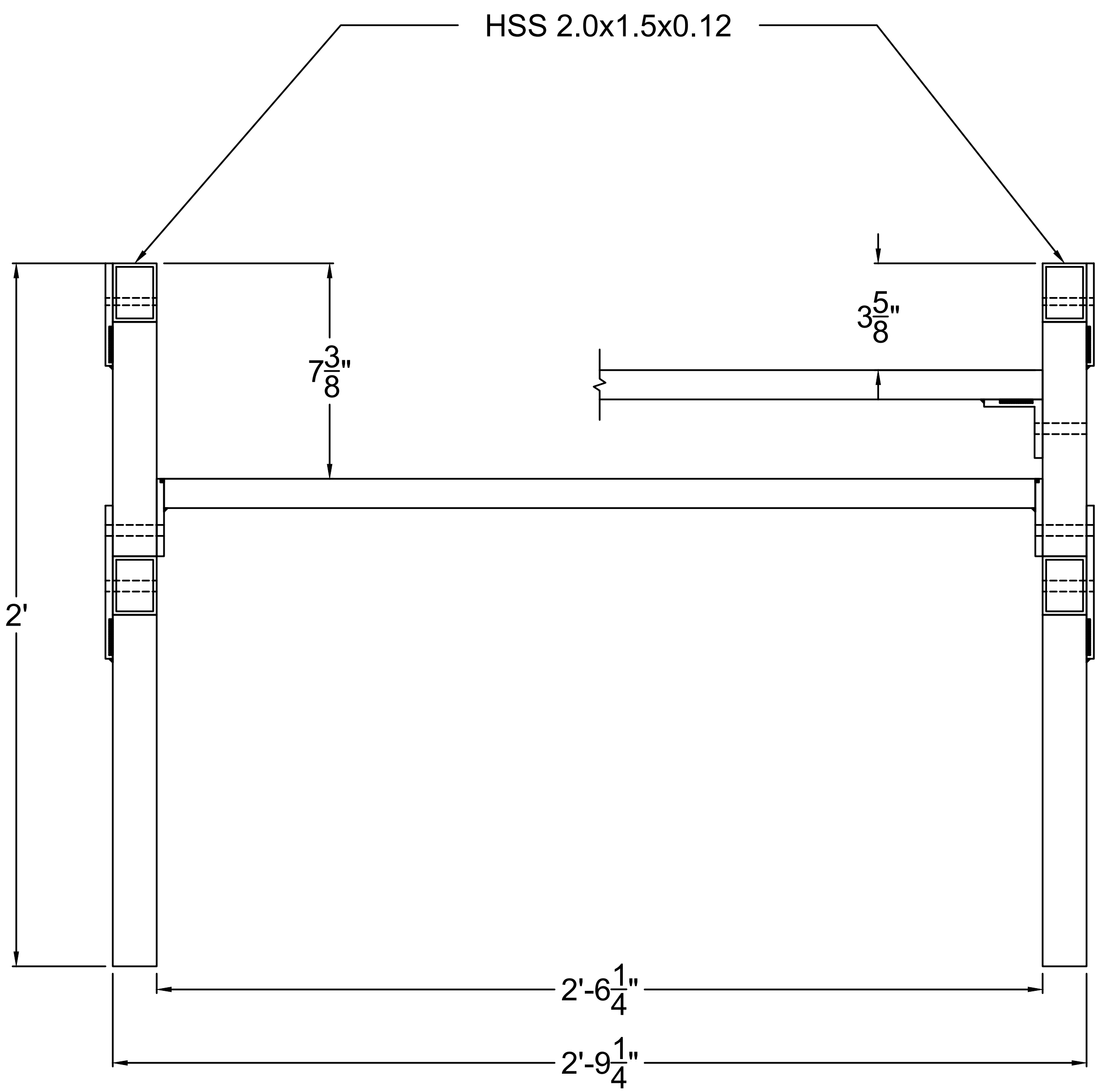
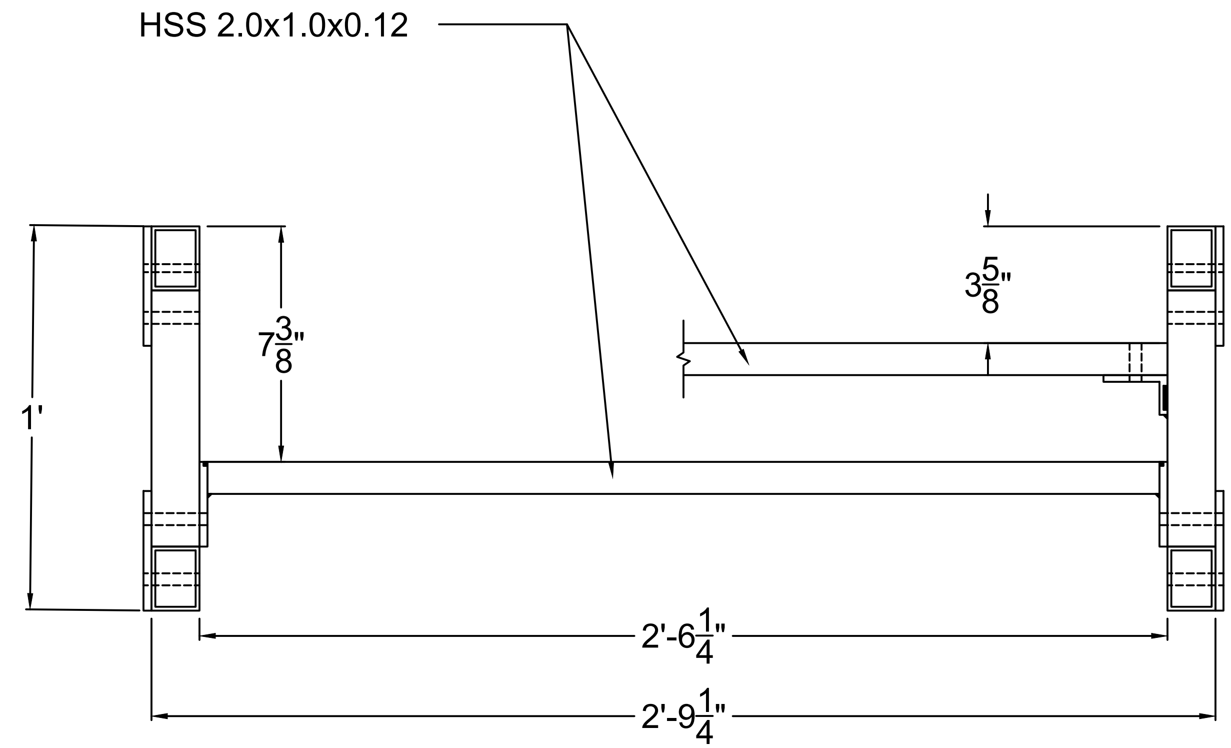
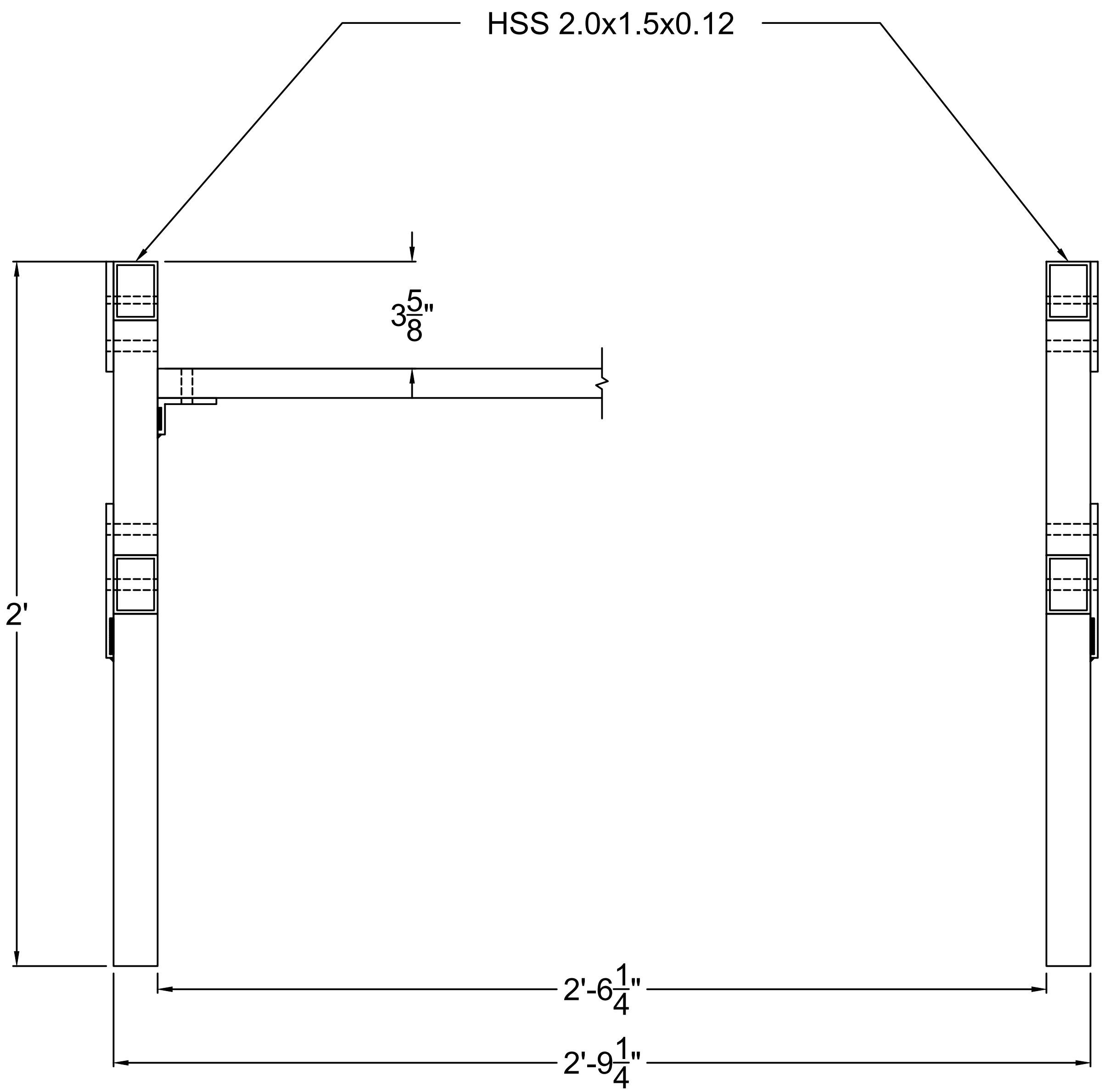


SECTION CUT A

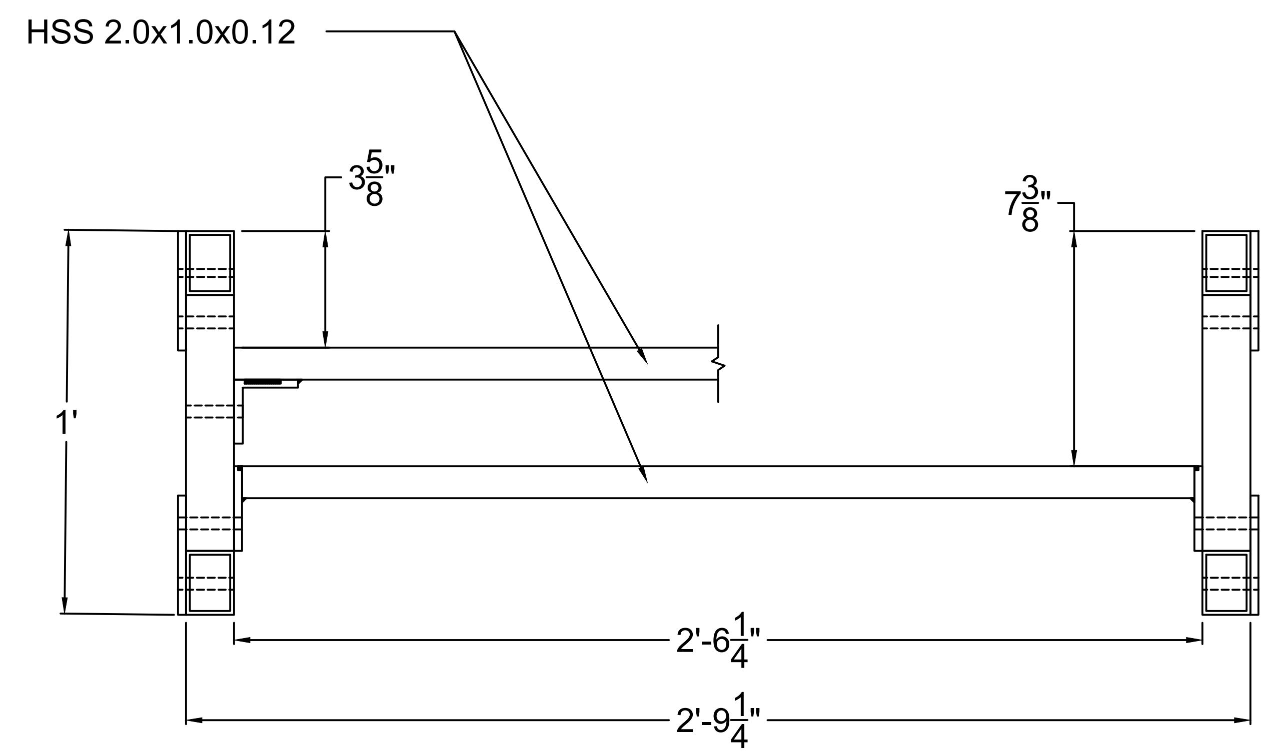


SECTION CUT B

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 3" 6"	BY: MA	REVISION: 03
	A & B SECTION CUTS	SCALE: 2" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 4 OF: 34
			DATE: MARCH 2026	



SECTION CUT C



SECTION CUT D

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 3" 6"	BY: MA	REVISION: 03
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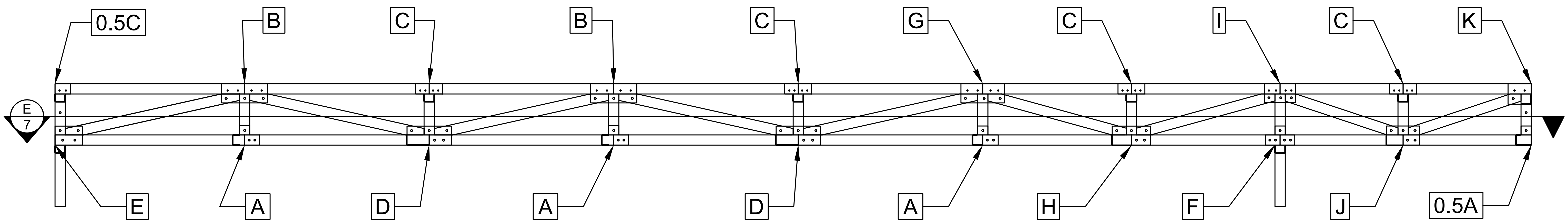

ELEVATION VIEW

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 6" 1' 1' 6"	BY: MA	REVISION: 03
	CONNECTION KEY - ELEVATION	SCALE: 5/8" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 6
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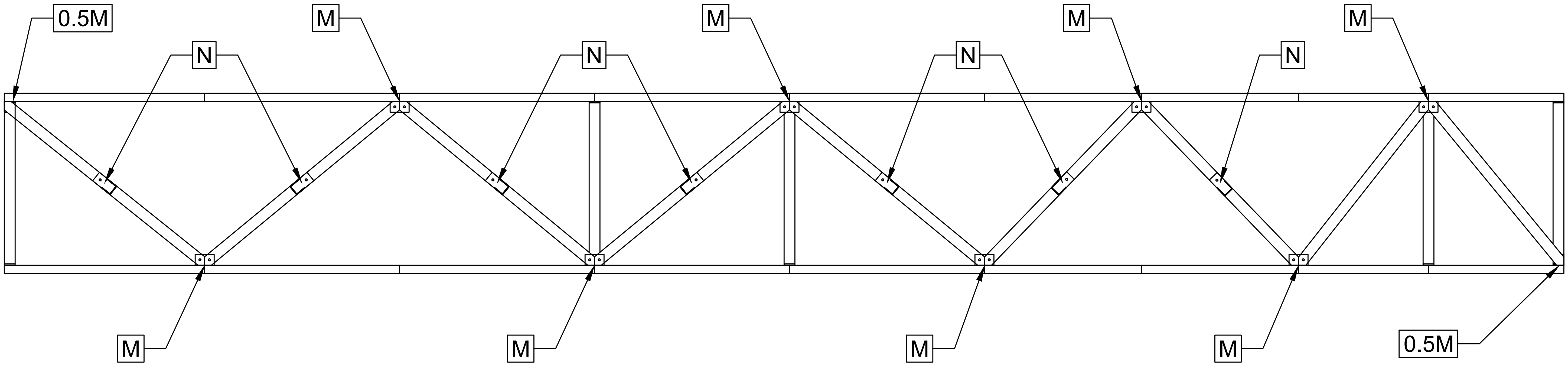
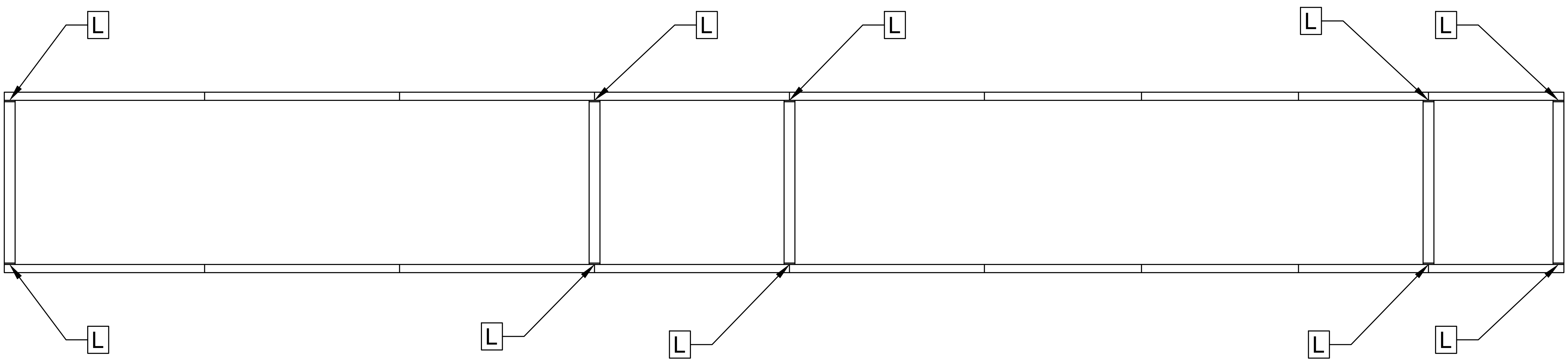
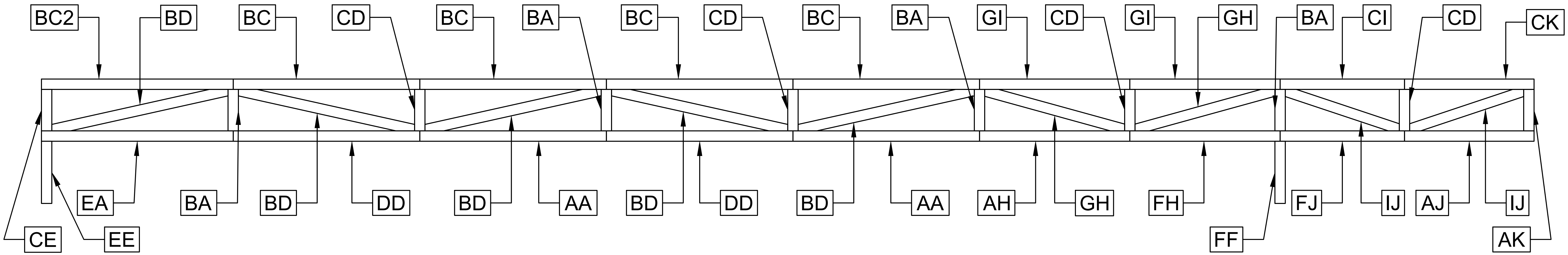


TOP VIEW

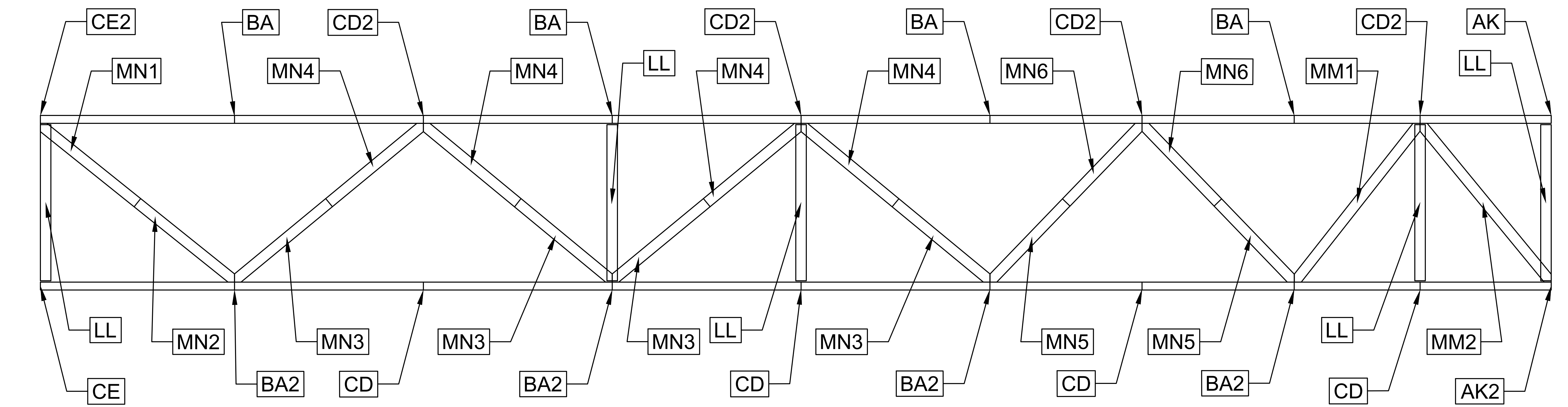


SECTION CUT E

PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 6" 1' 1' 6"	BY: MA	REVISION: 03	
CONNECTION KEY - TOP VIEW	SCALE: 5/8" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 7	OF: 34
		DATE: MARCH 2026		

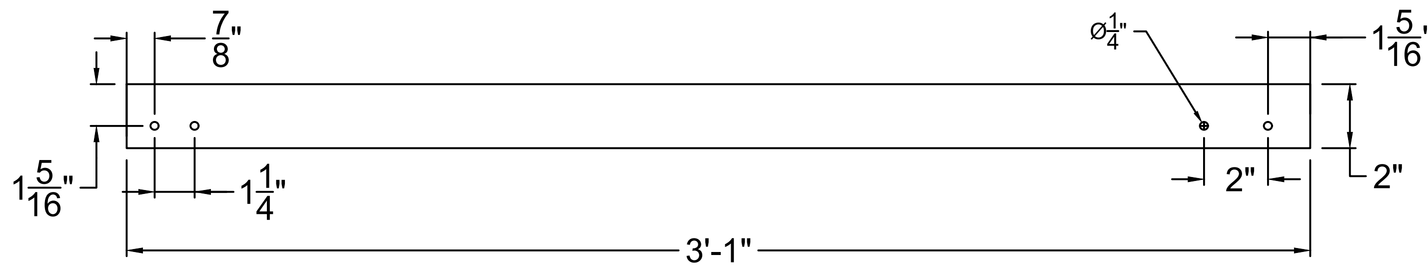


ELEVATION VIEW



TOP VIEW

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 6" 1' 1' 6"	BY: MA	REVISION: 03
	MEMBER KEY	SCALE: 5/8" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 8 OF: 34
			DATE: MARCH 2026	



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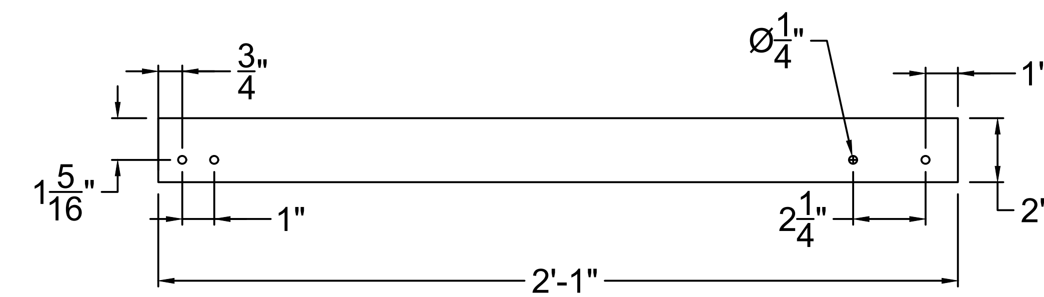
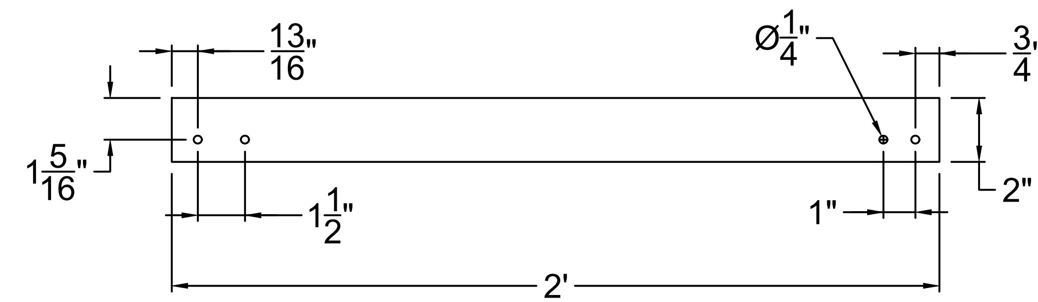


MEMBER CK

MEMBER BC



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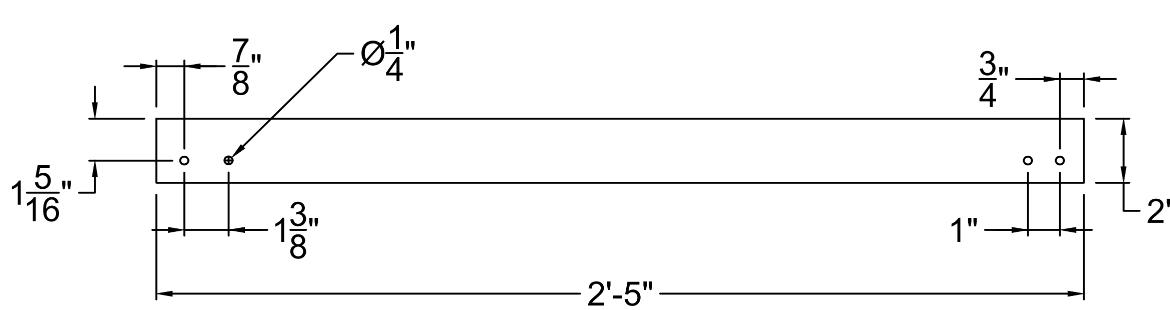
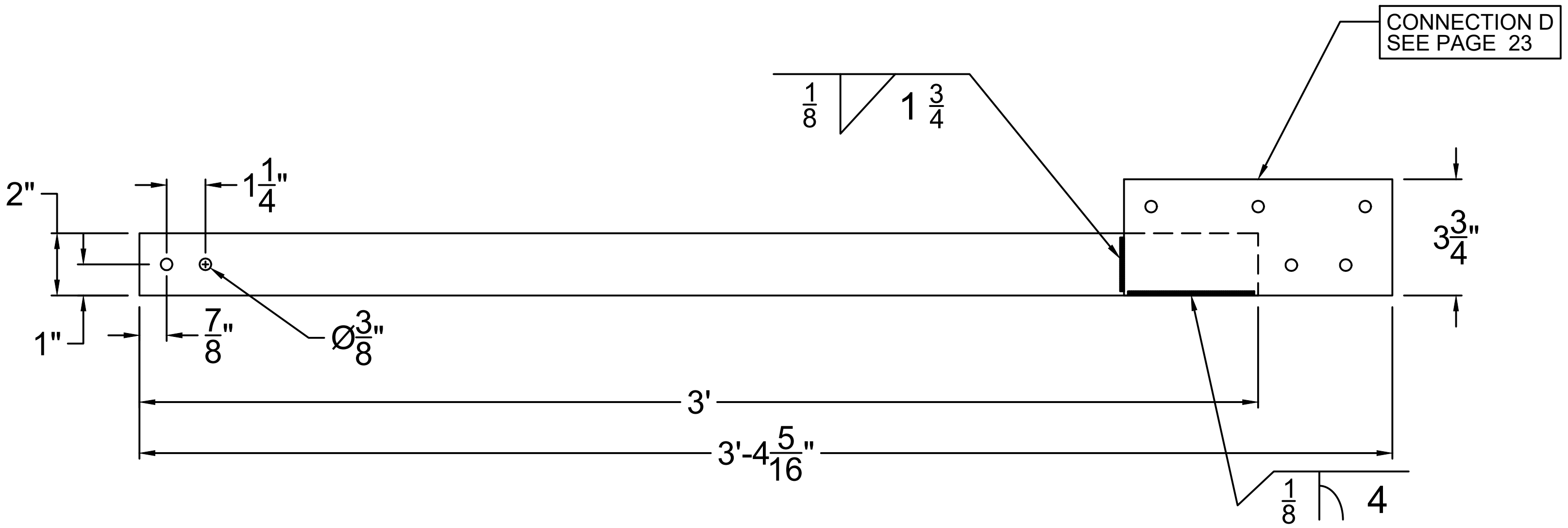
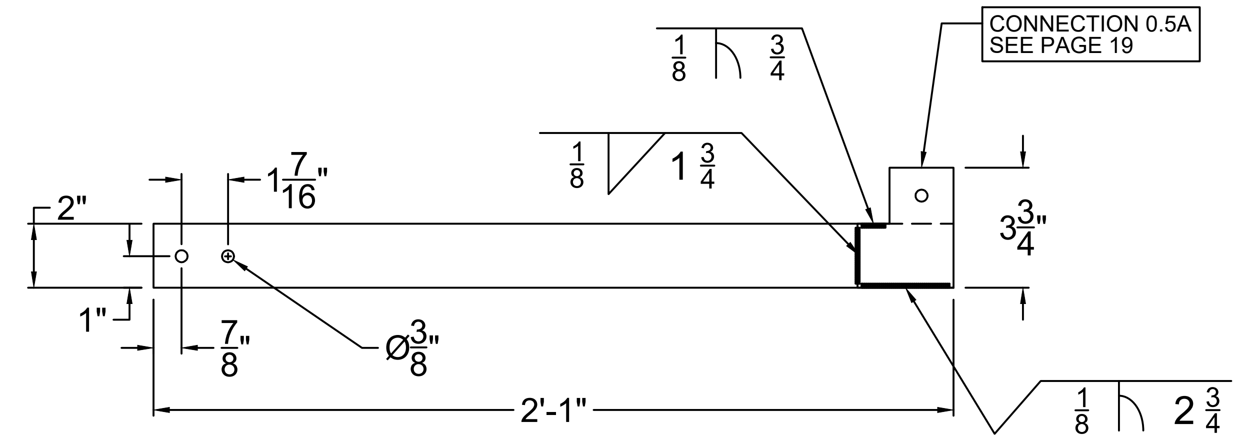


MEMBER GI

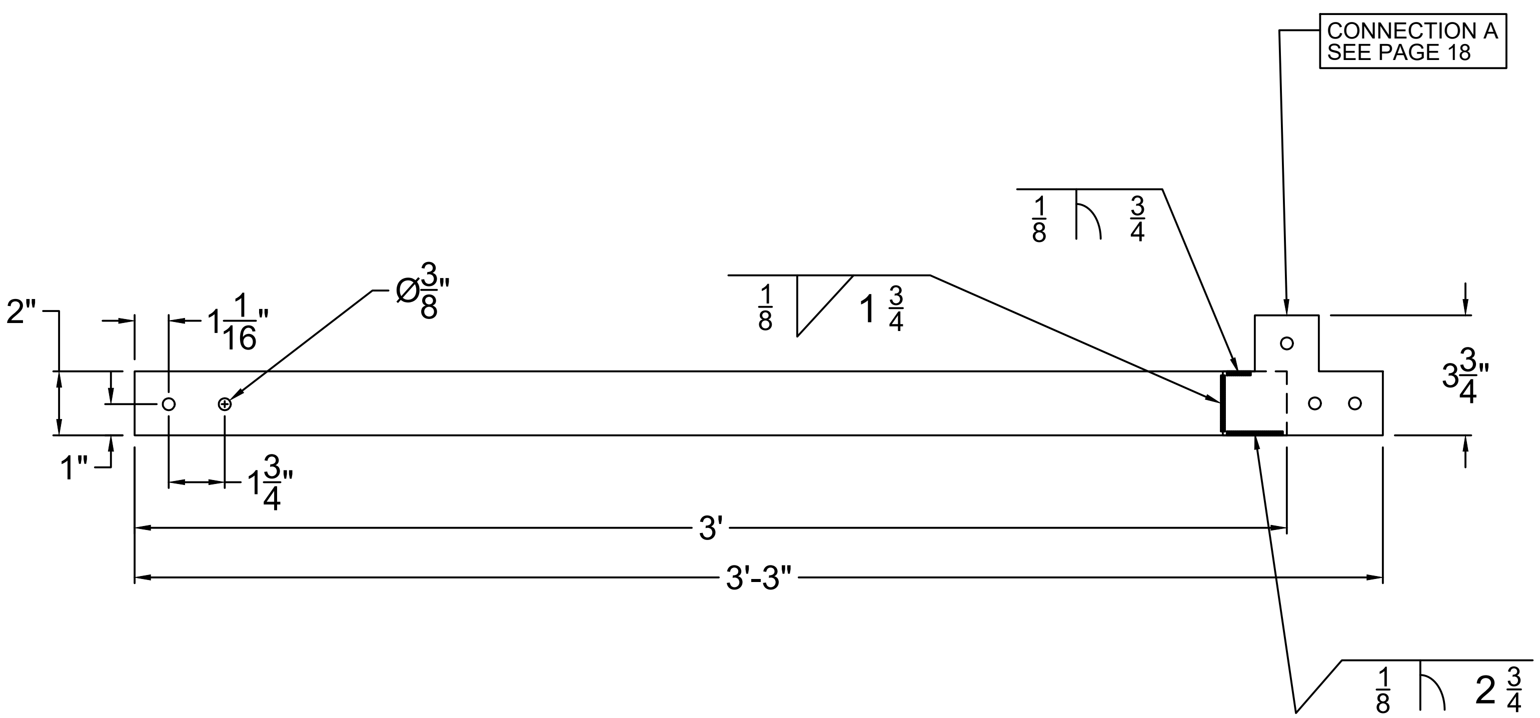
Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 3" 6"	BY: MA	REVISION: 03
	TOP CHORD DETAILS	SCALE: 2" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 9
			DATE: MARCH 2026	OF: 34



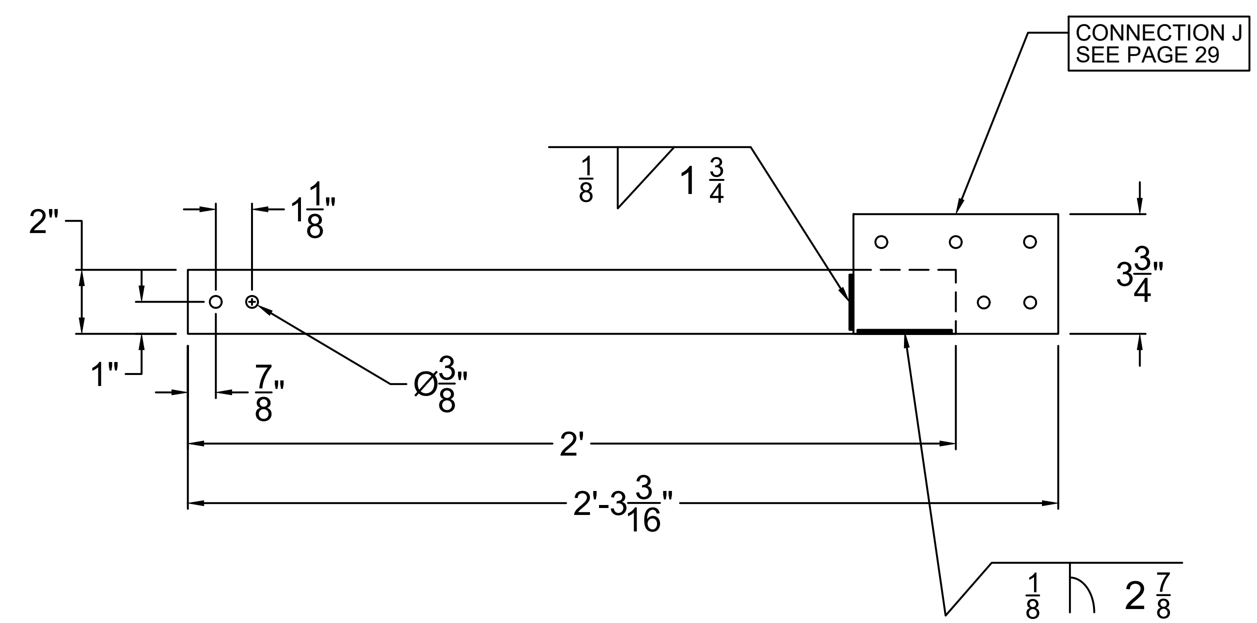
MEMBER DD



MEMBER AJ

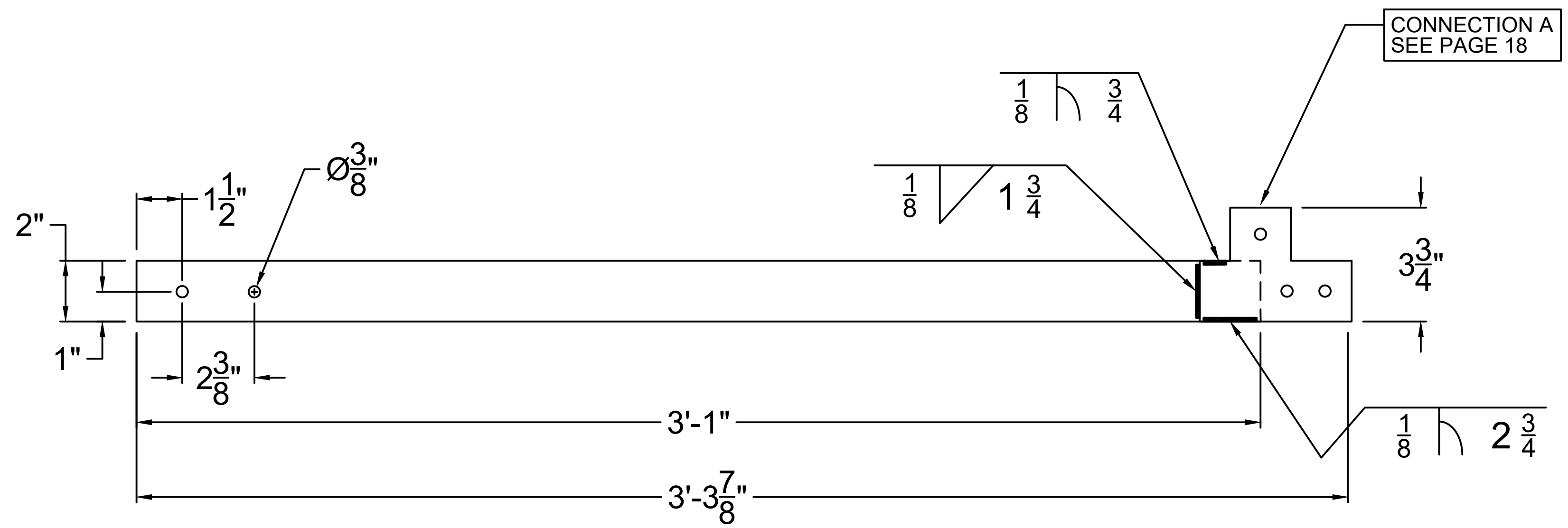


MEMBER AA

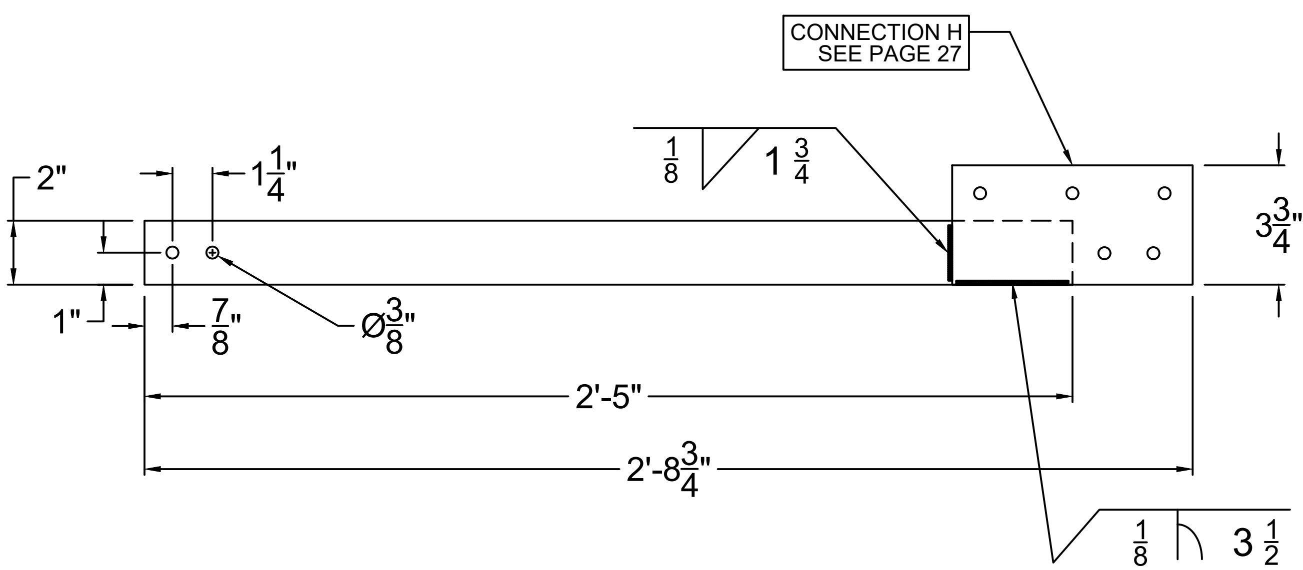


MEMBER FJ

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 3" 6"	BY: MA	REVISION: 03
	BOTTOM CHORD DETAIL	SCALE: 2" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 10
			DATE: MARCH 2026	OF: 34



MEMBER EA



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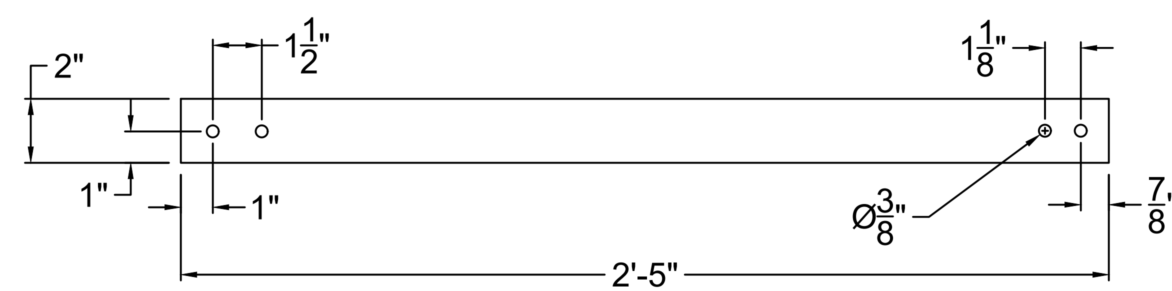
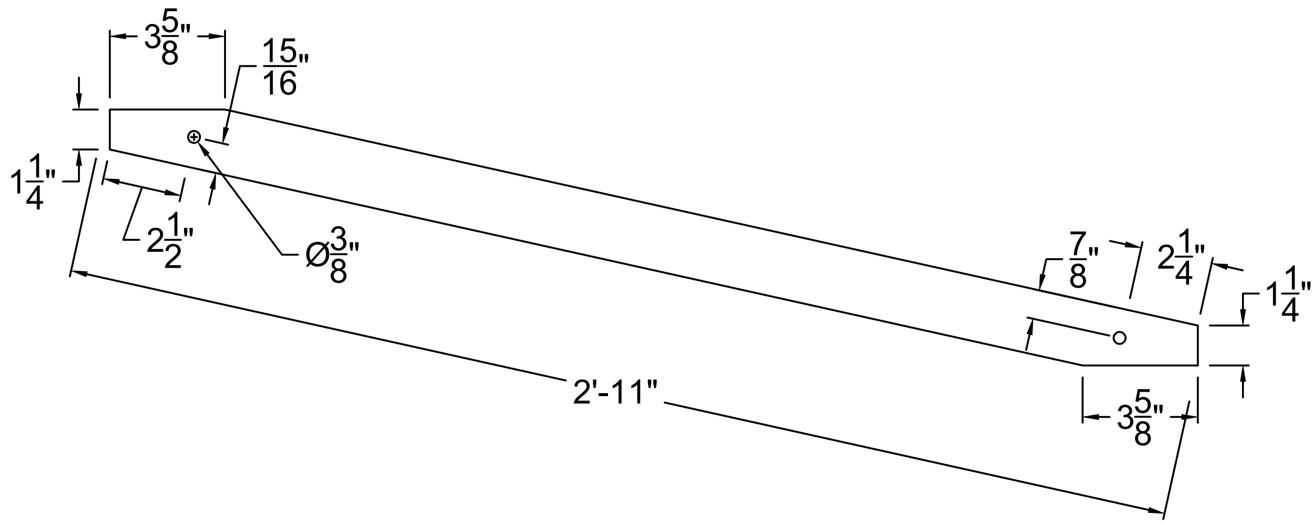


MEMBER FH

PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 3" 6"	BY: MA	REVISION: 03
BOTTOM CHORD DETAIL	SCALE: 2" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 11
		DATE: MARCH 2026	OF: 26



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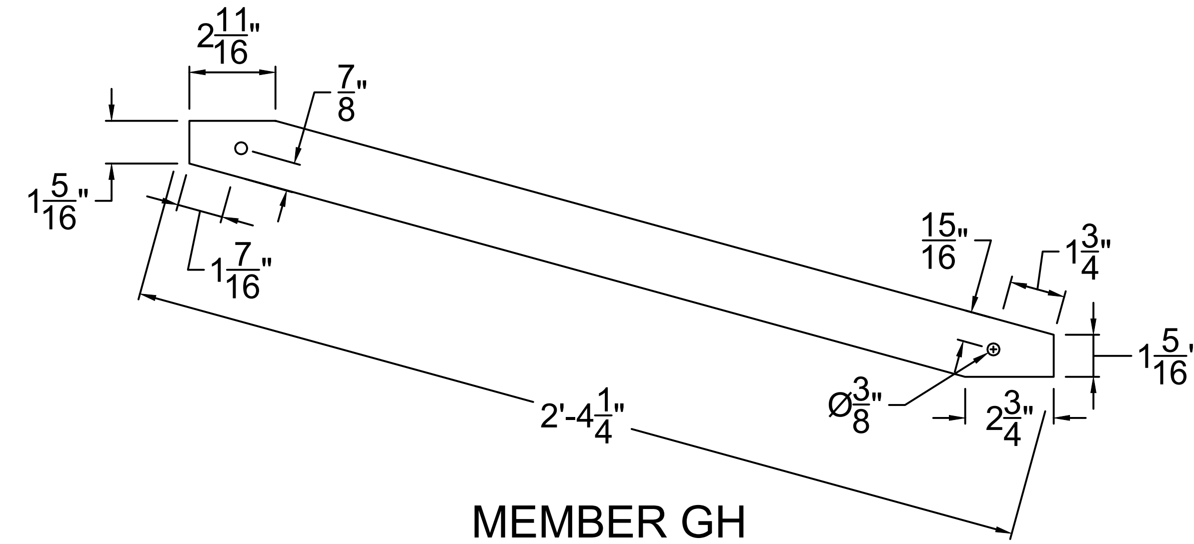

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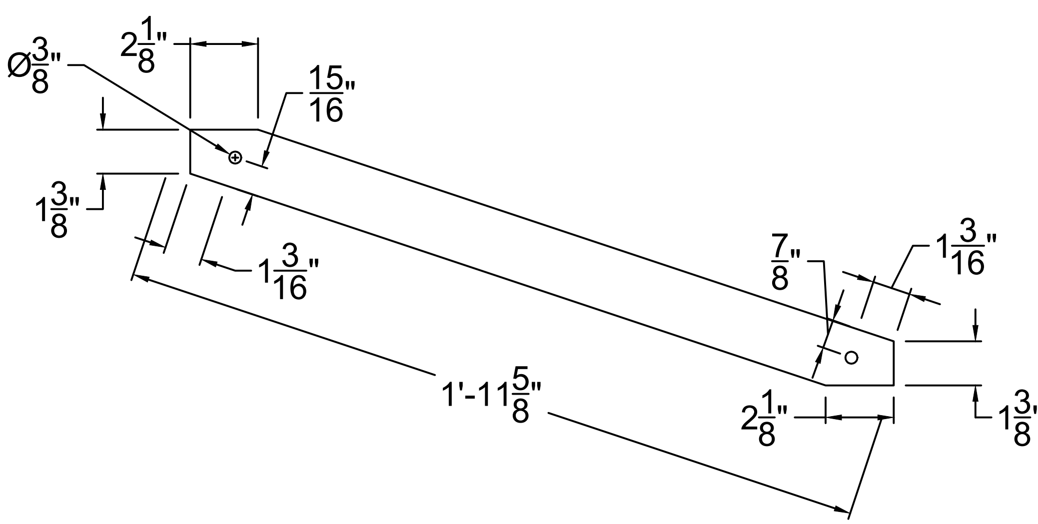
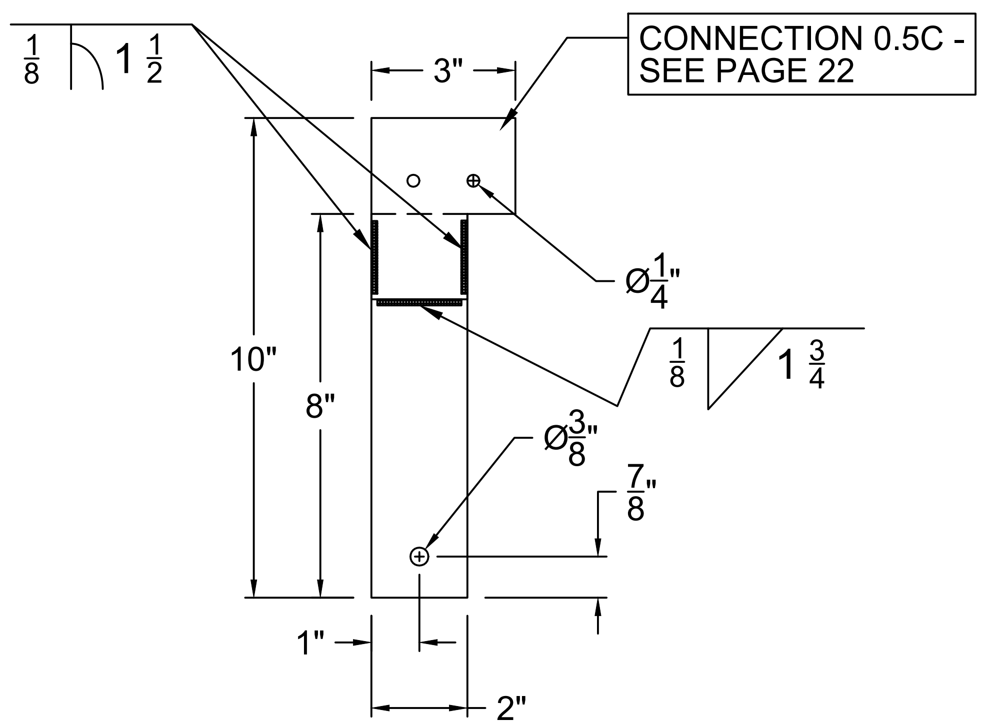


MEMBER IJ

PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 3" 6"	BY: MA	REVISION: 03	
DIAGONAL MEMBER DETAILS	SCALE: 2" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 12	OF: 34
		DATE: MARCH 2026		



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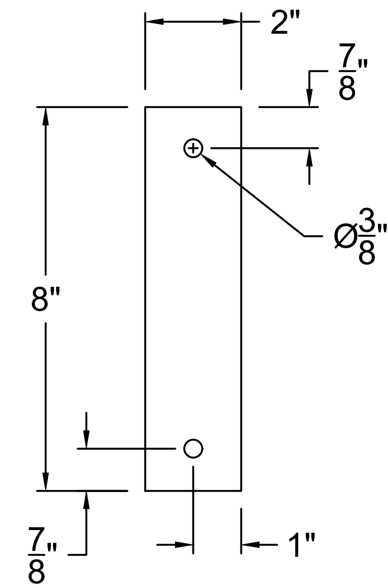

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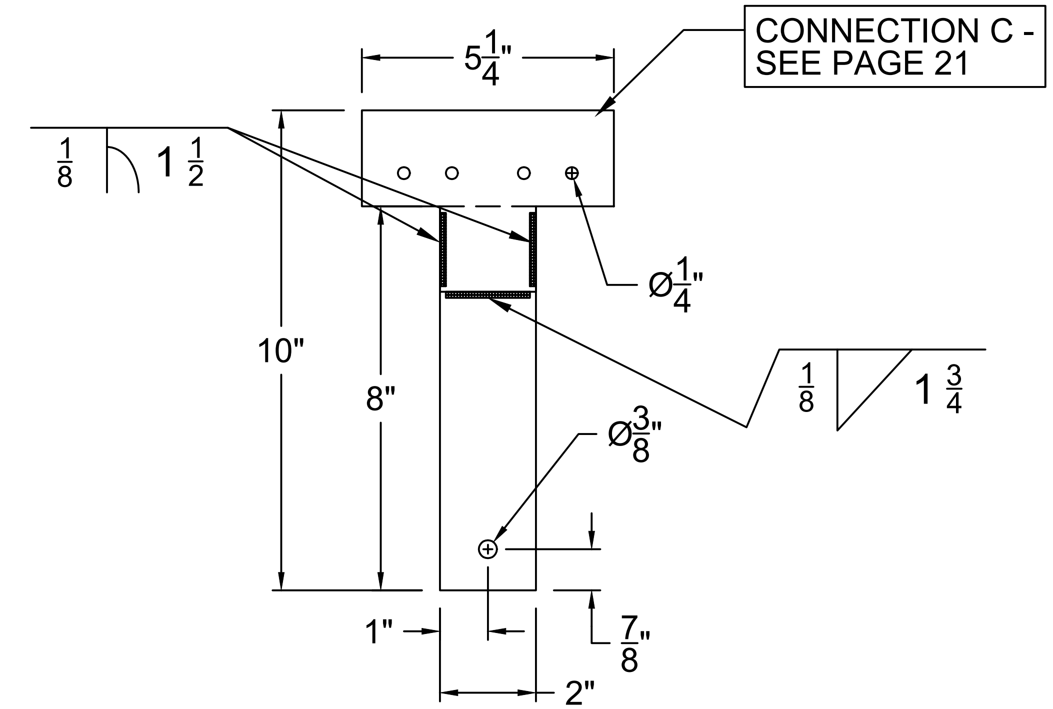

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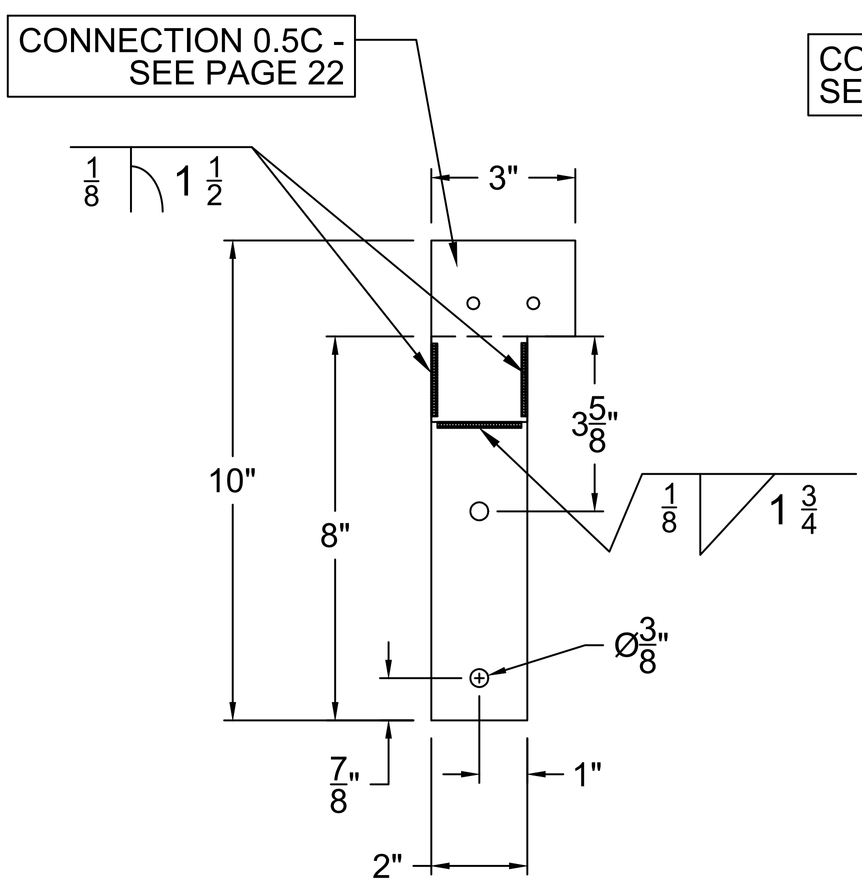

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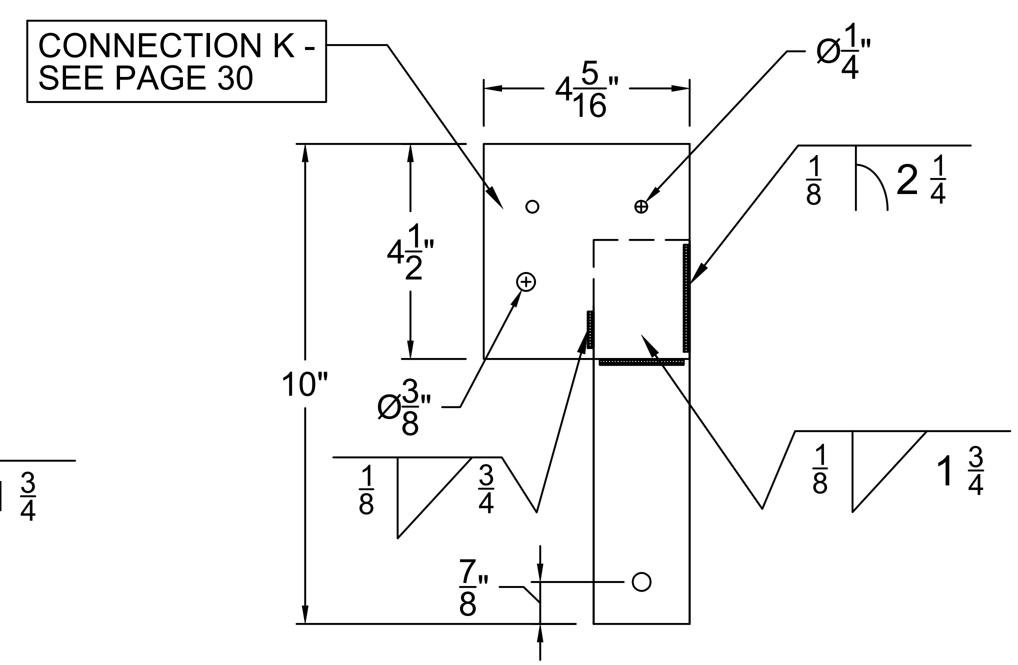

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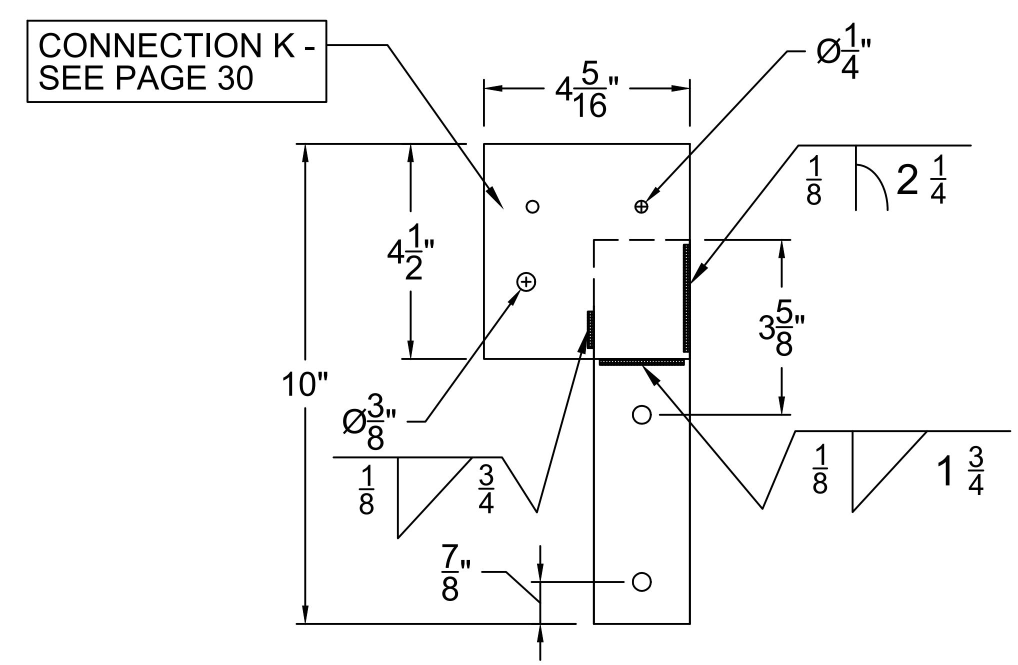
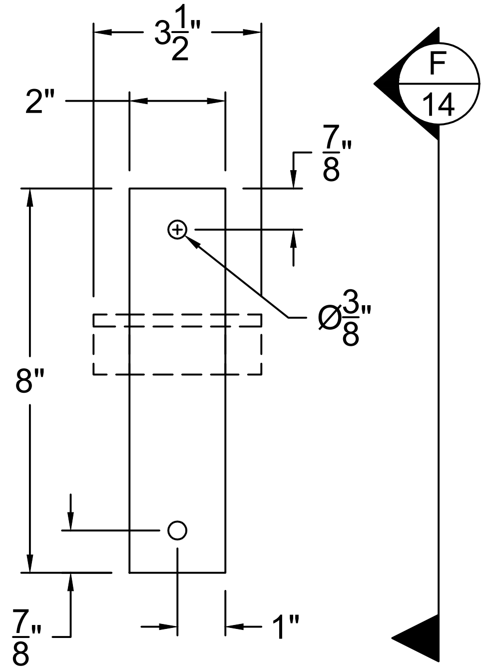


MEMBER AK2

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 2" 4" SCALE: 3" = 1'	BY: MA	REVISION: 03
	VERTICAL MEMBER DIMENSIONS		CHECKED BY: KJ, KF, & IK	PAGE: 13
			DATE: MARCH 2026	OF: 34



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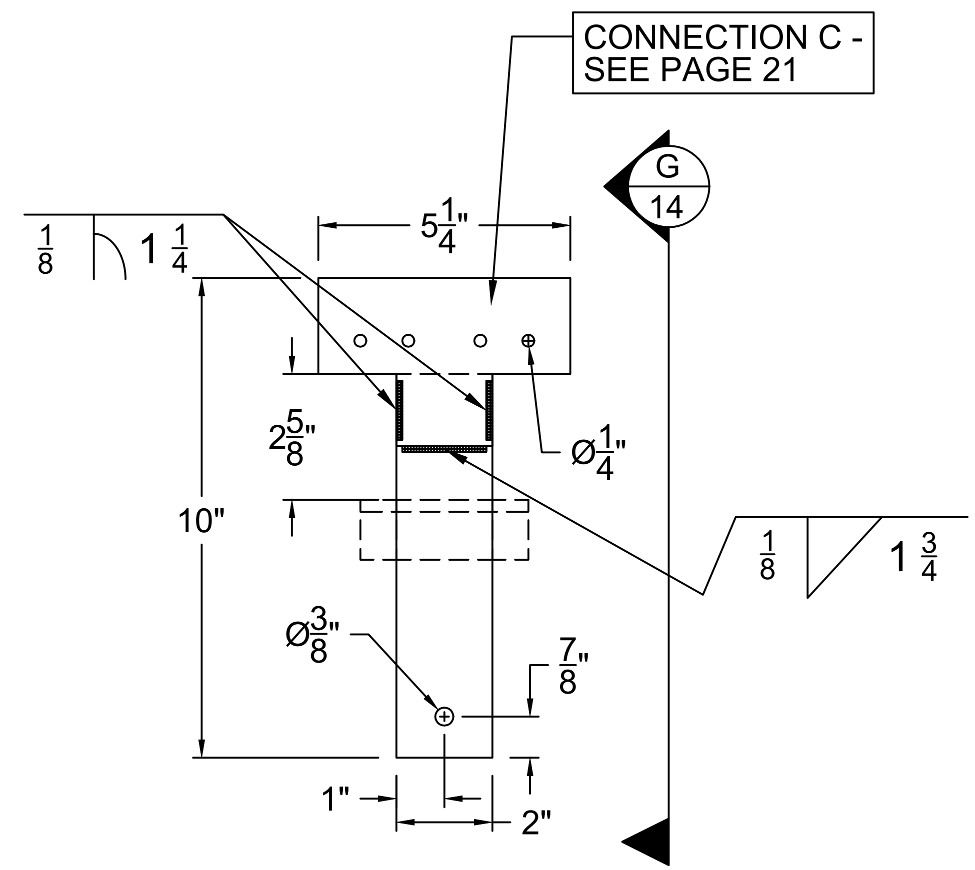

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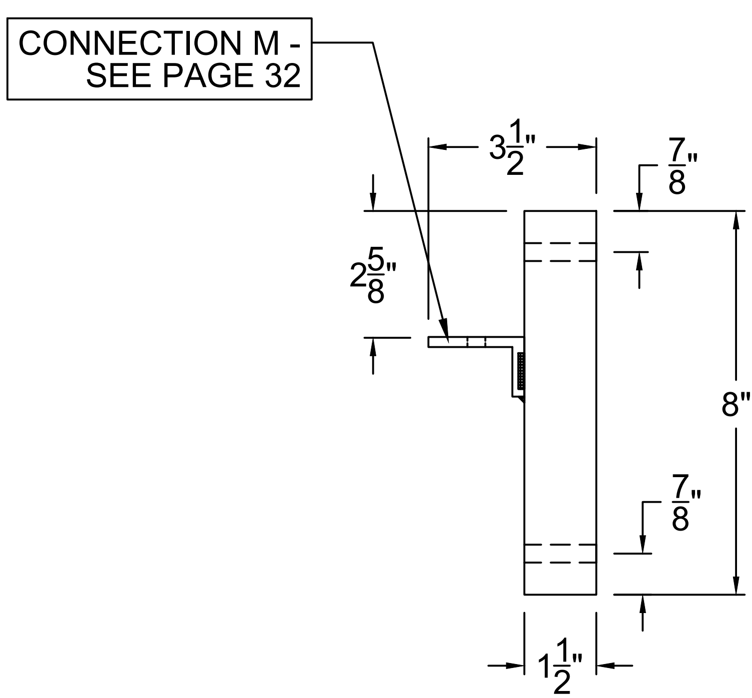
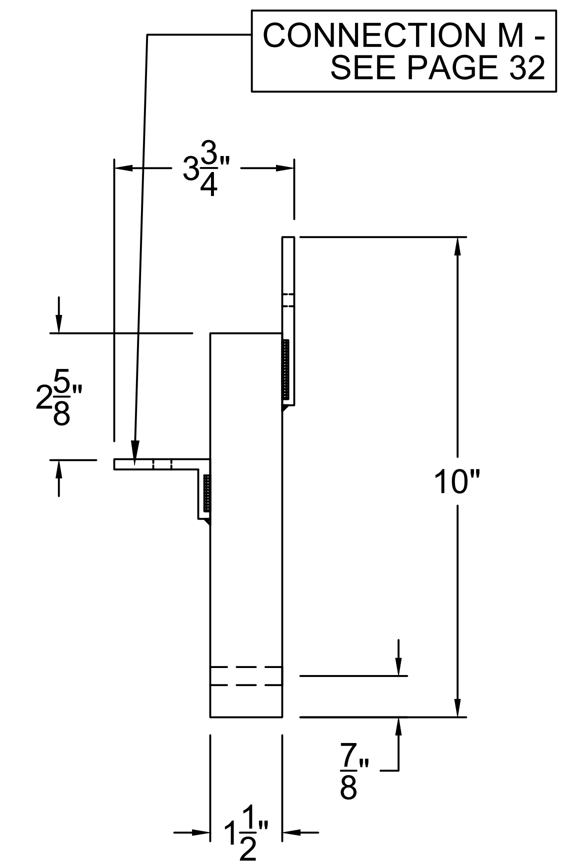
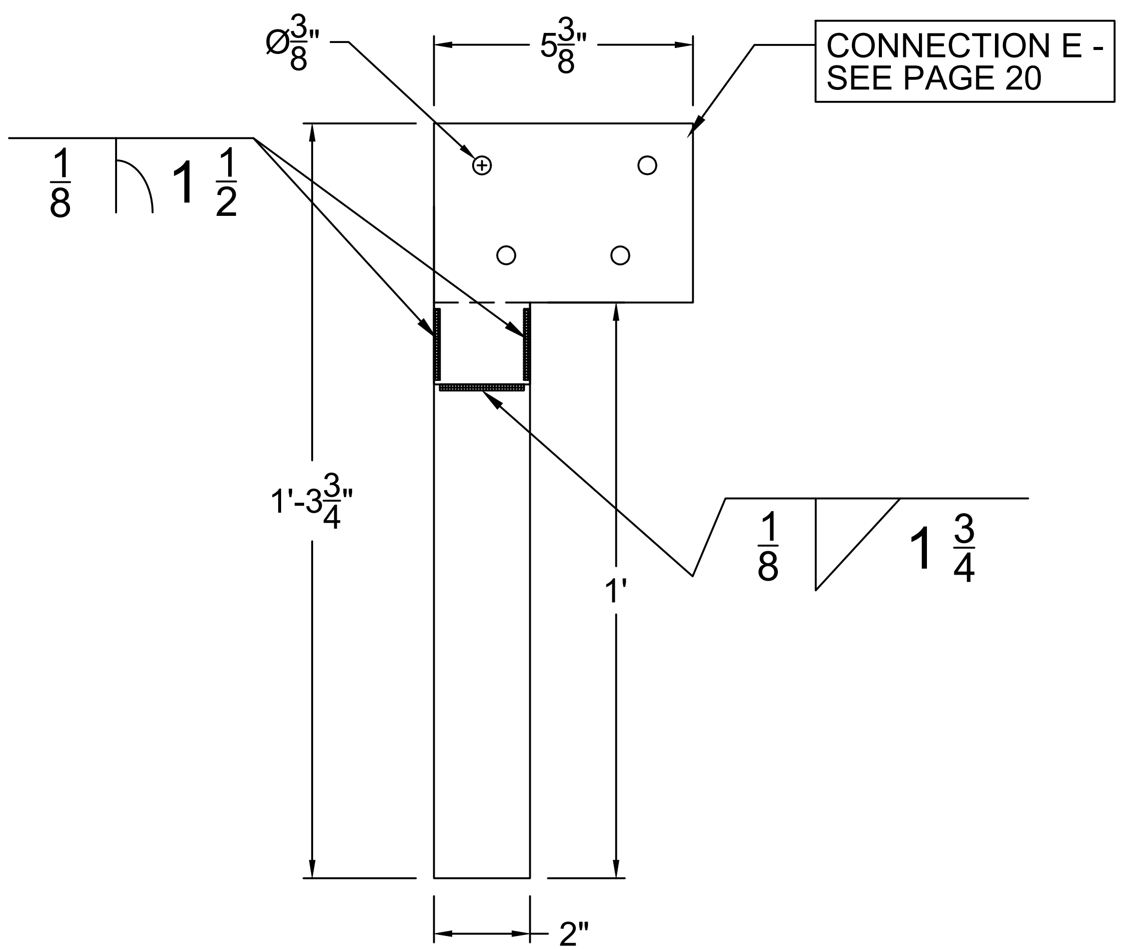


DETAIL E



DETAIL F

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 2" 4"	BY: MA	REVISION: 03
	VERTICAL MEMBER DIMENSIONS	SCALE: 3" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 14
			DATE: MARCH 2026	OF: 34



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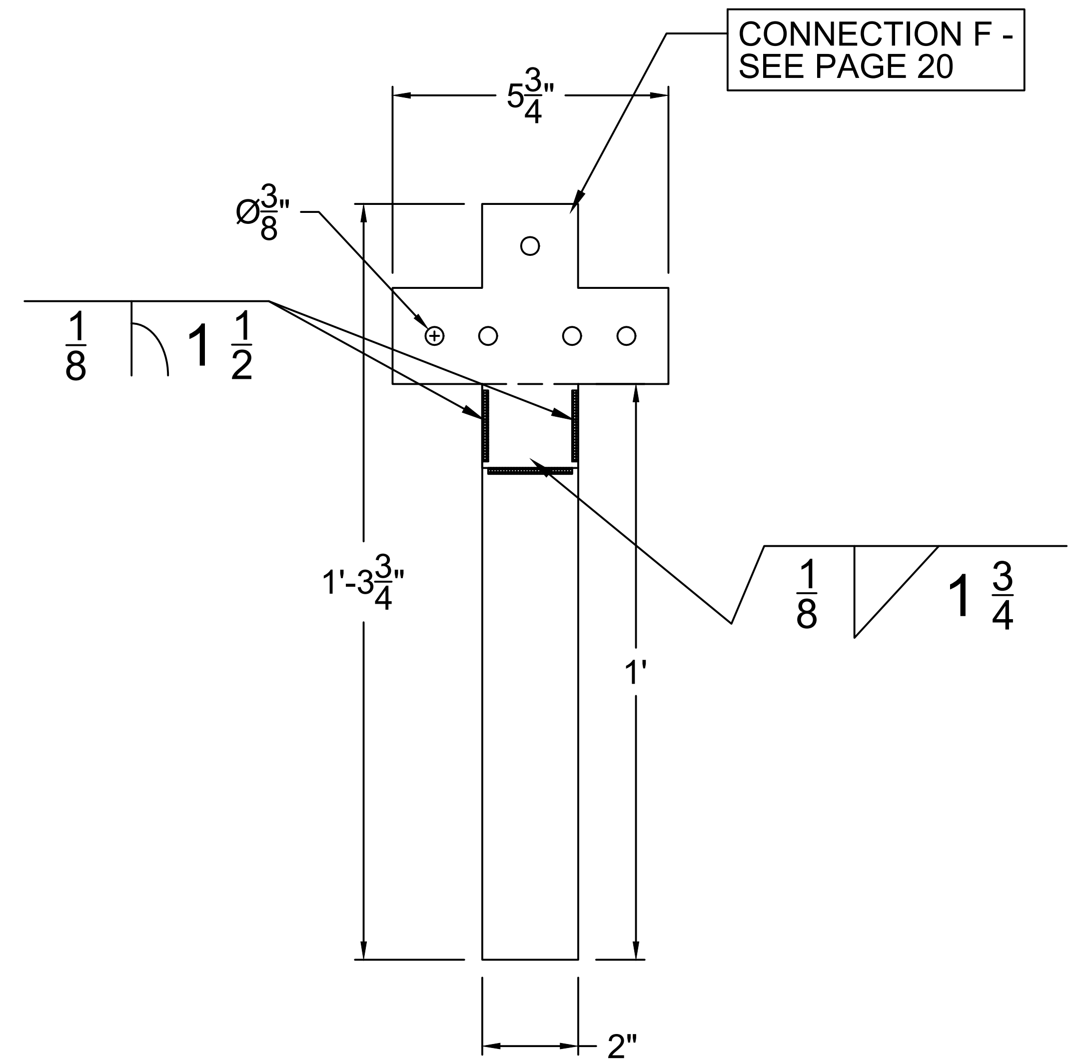
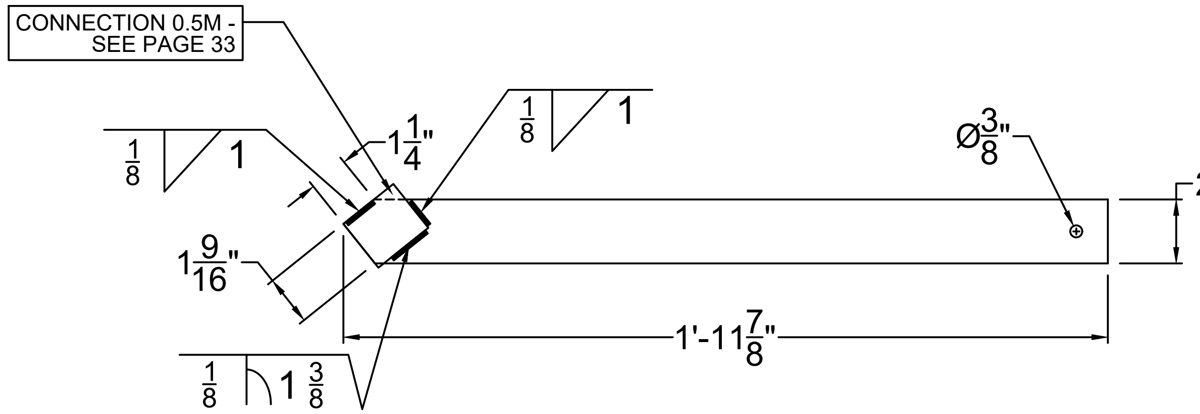
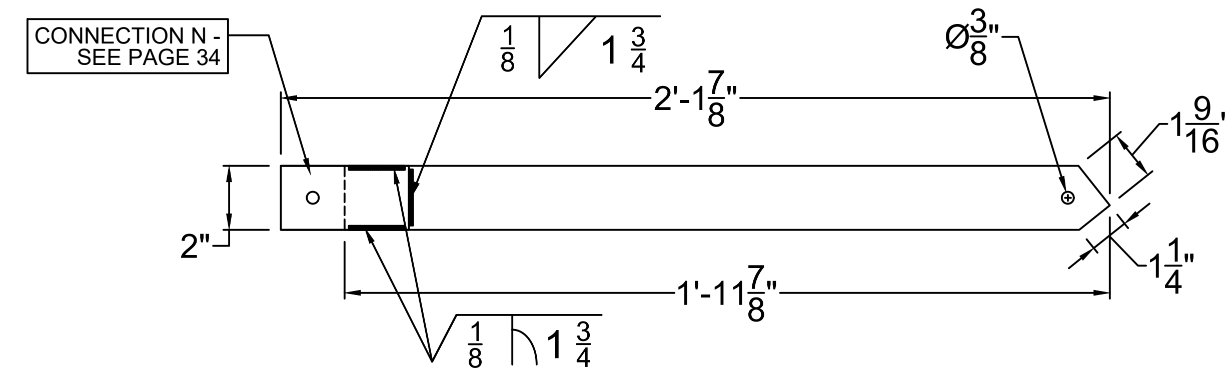


MEMBER FF

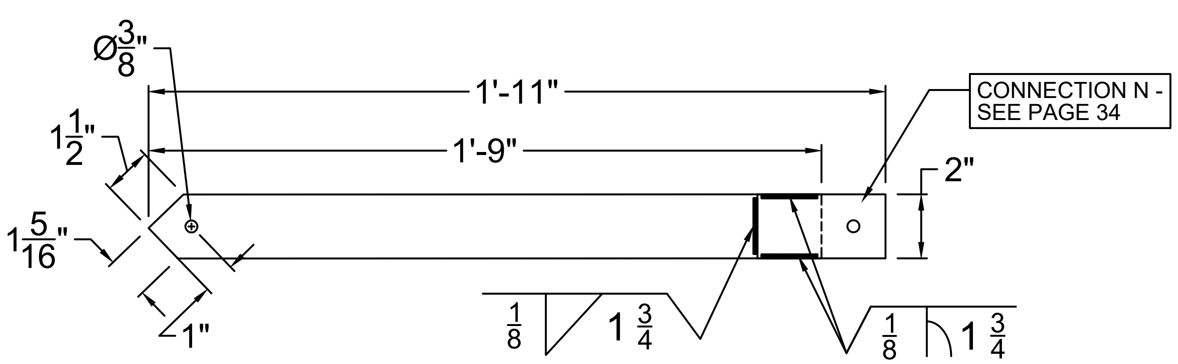
Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 2" 4"	BY: MA	REVISION: 03	
	FOOTING DETAIL	SCALE: 3" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 15	OF: 34
			DATE: MARCH 2026		



MEMBER MN1



MEMBER MN2



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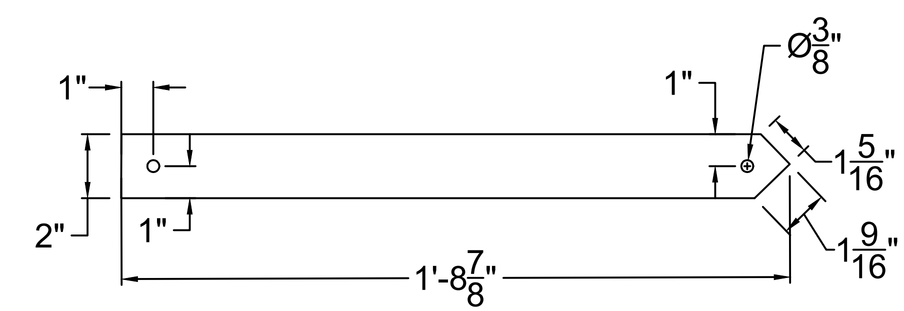

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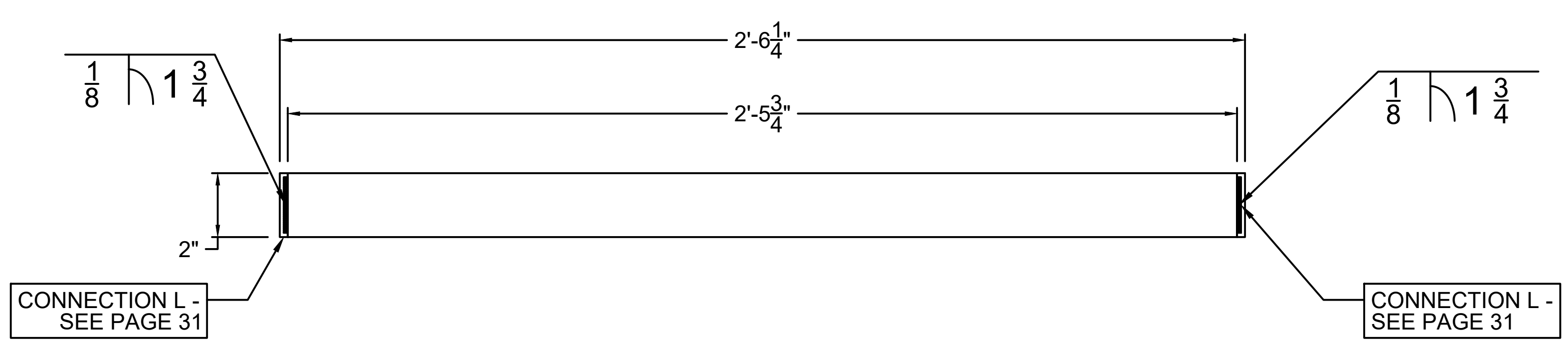
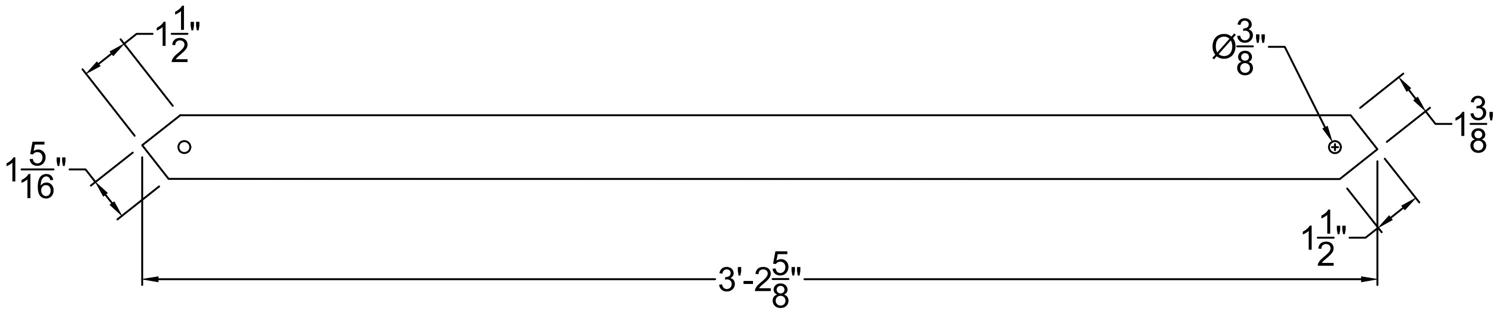


MEMBER LL

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE	0" 3" 6"	BY: MA	REVISION: 03
	LATERAL BRACING MEMBERS	SCALE: 2" = 1'	CHECKED BY: KJ, KF, & IK	PAGE: 16
			DATE: MARCH 2026	OF: 34



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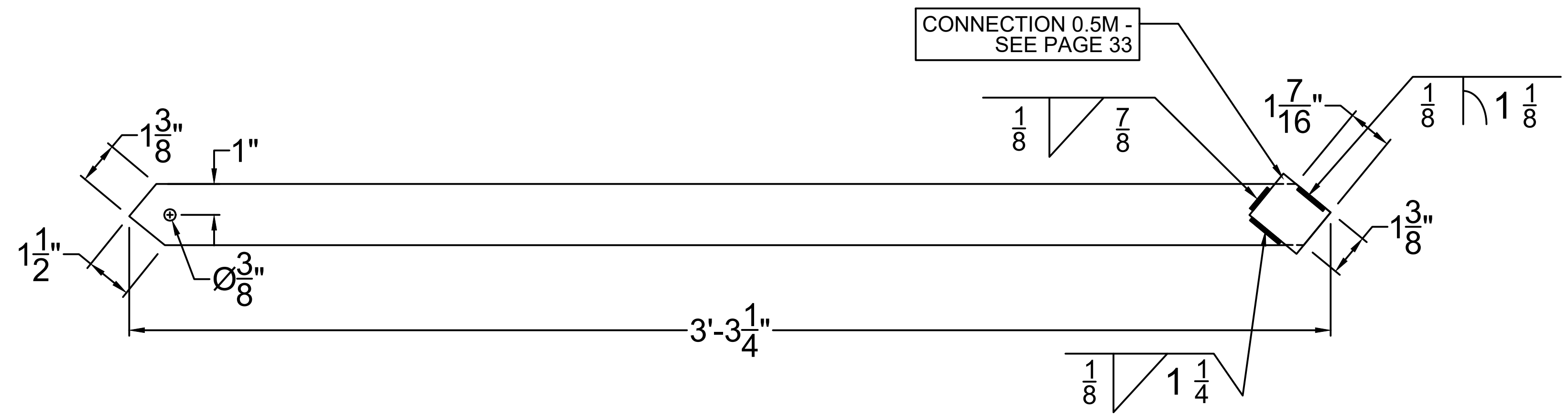

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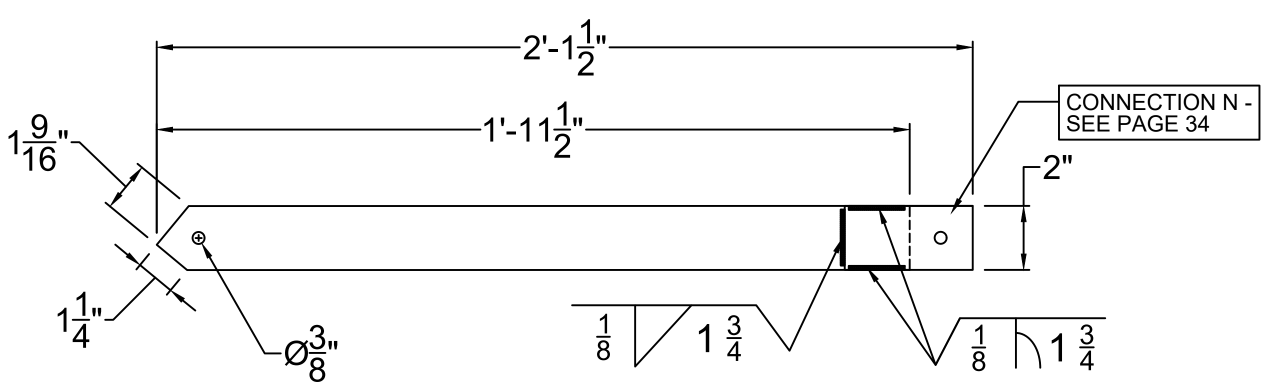

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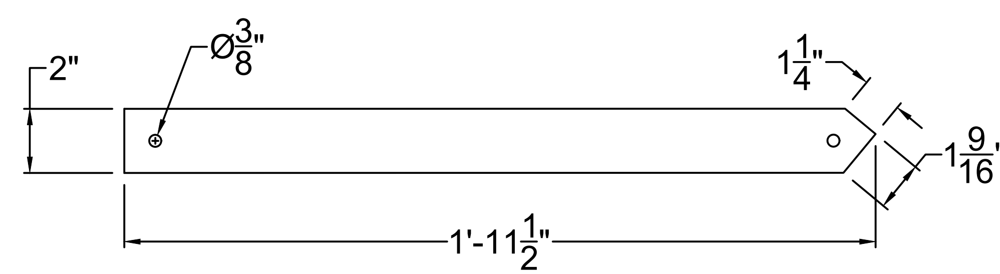

MEMBER MN4

Beyond Bridges

PROJECT: NAU 2025-2026 STEEL BRIDGE

LATERAL BRACING MEMBERS

0" 3" 6"

SCALE: 2" = 1'

BY: MA

CHECKED BY: KJ, KF, & IK

DATE: MARCH 2026

REVISION: 03

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OF: 34

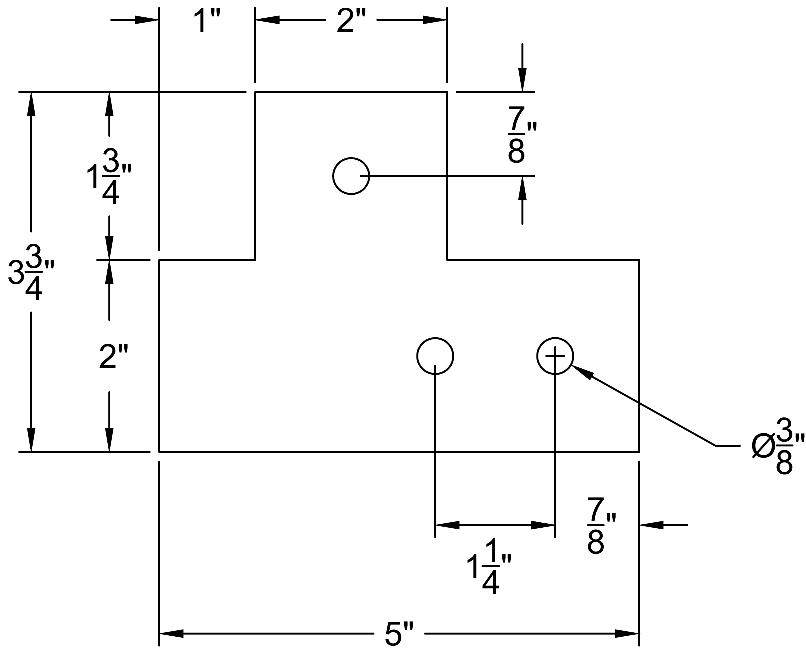
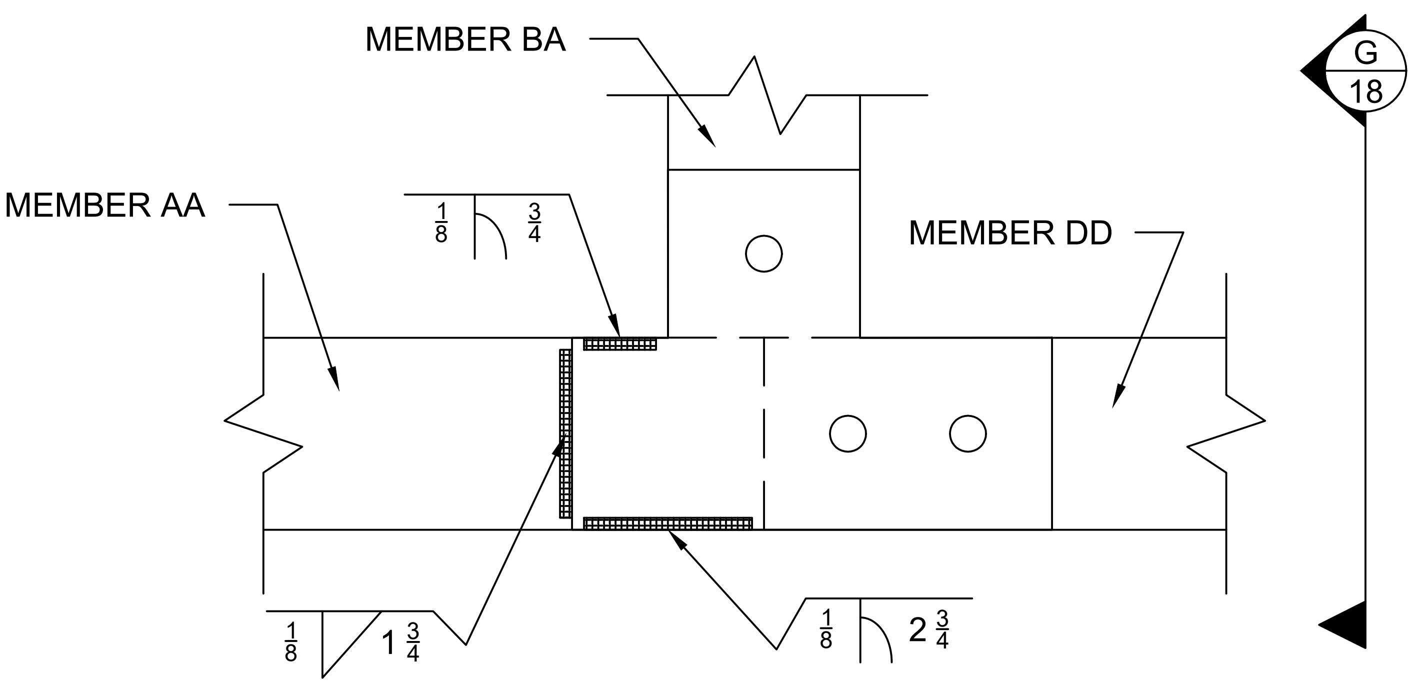


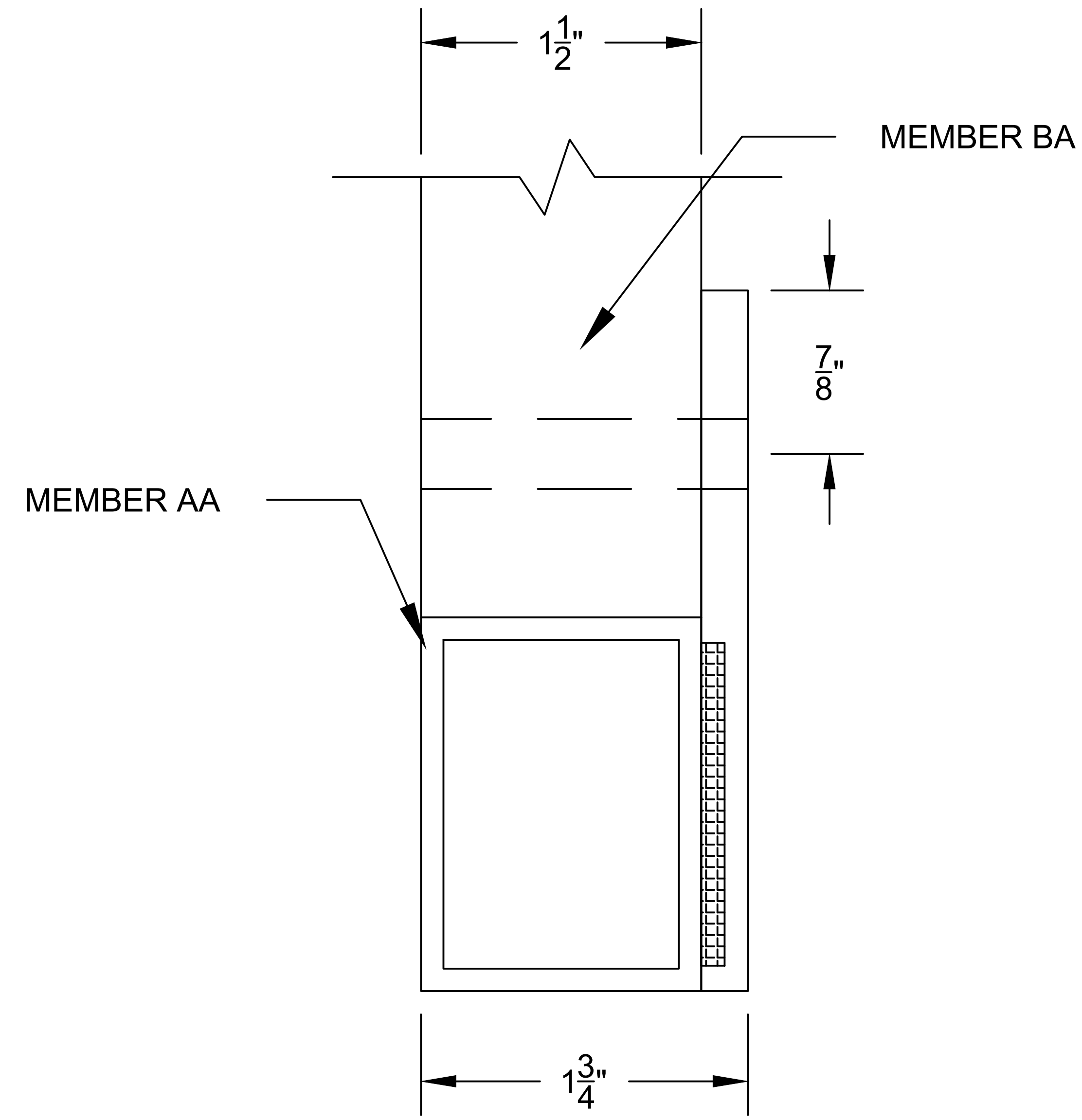
PLATE DIMENSION - CONNECTION A

SCALE: 6" = 1'



PROFILE VIEW - CONNECTION A

SCALE: 6" = 1'



DETAIL G

SCALE: 1" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03
	CONNECTION DETAIL A	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 18
			DATE: MARCH 2026	OF: 34

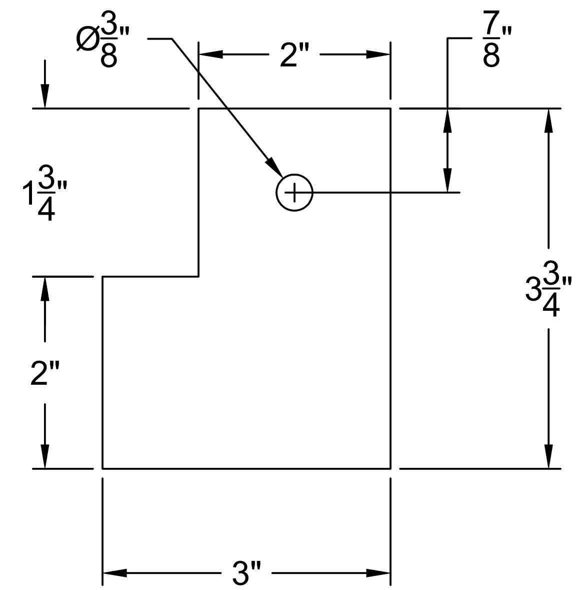


PLATE DIMENSION - CONNECTION 0.5A

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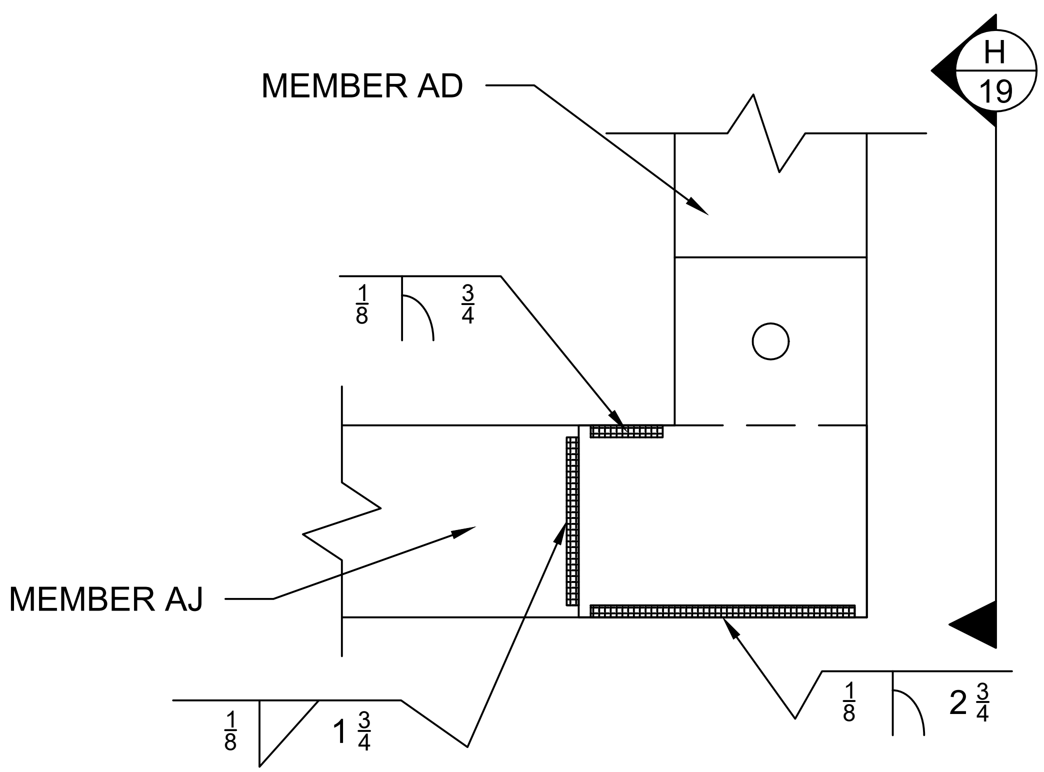
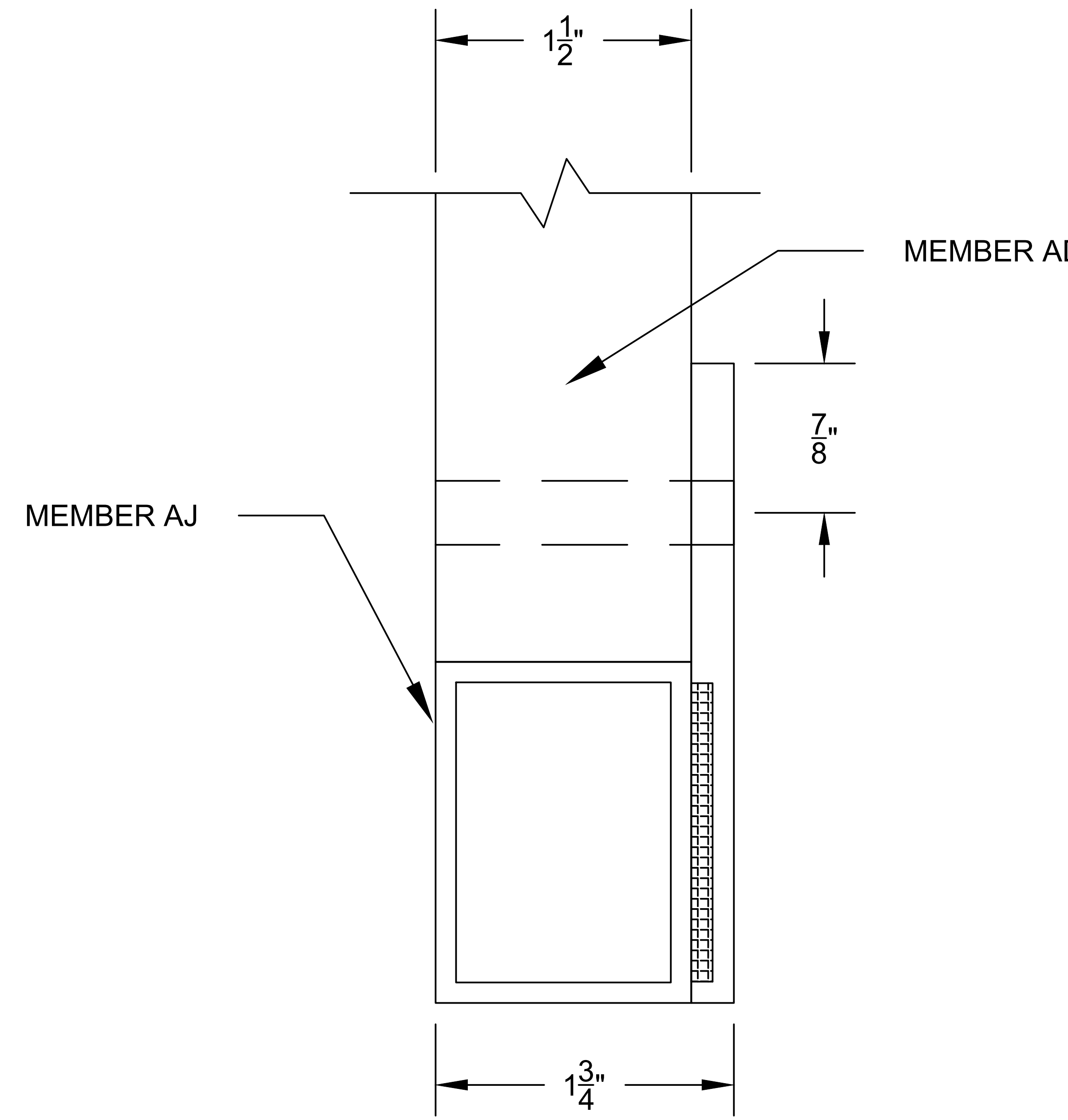


PROFILE VIEW - CONNECTION 0.5A

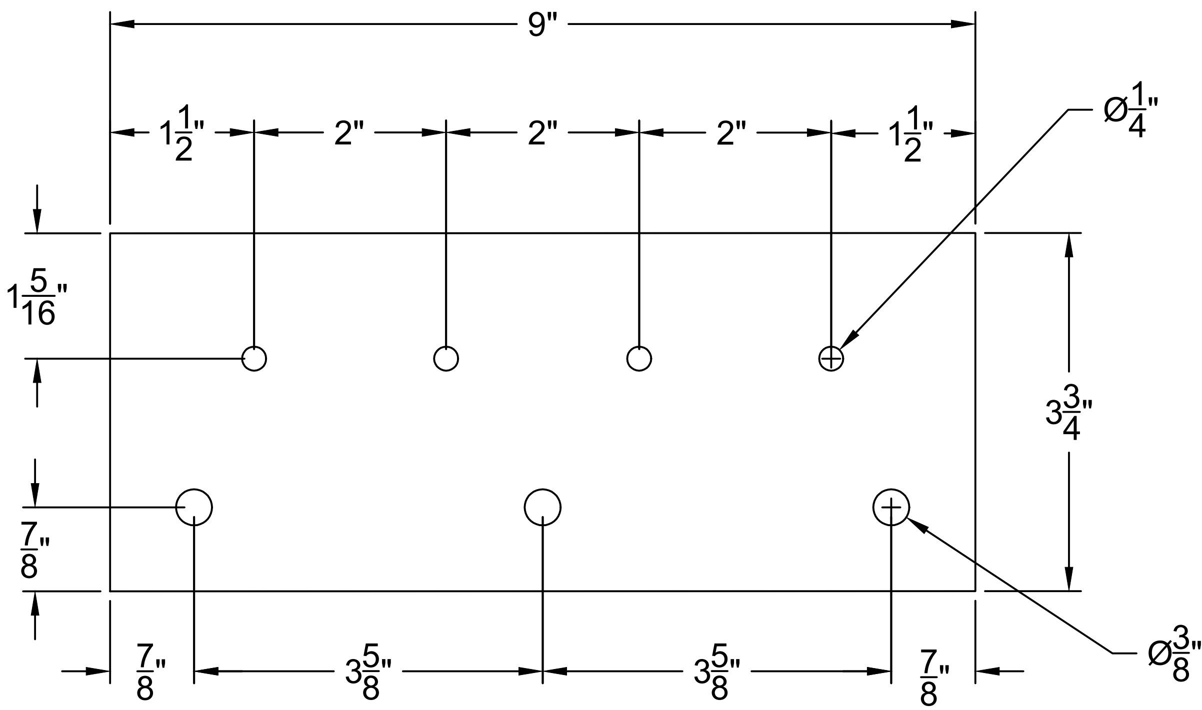
SCALE: 6" = 1'



DETAIL H

SCALE: 1" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03
	CONNECTION DETAIL 0.5A	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 19 / OF: 34
			DATE: MARCH 2026	



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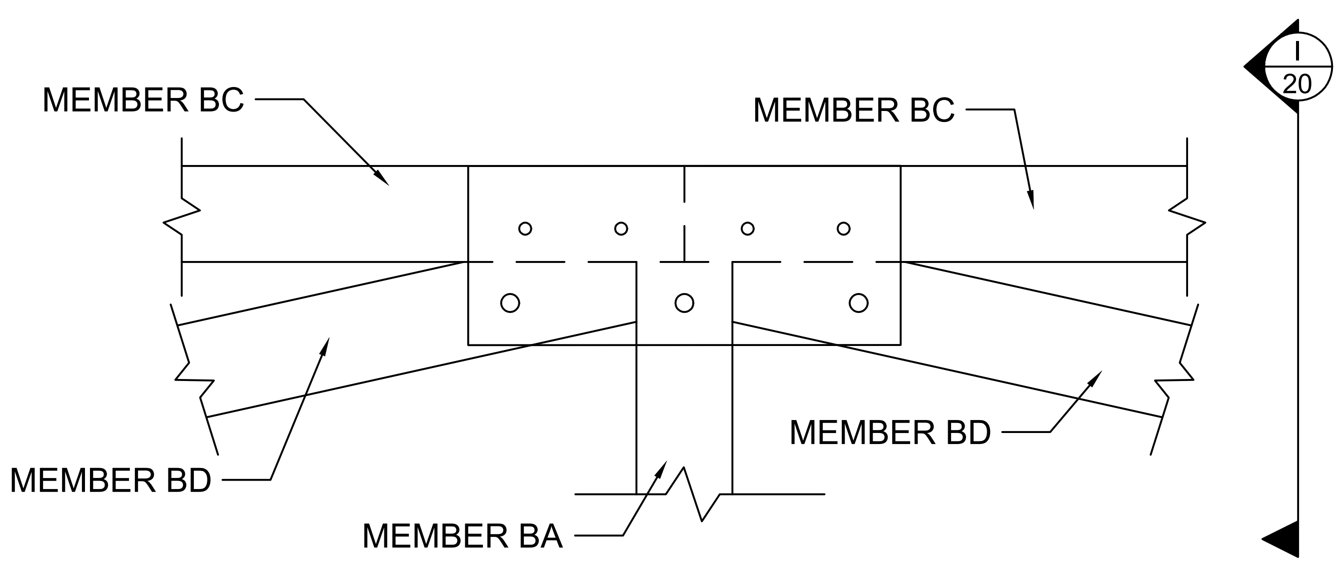

PROFILE VIEW - CONNECTION B
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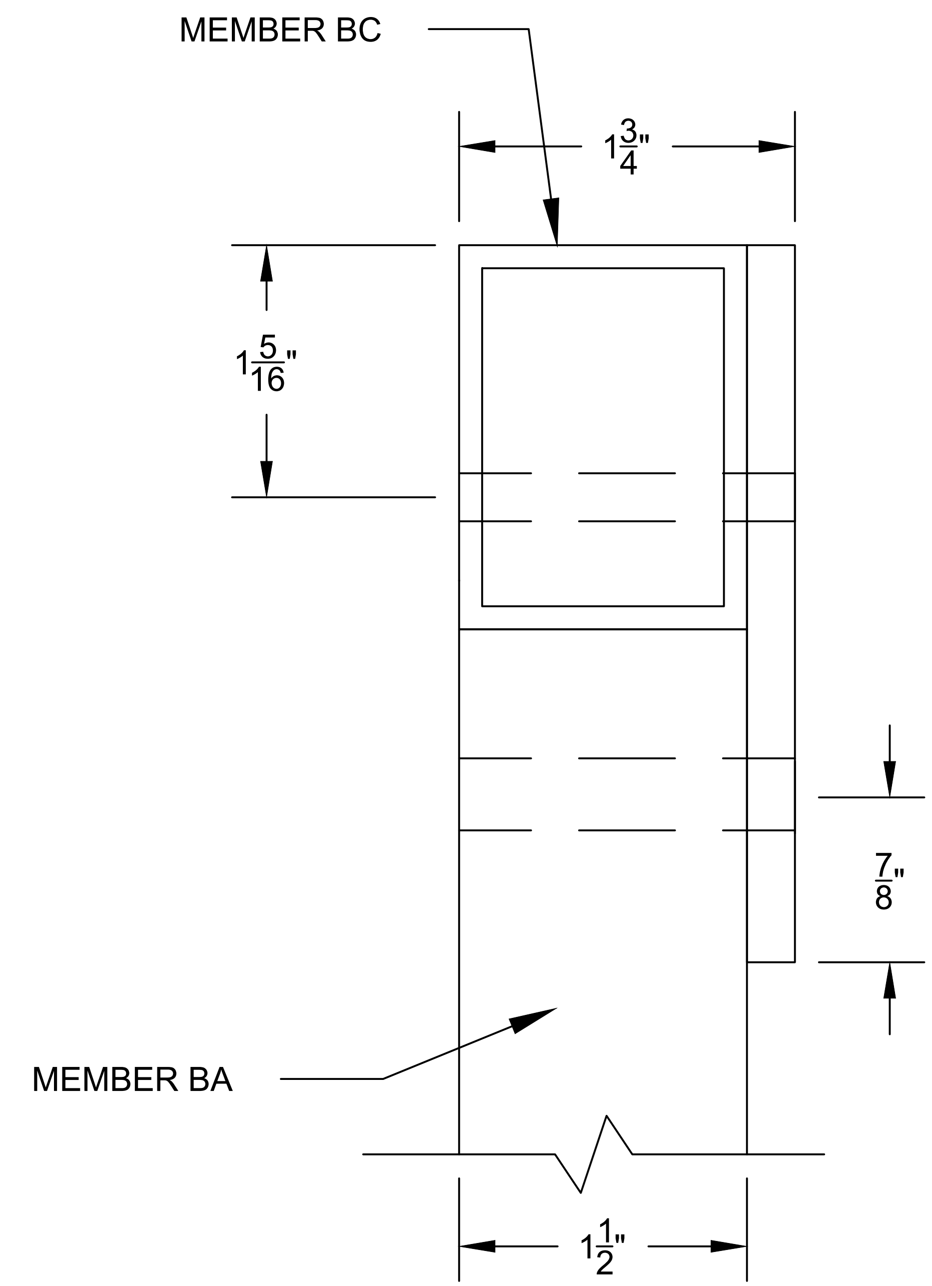
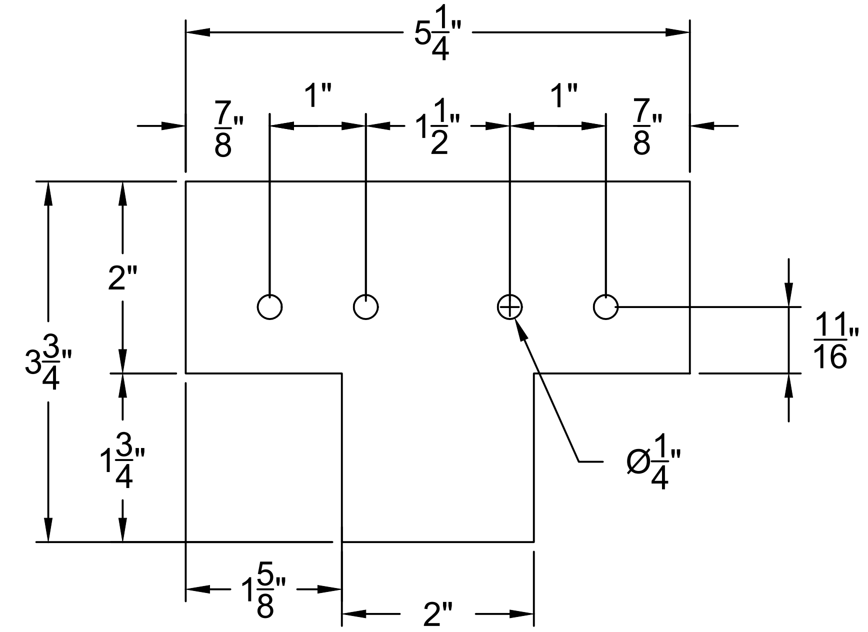


DETAIL I
SCALE: 1" = 1'

Beyond Bridges

PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03	
CONNECTION DETAIL B	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 20	OF: 34
		DATE: MARCH 2026		



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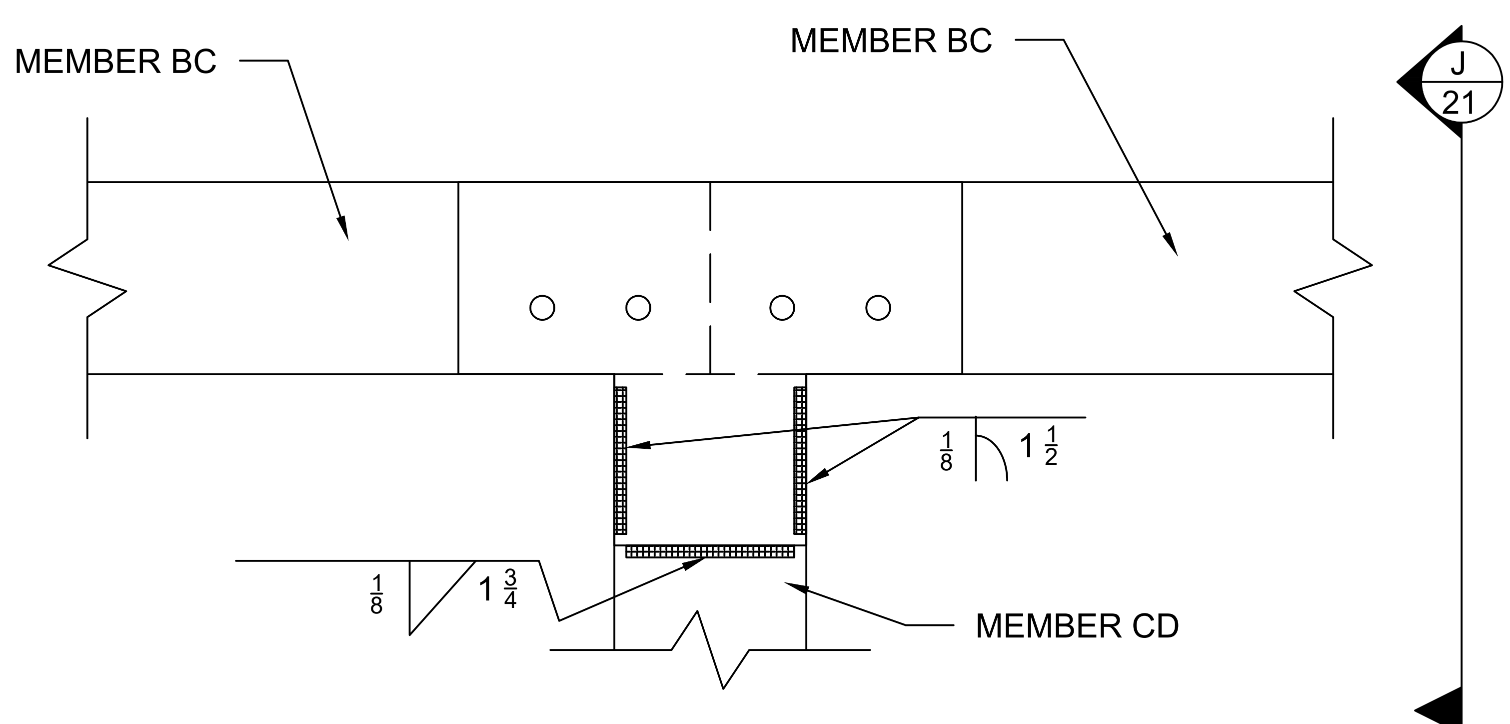

PROFILE VIEW - CONNECTION C
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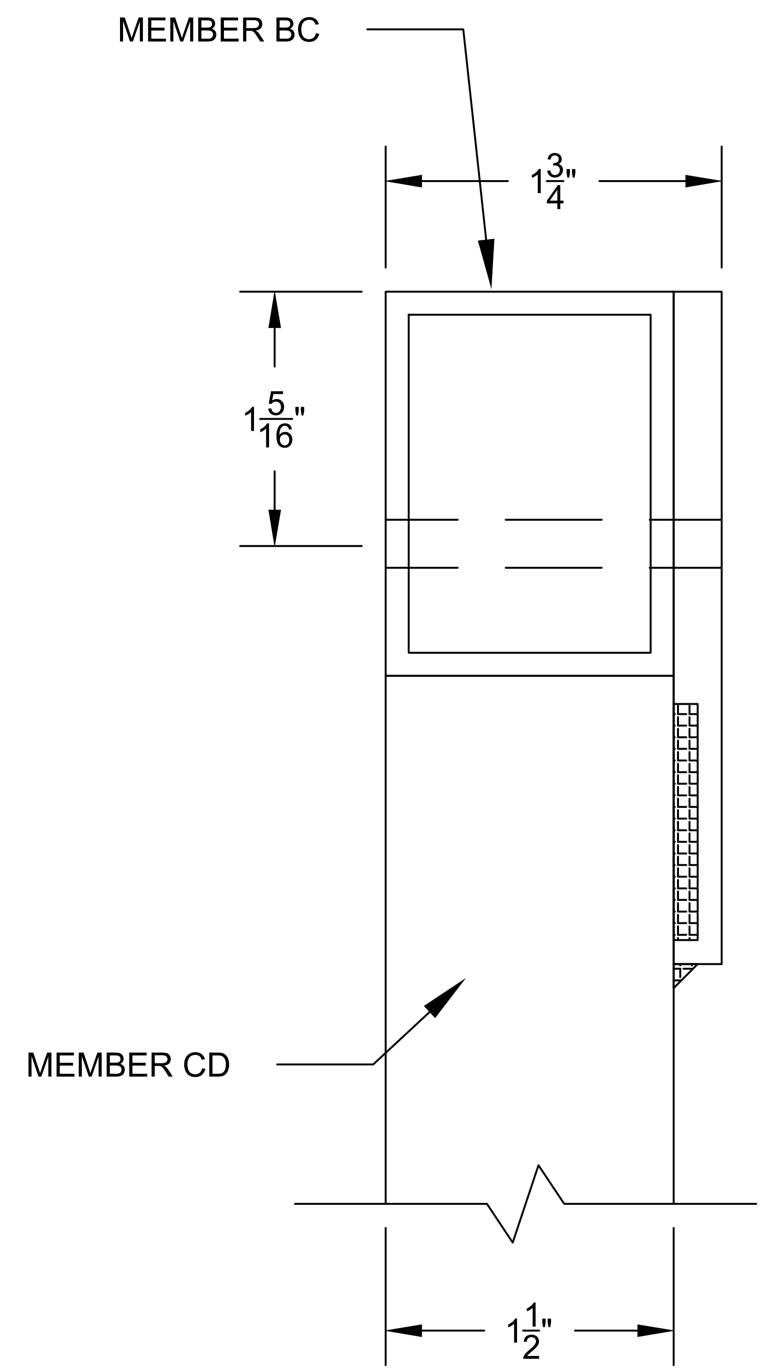

DETAIL J
SCALE: 1" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03
	CONNECTION DETAIL C		CHECKED BY: KJ, KF, & IK	PAGE: 21
		SCALE: N/A	DATE: MARCH 2026	OF: 34

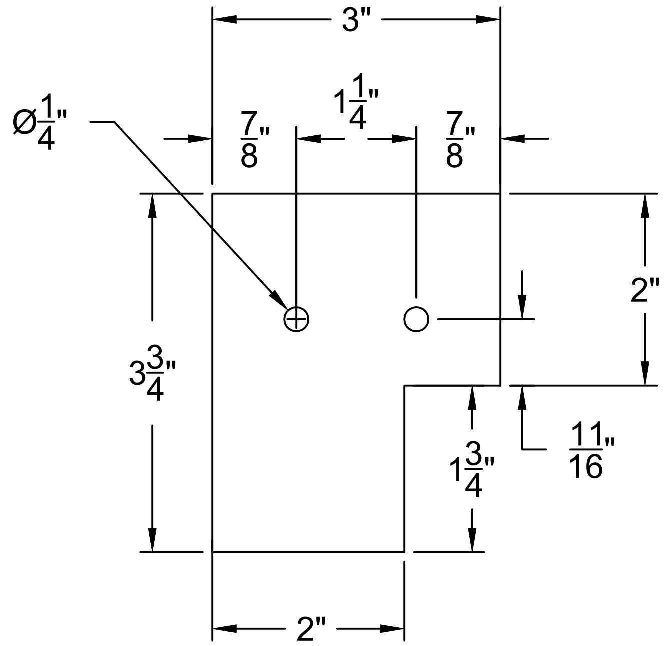

PLATE DIMENSION - CONNECTION 0.5C

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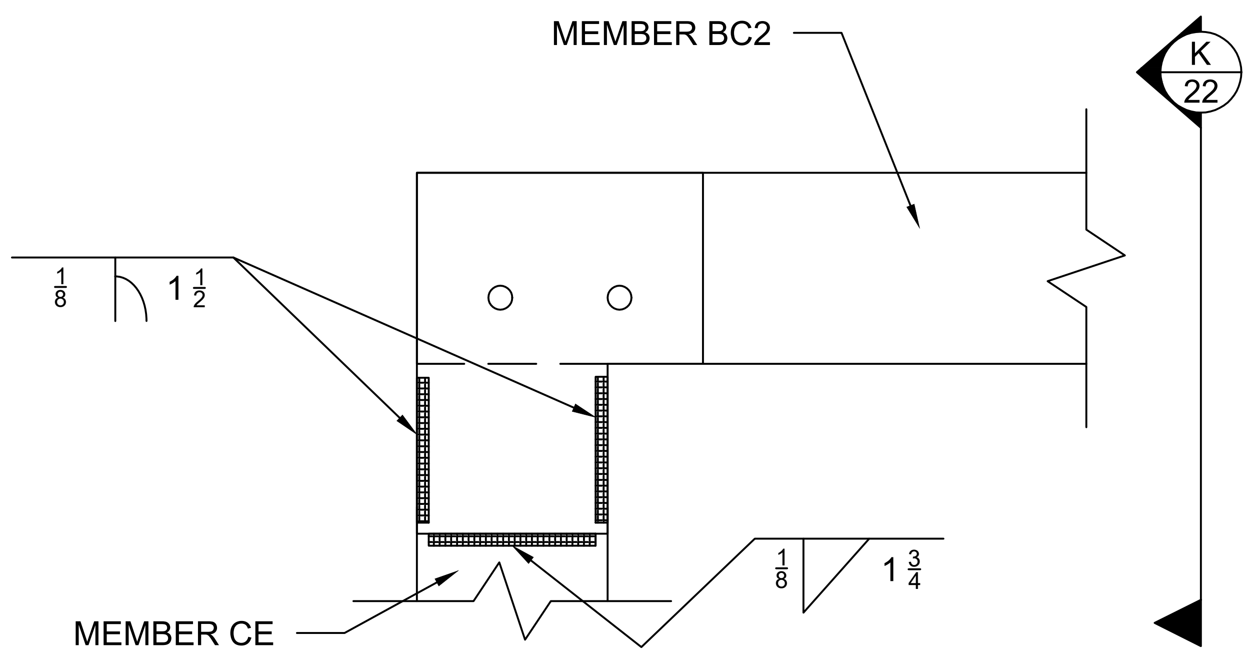

PROFILE VIEW - CONNECTION 0.5C

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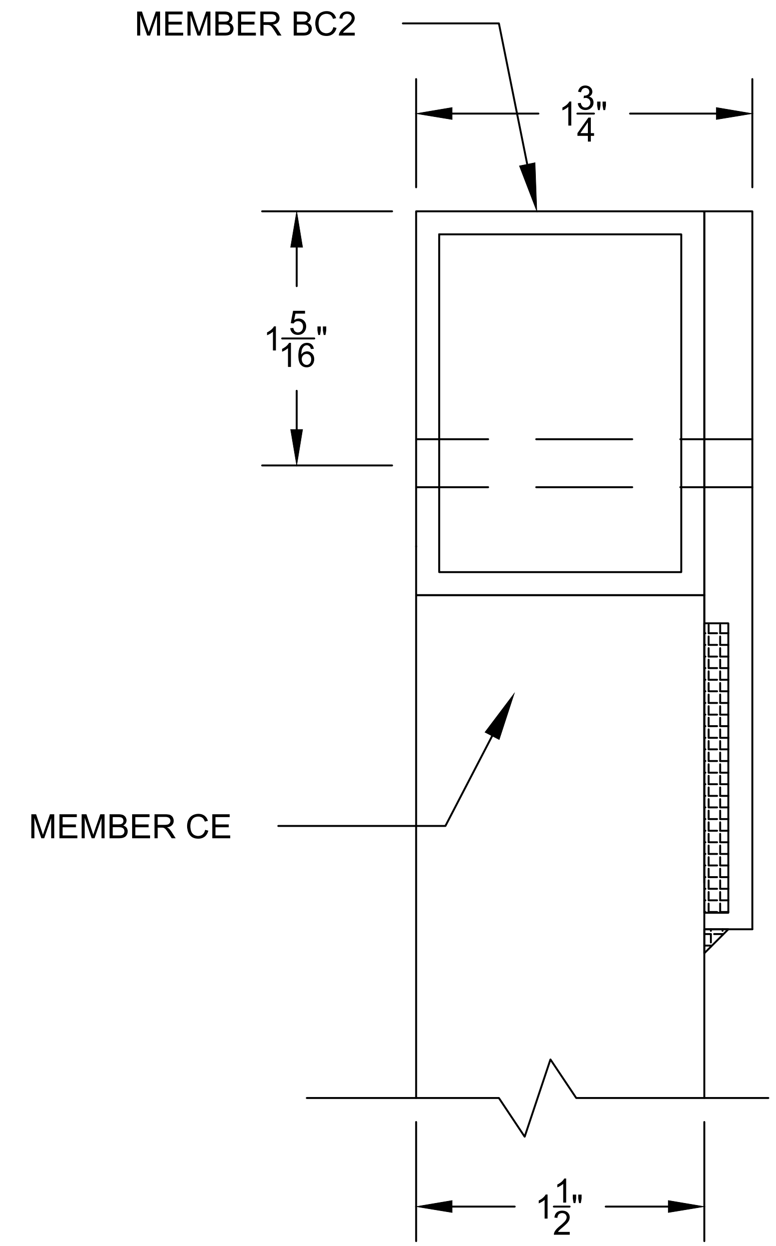


DETAIL K

SCALE: 1" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03	
	CONNECTION DETAIL 0.5C	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 22	OF: 34
			DATE: MARCH 2026		

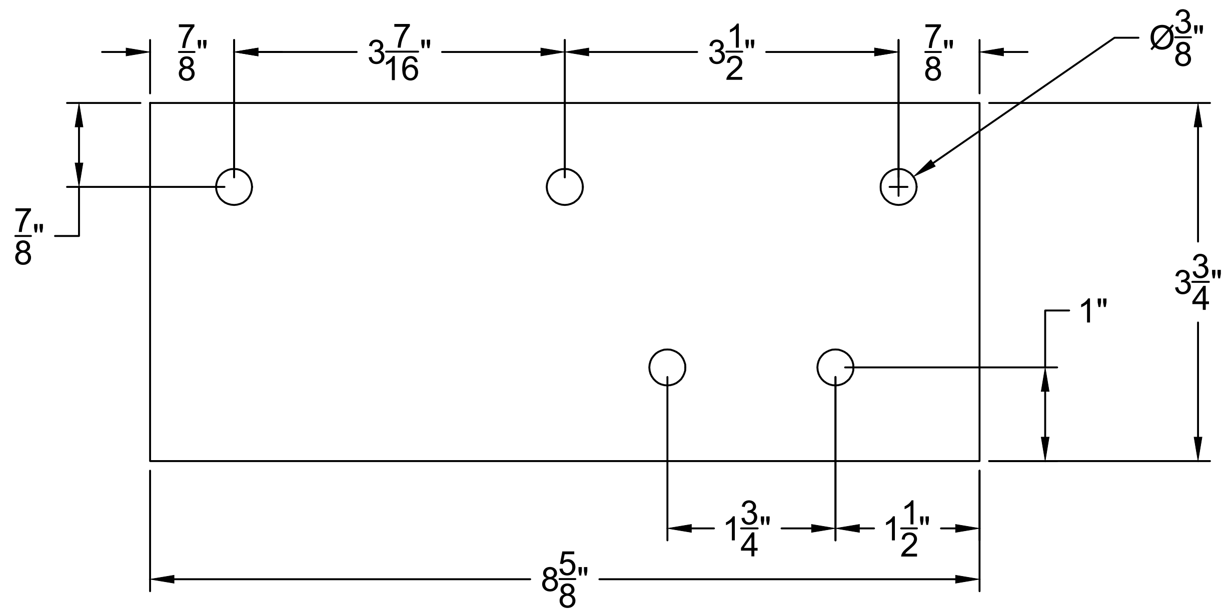
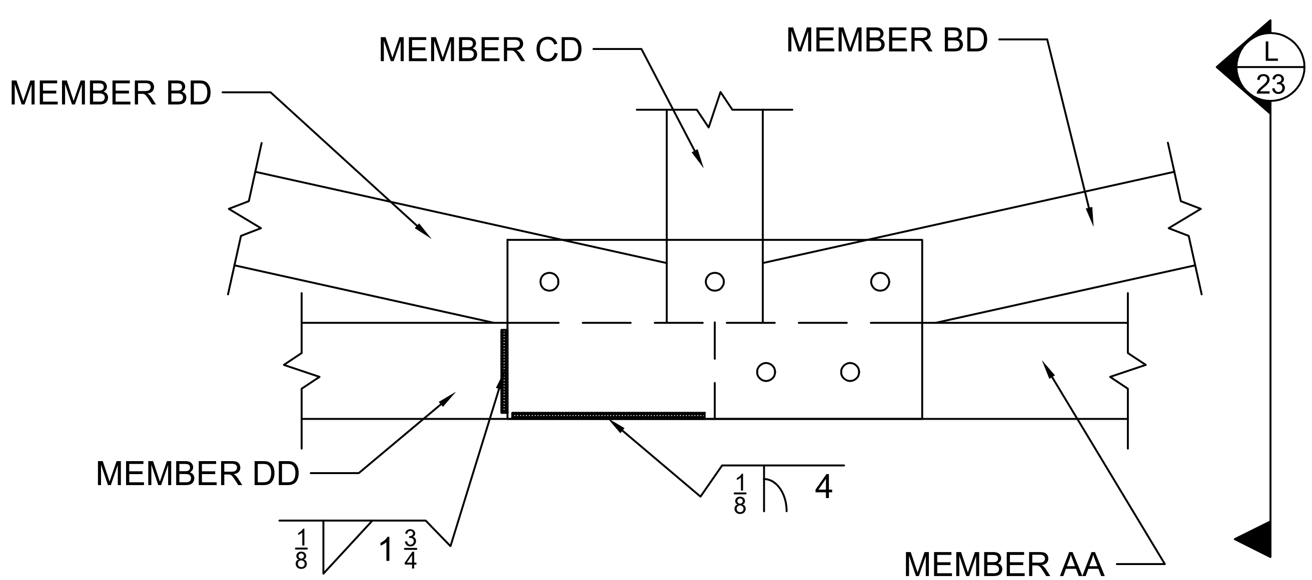


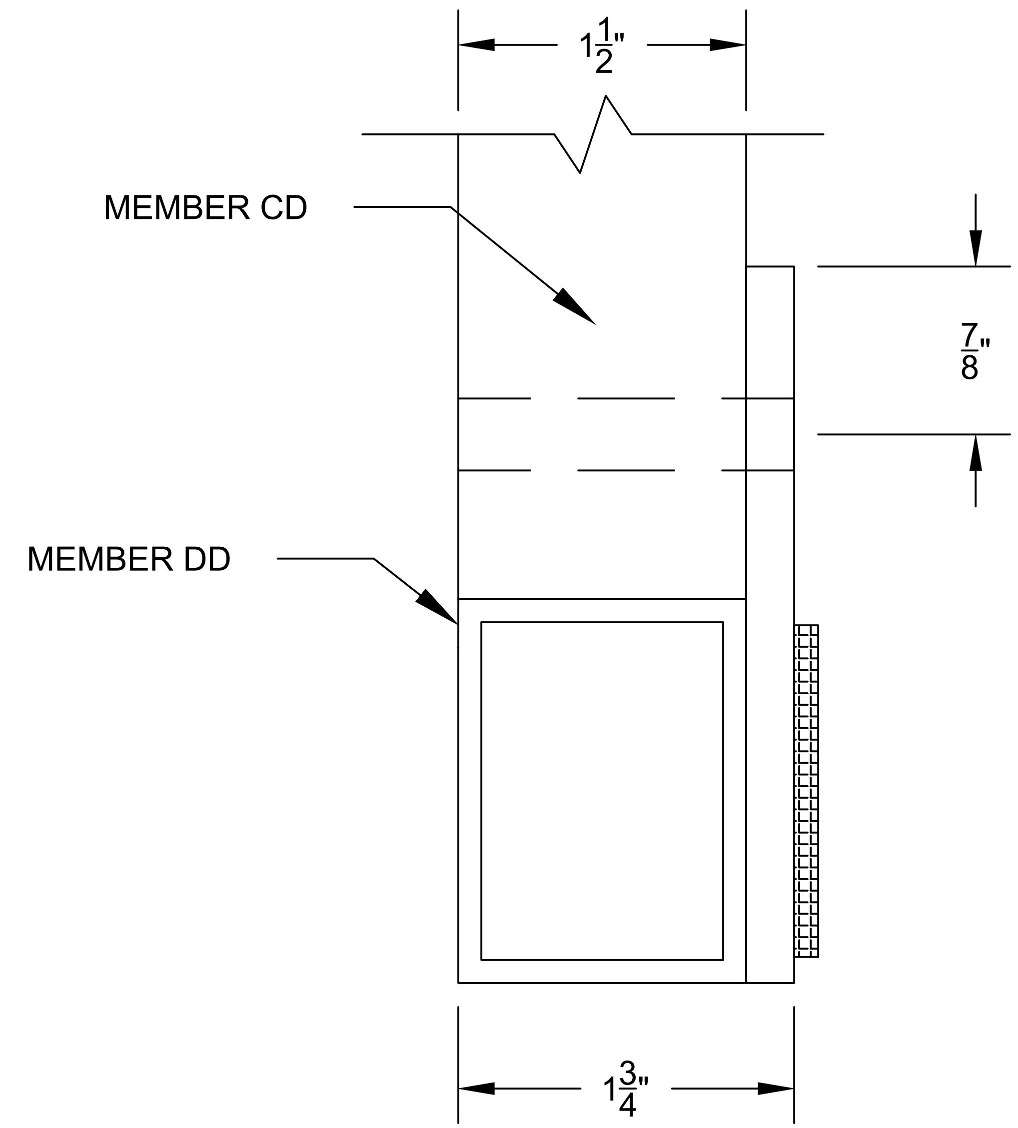
PLATE DIMENSION - CONNECTION D

SCALE: 6" = 1'



PROFILE VIEW - CONNECTION D

SCALE: 3" = 1'



DETAIL L

SCALE: 1" = 1'

Beyond Bridges

PROJECT: NAU 2025-2026 STEEL BRIDGE

CONNECTION DETAIL D

SCALE: N/A

BY: MA

CHECKED BY: KJ, KF, & IK

DATE: MARCH 2026

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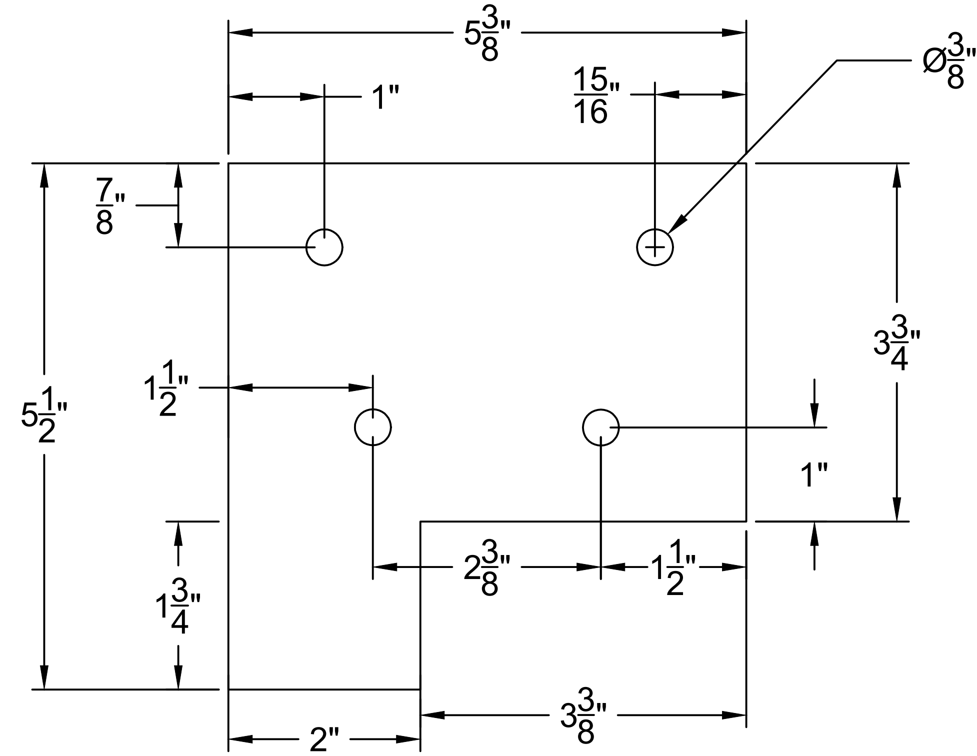
PAGE: 23

OF: 34



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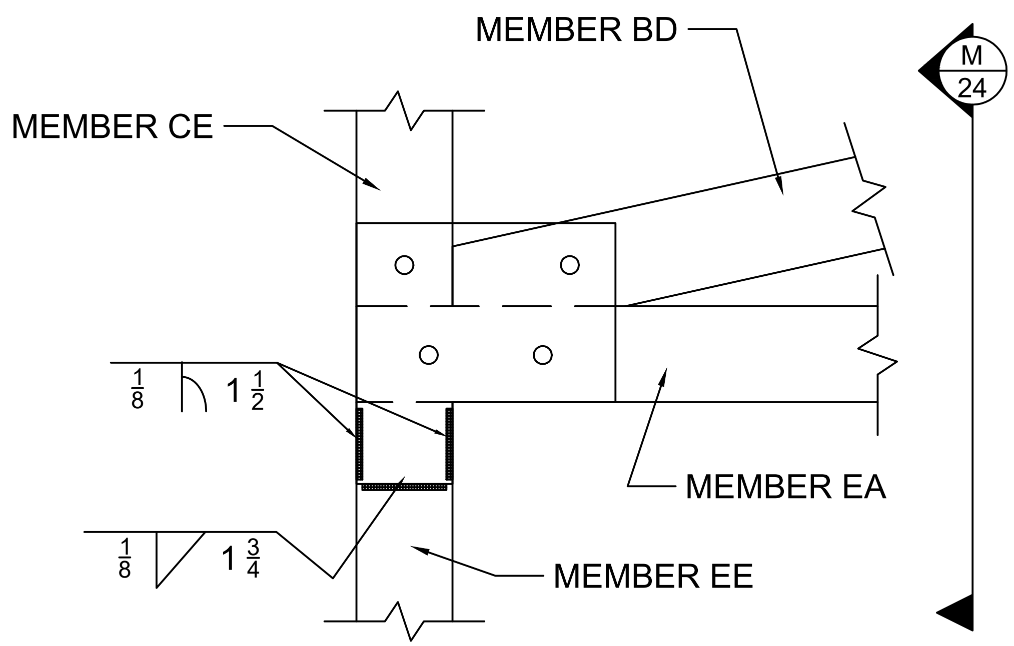
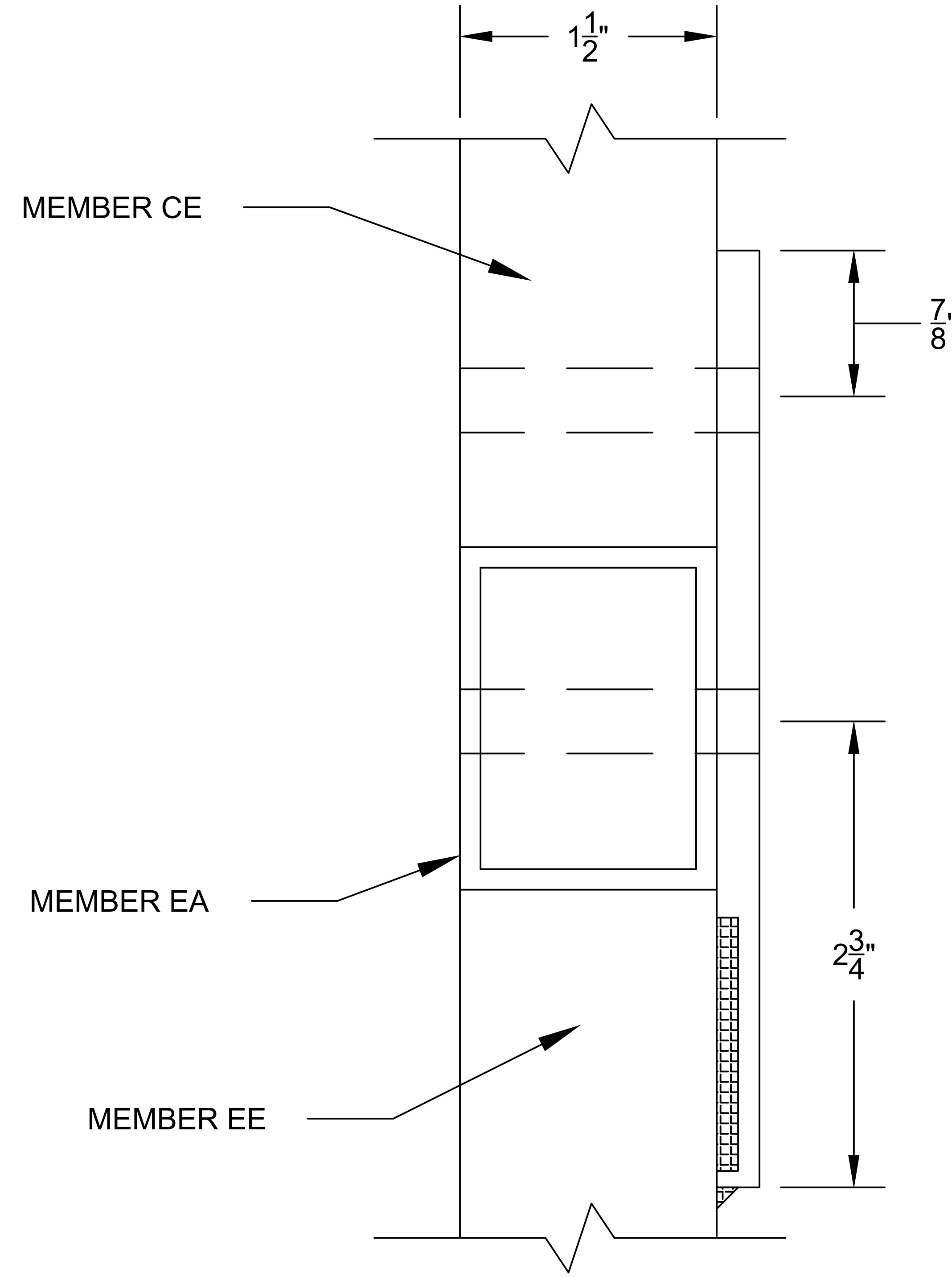


PROFILE VIEW - CONNECTION E
SCALE: 3" = 1'



DETAIL M
SCALE: 1" = 1'

PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03
CONNECTION DETAIL E	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 24
		DATE: MARCH 2026	OF: 34

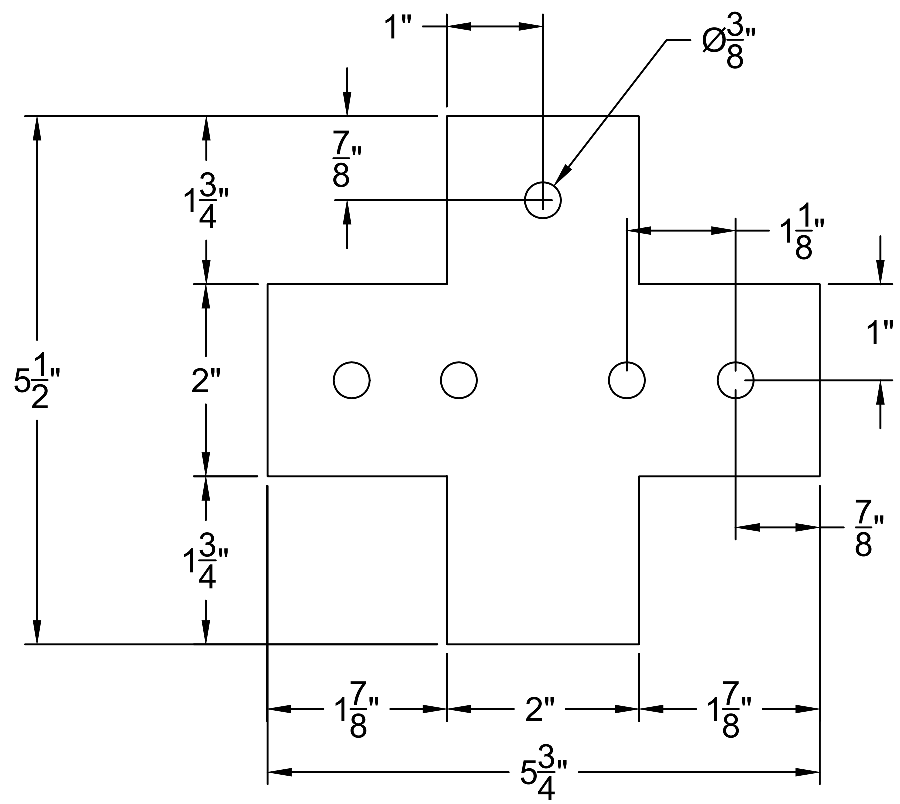


PLATE DIMENSION- CONNECTION F

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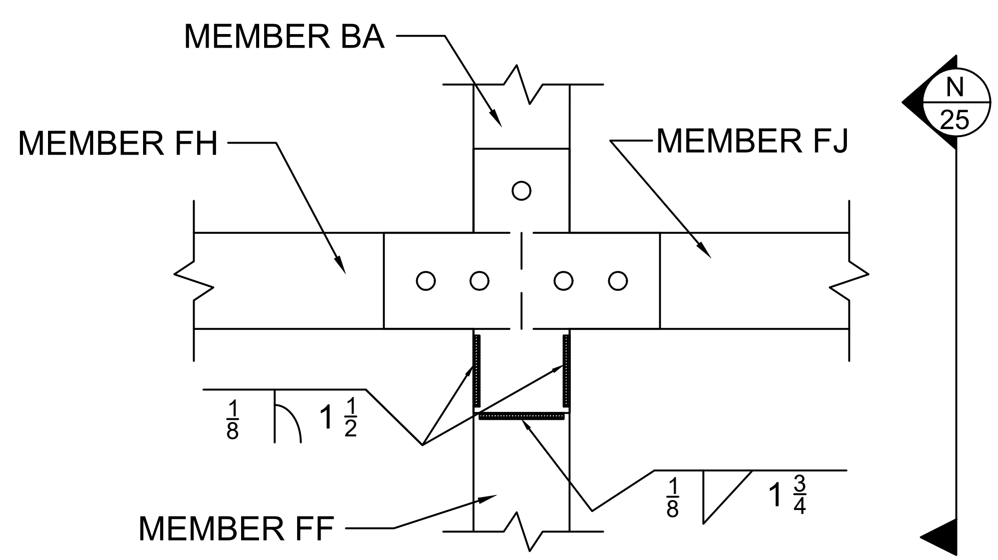
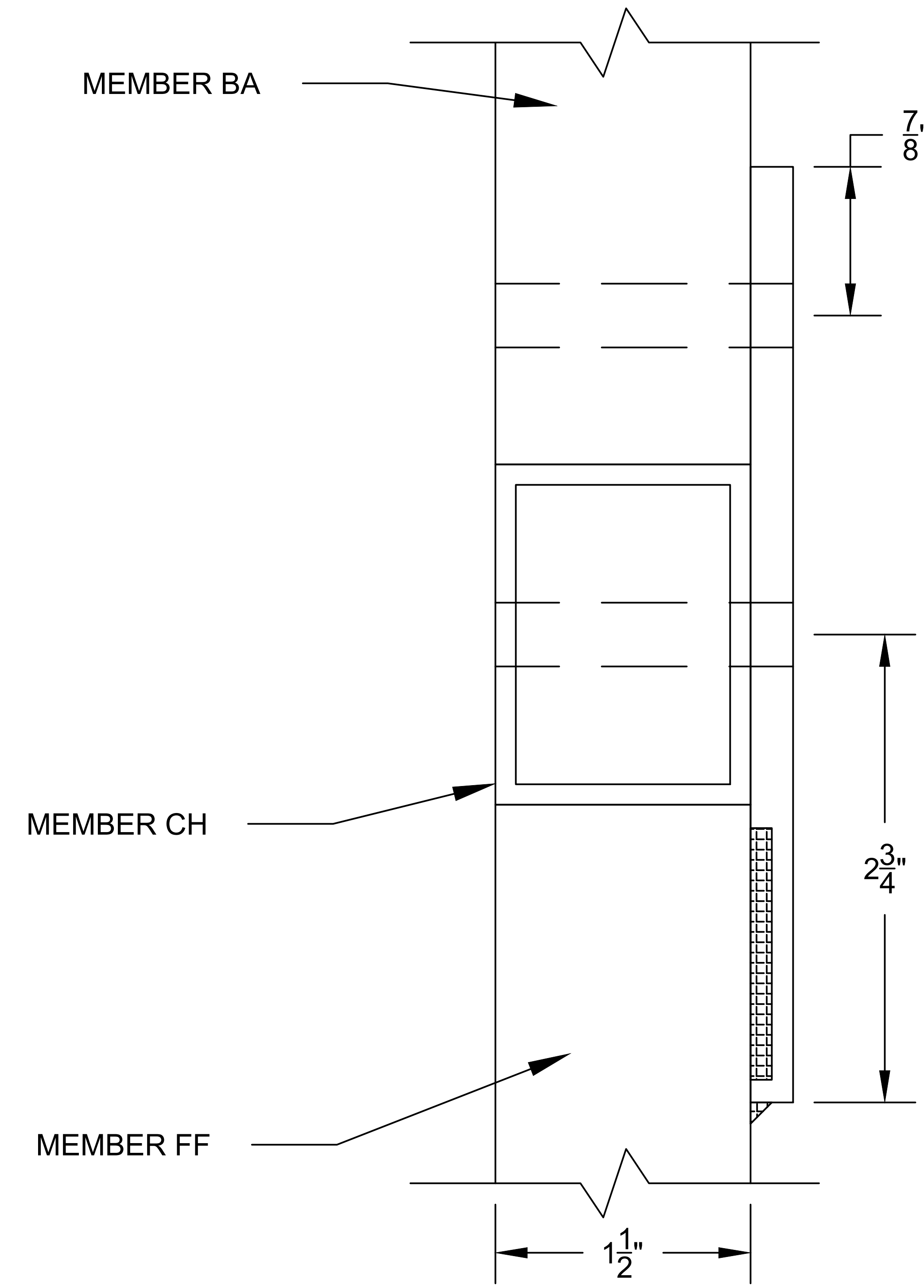


PROFILE VIEW - CONNECTION F

SCALE: 3" = 1'



DETAIL N

SCALE: 1" = 1'

PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: KJ	REVISION: 03	
CONNECTION DETAIL F	SCALE: N/A	CHECKED BY: MA, KF, & IK	PAGE: 25	OF: 34
		DATE: MARCH 2026		

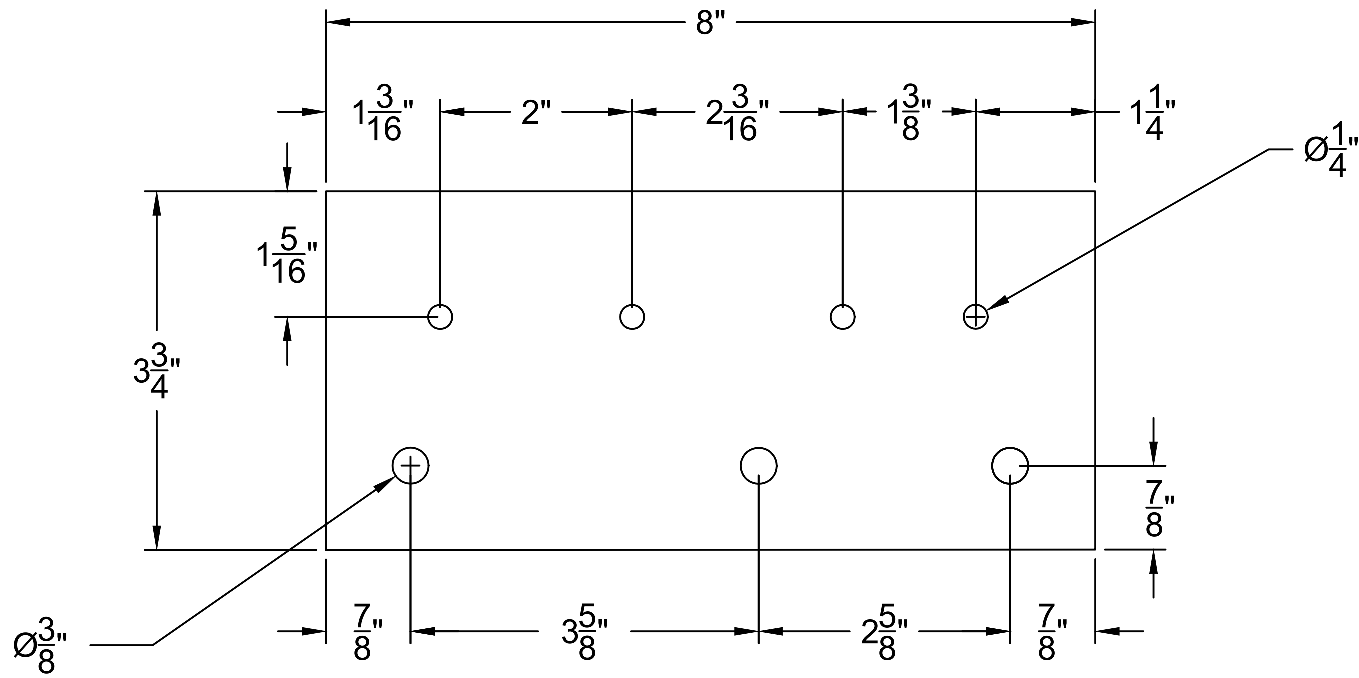


PLATE DIMENSION - CONNECTION G
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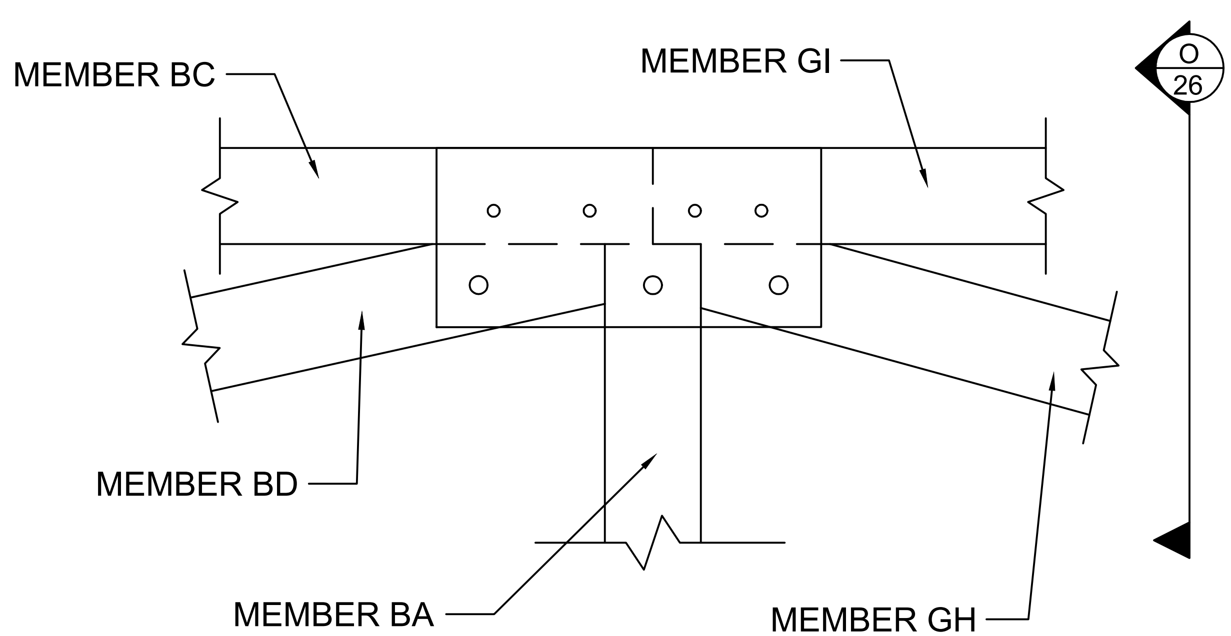
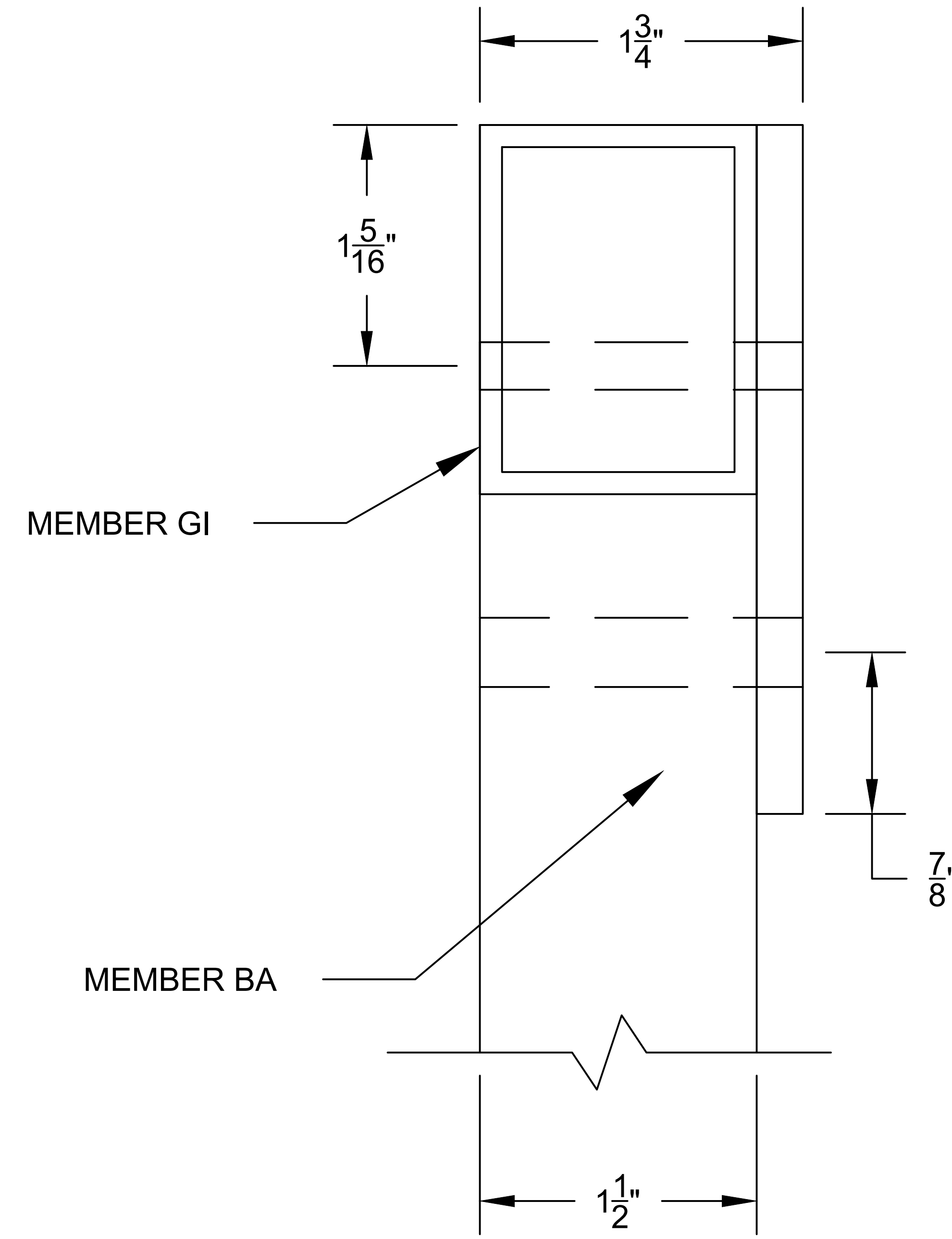
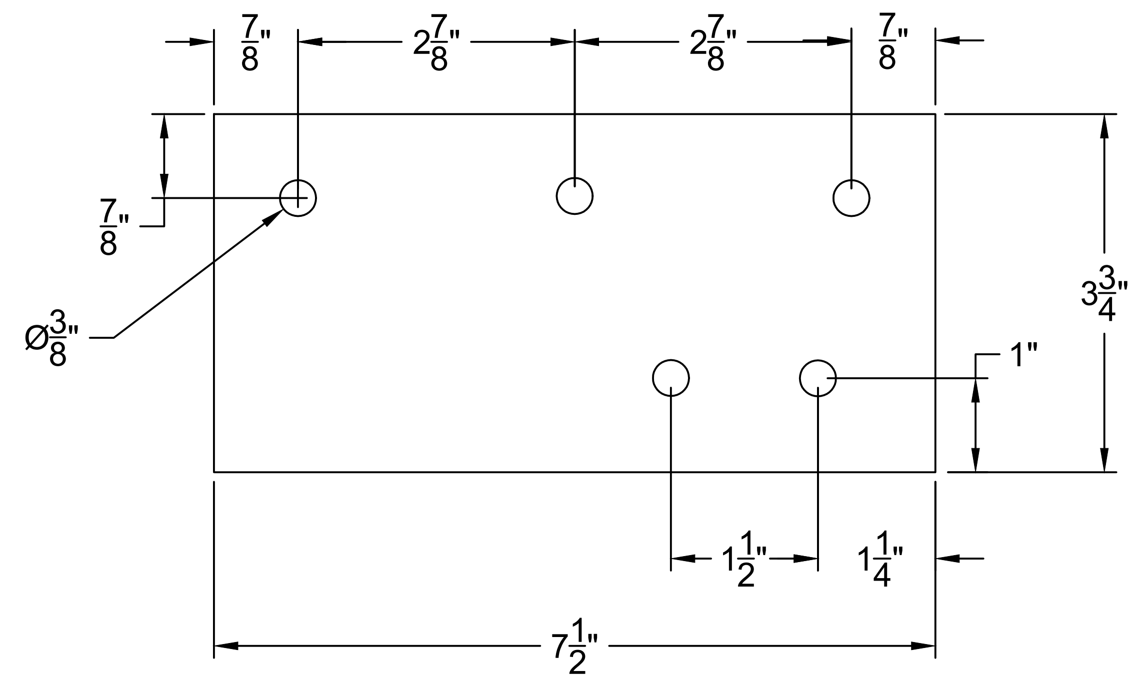


PROFILE VIEW- CONNECTION G
SCALE: 3" = 1'



DETAIL O
SCALE: 1" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: KJ	REVISION: 03	
	CONNECTION DETAIL G		CHECKED BY: MA, KF, & IK	PAGE: 26	OF: 34
		SCALE: N/A	DATE: MARCH 2026		



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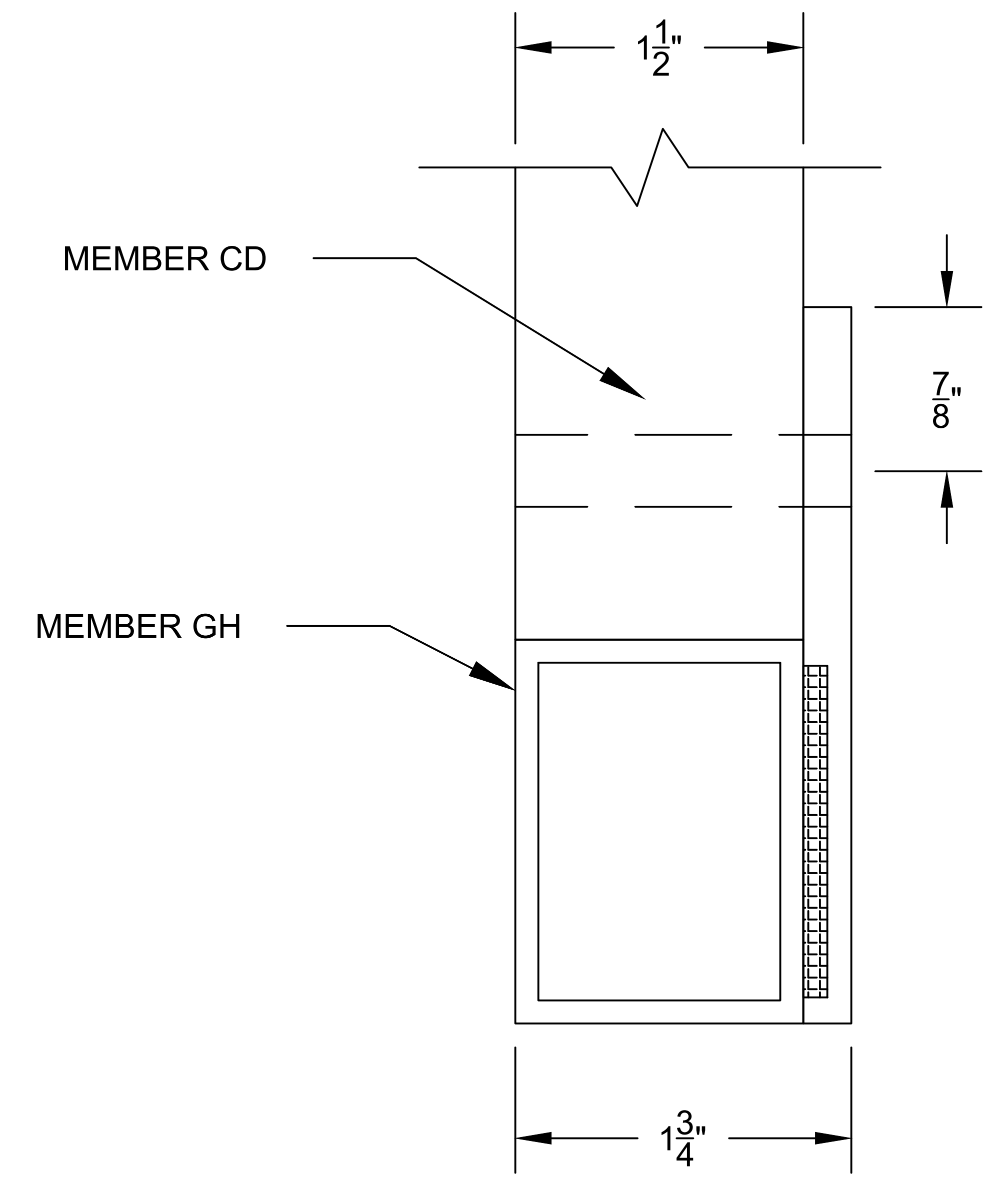
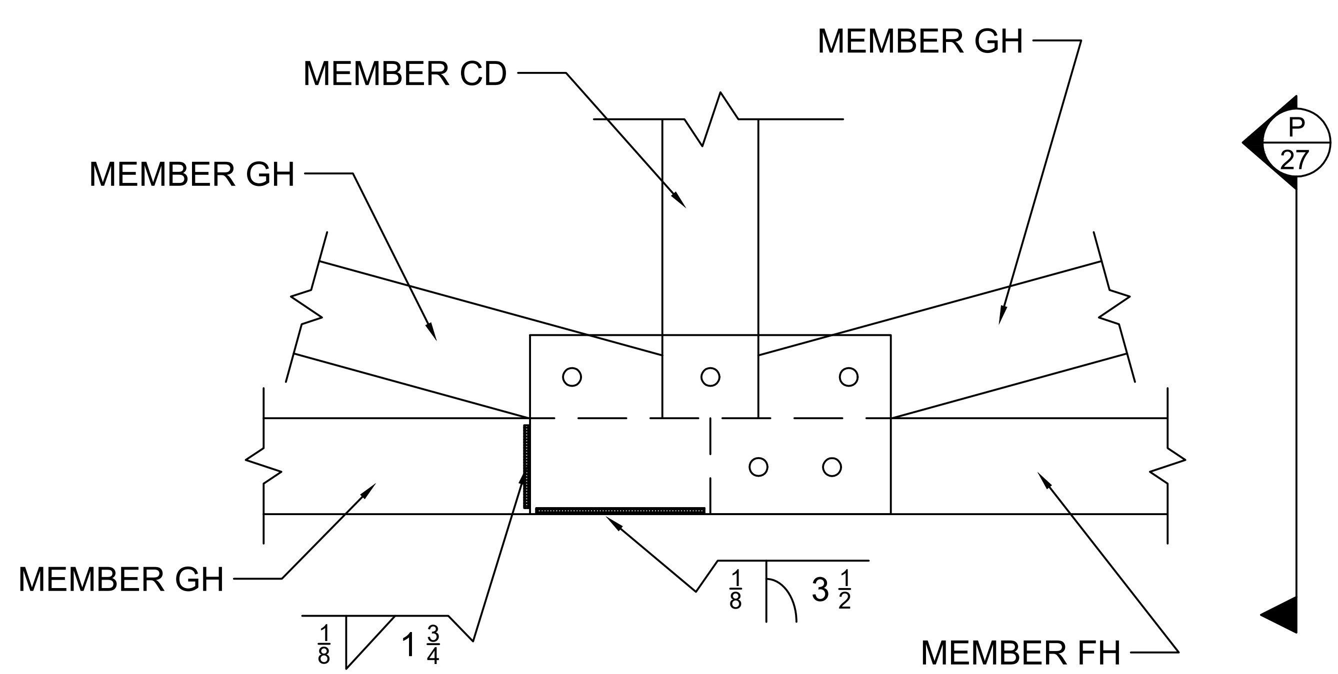


DETAIL P
SCALE: 1" = 1'



PROFILE VIEW - CONNECTION H
SCALE: 3" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: KJ	REVISION: 03
	CONNECTION DETAIL H	SCALE: N/A	CHECKED BY: MA, KF, & IK	PAGE: 27
			DATE: MARCH 2026	OF: 34

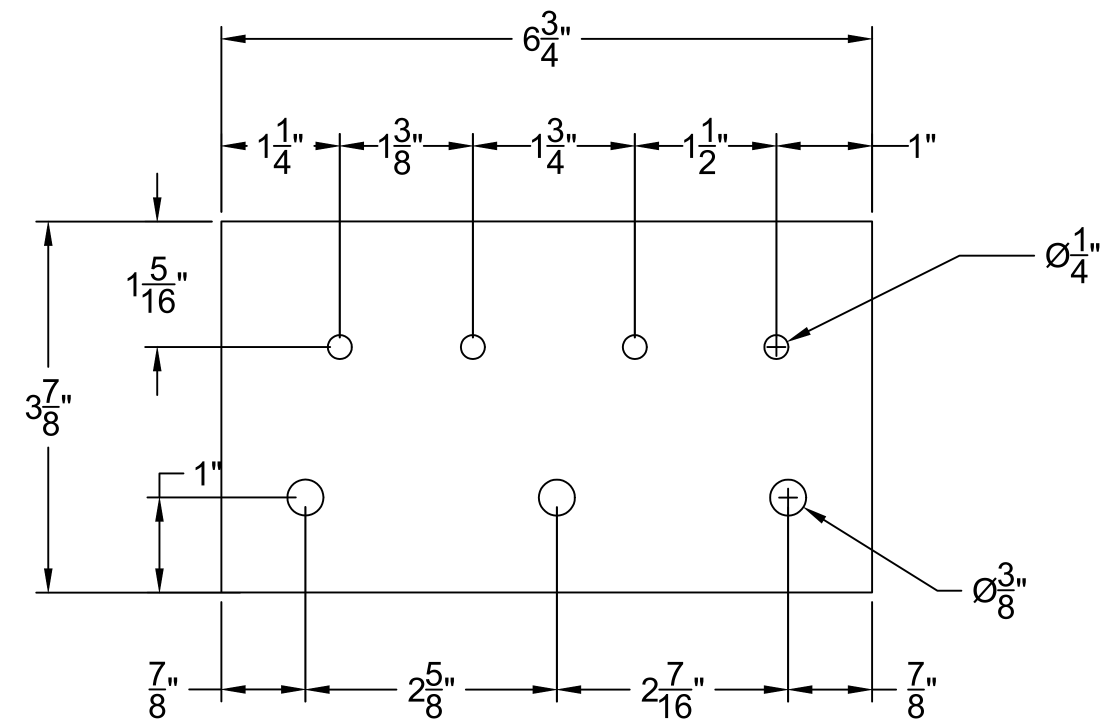
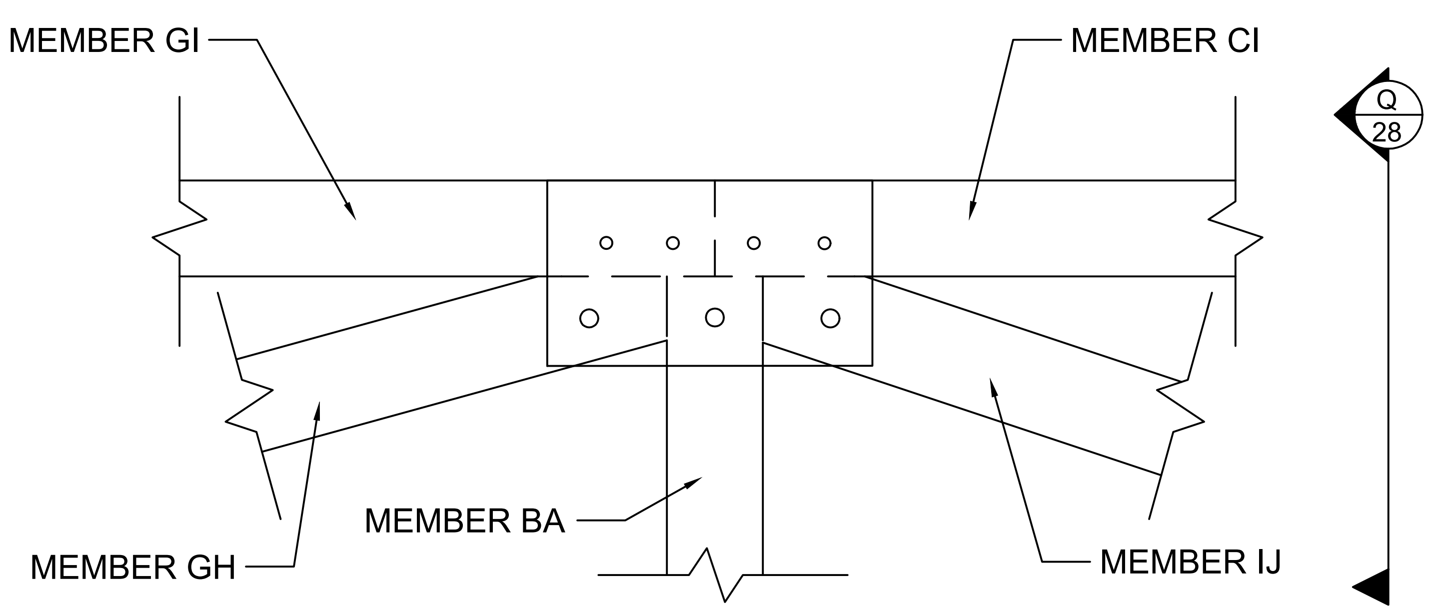
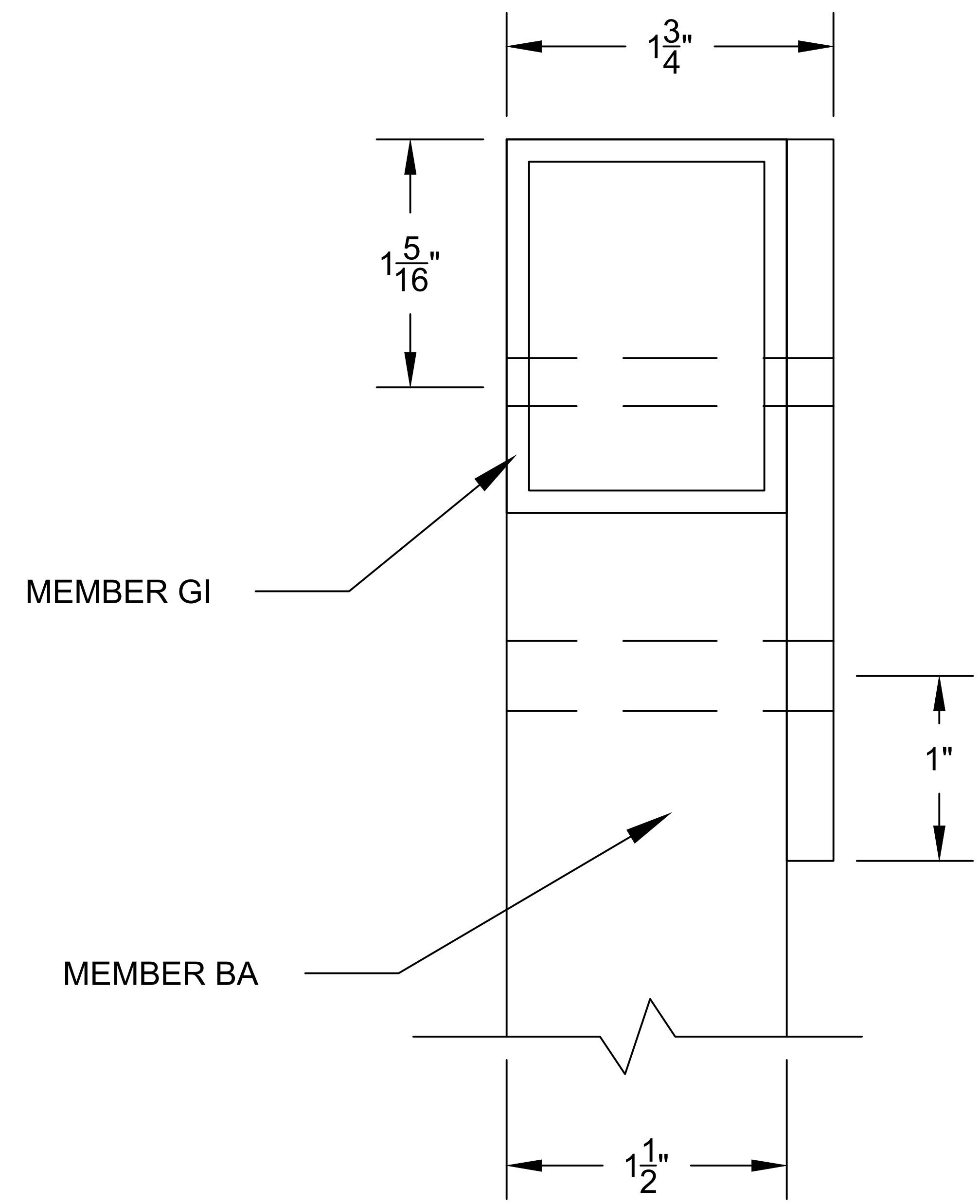


PLATE DIMENSION - CONNECTION I
SCALE: 6" = 1'



PROFILE VIEW - CONNECTION I
SCALE: 3" = 1'



DETAIL Q
SCALE: 1" = 1'

PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: KJ	REVISION: 03	
CONNECTION DETAIL I	SCALE: N/A	CHECKED BY: MA, KF, & IK	PAGE: 28	OF: 34
		DATE: MARCH 2026		

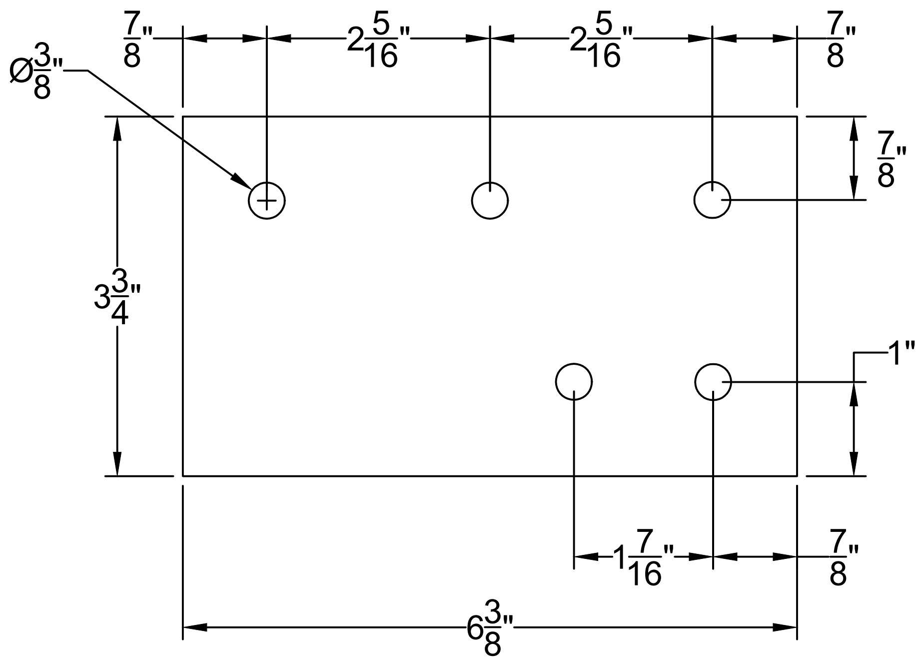


PLATE DIMENSION - CONNECTION J
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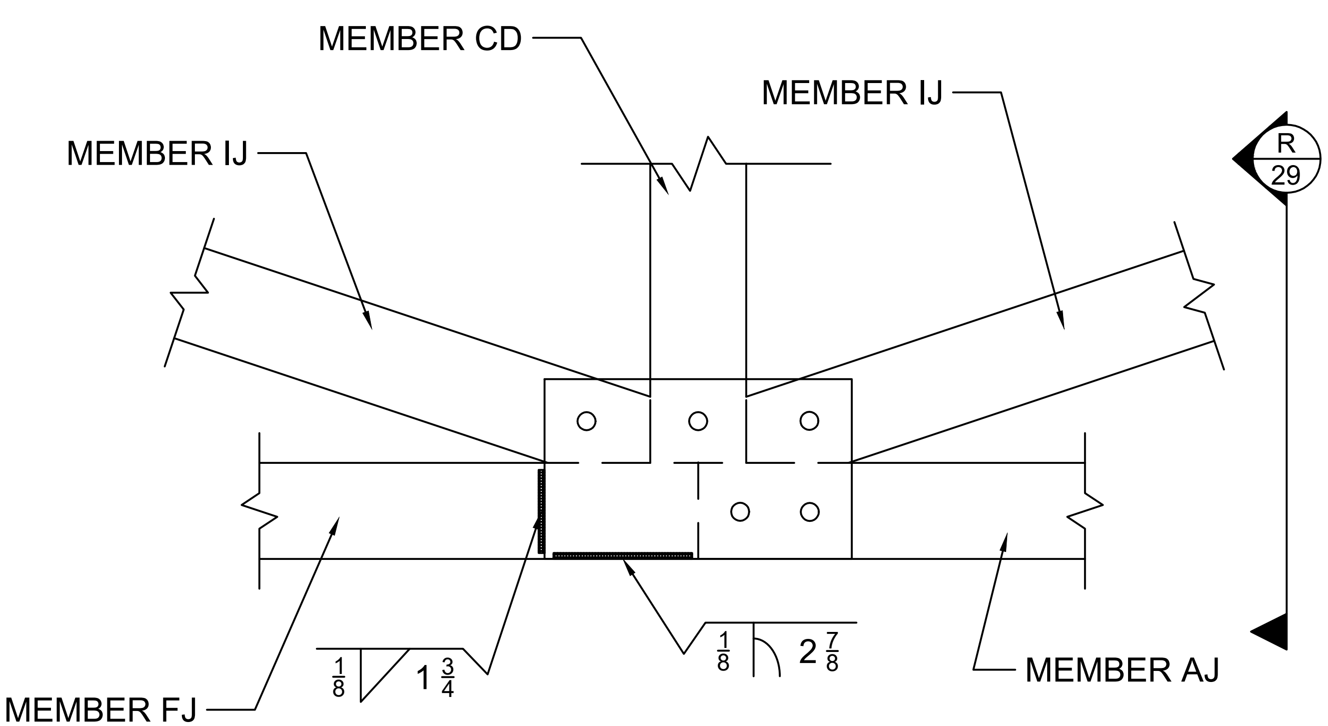

PROFILE VIEW - CONNECTION J
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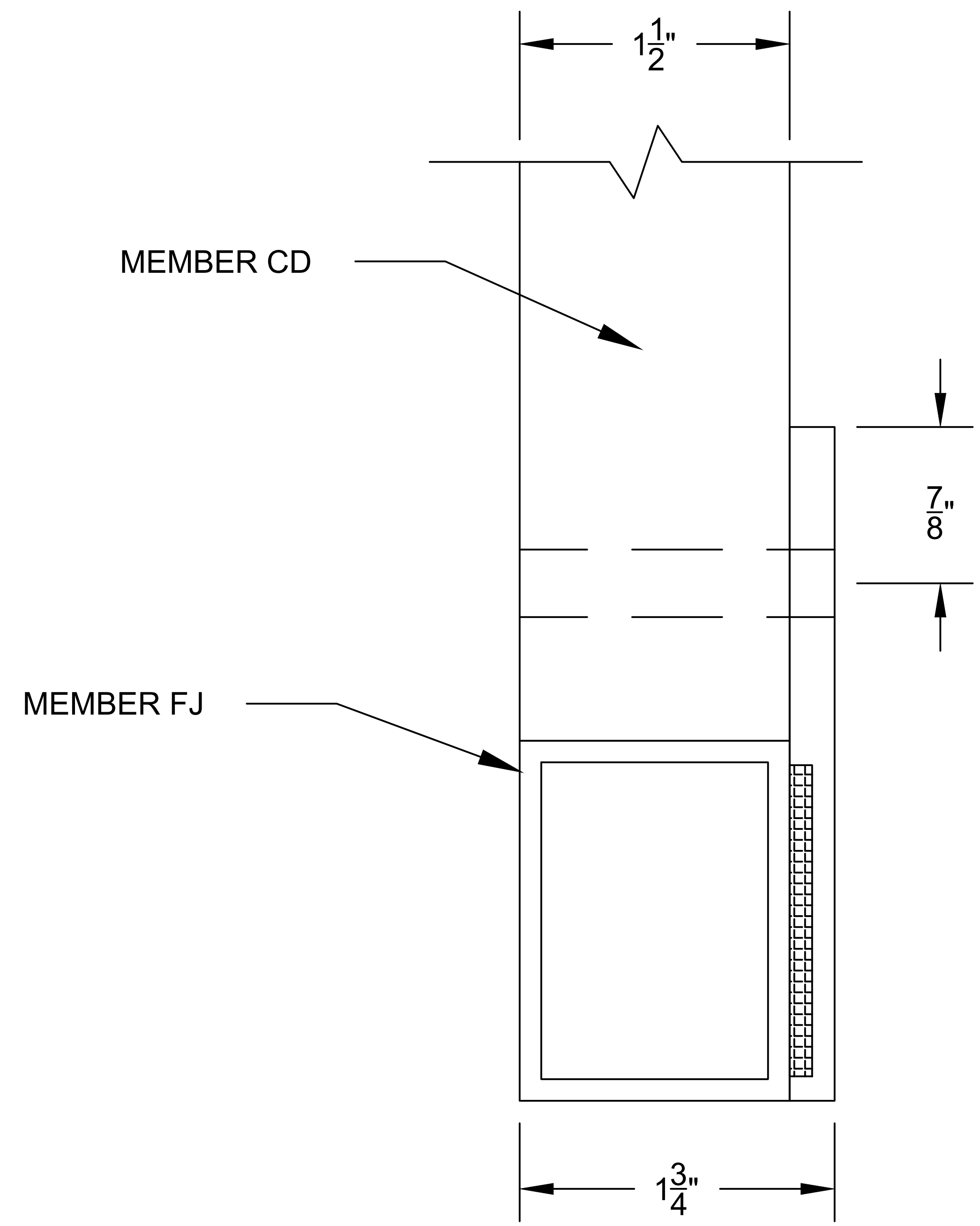


DETAIL R
SCALE: 1" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03
	CONNECTION DETAIL J		CHECKED BY: KJ, KF, & IK	PAGE: 29
		SCALE: N/A	DATE: MARCH 2026	OF: 34

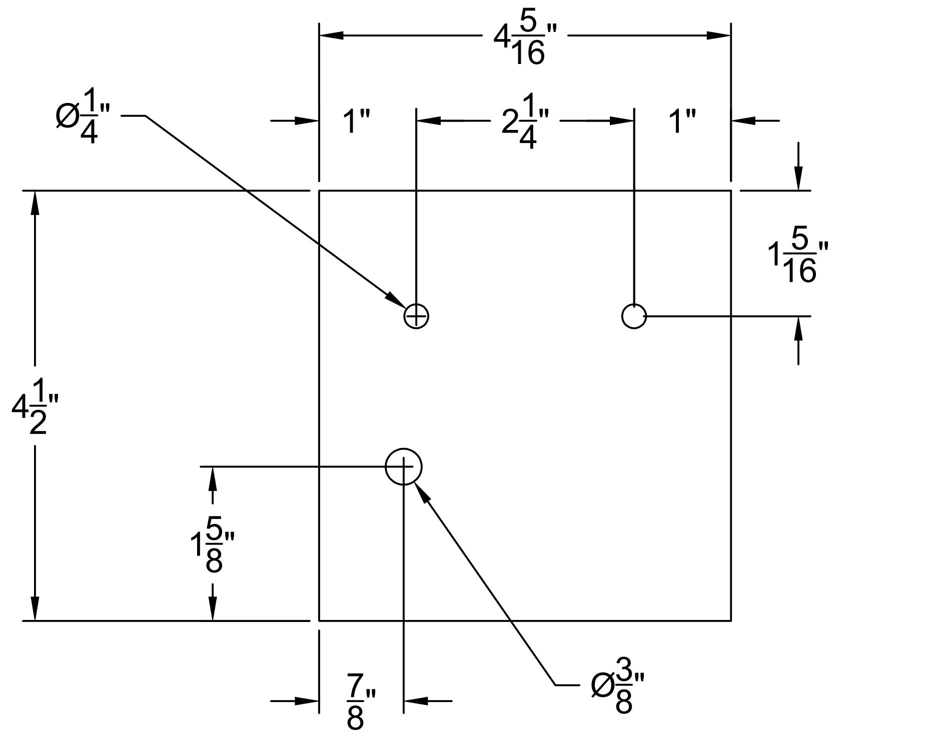
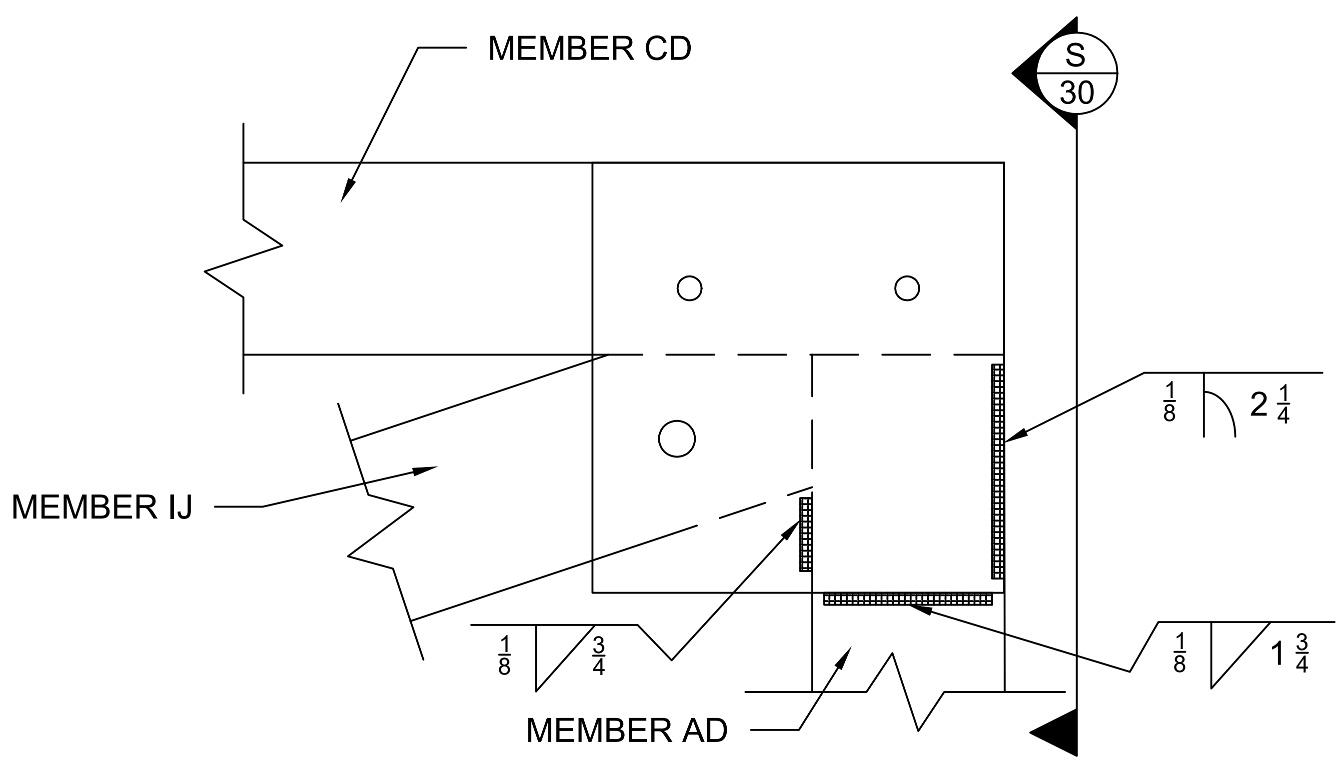


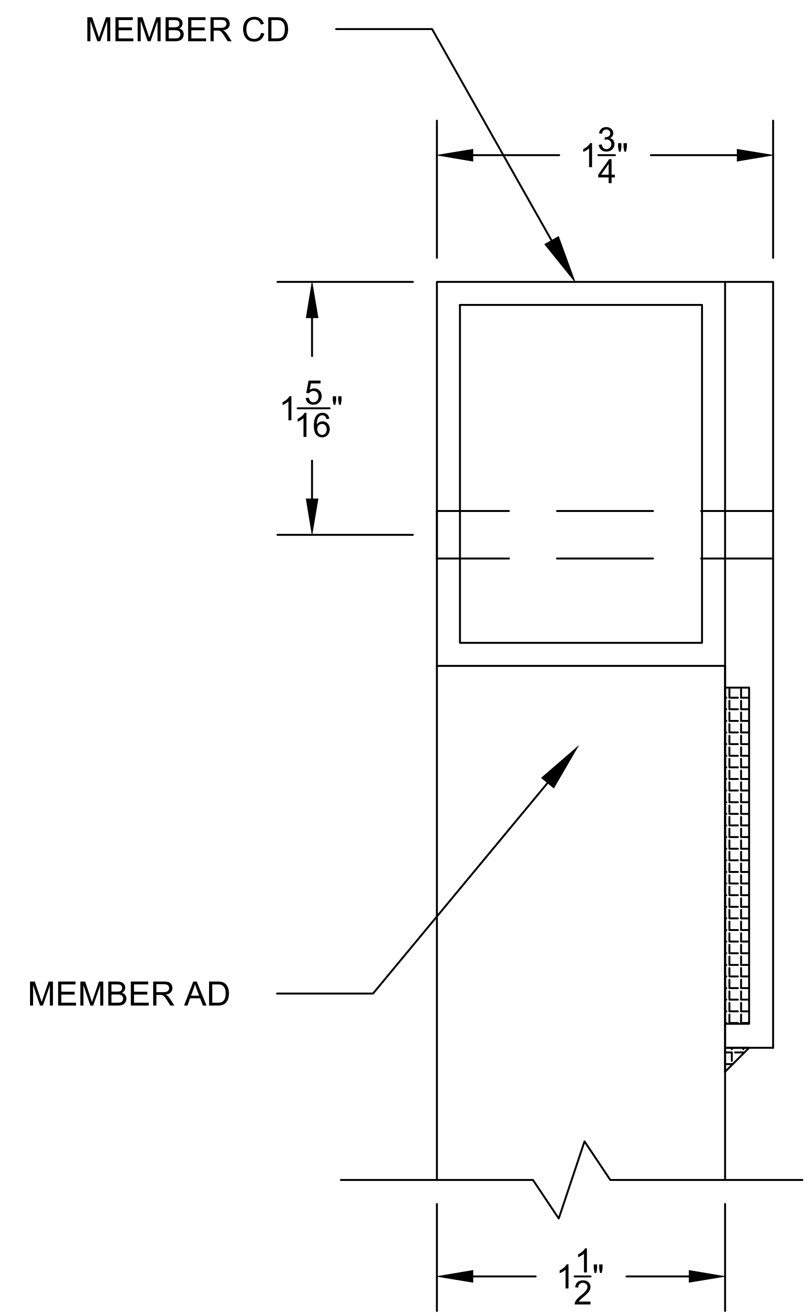
PLATE DIMENSION - CONNECTION K

SCALE: 6" = 1'



PROFILE VIEW - CONNECTION K

SCALE: 6" = 1'



DETAIL S

SCALE: 1" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03
	CONNECTION DETAIL K	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 30 / OF: 34
			DATE: MARCH 2026	

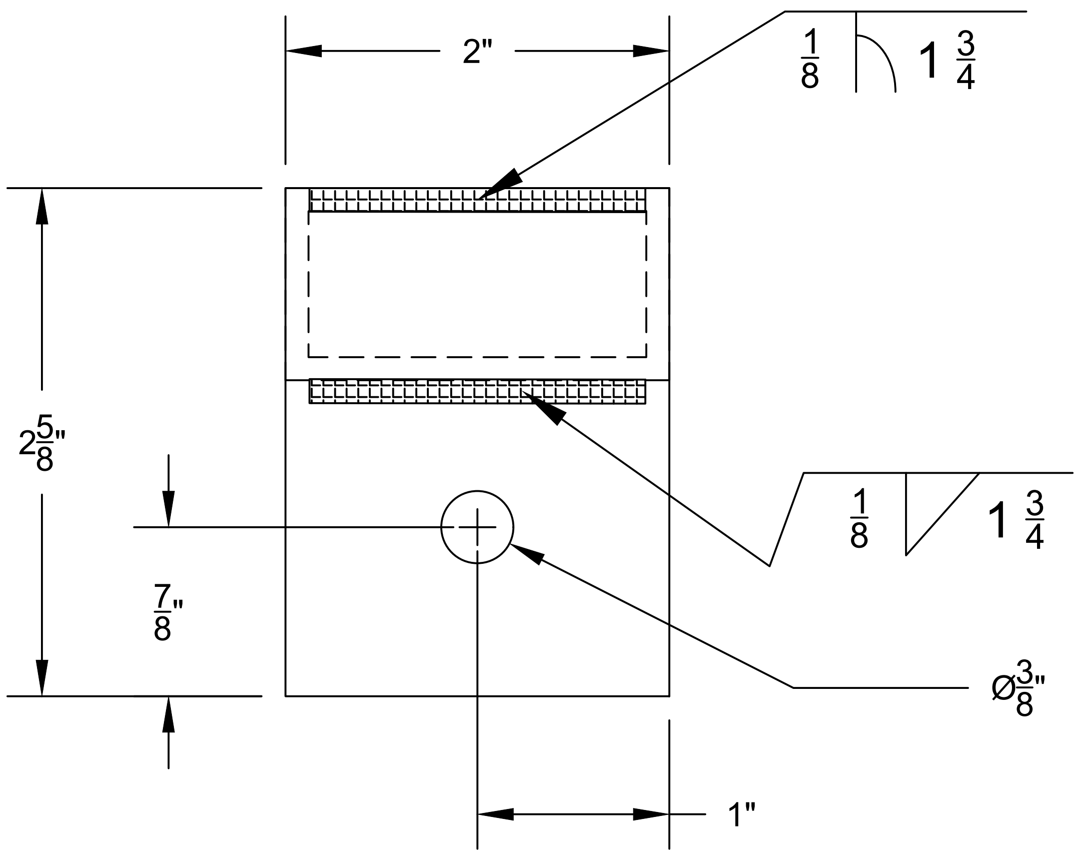
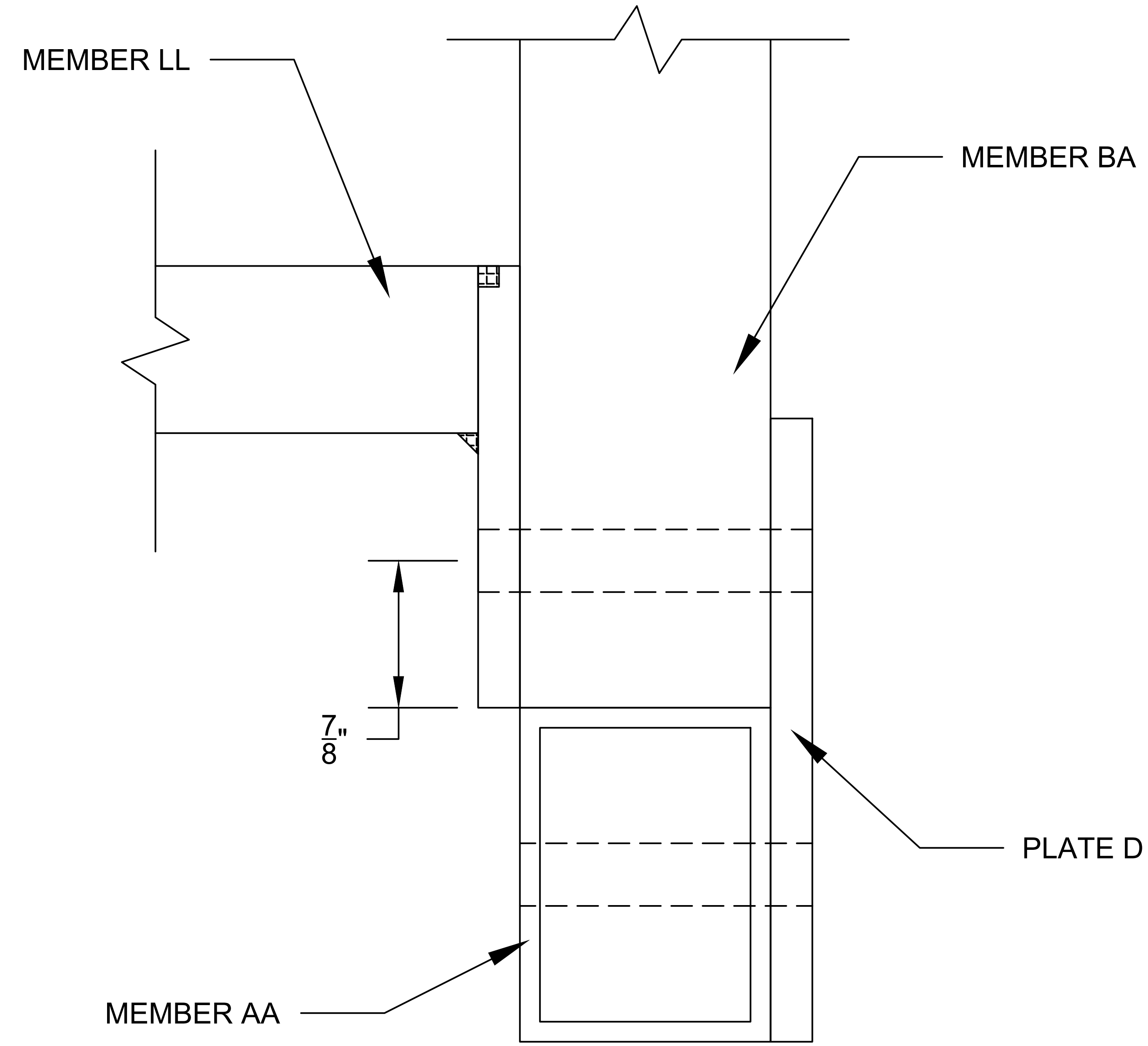


PLATE DIMENSION - CONNECTION L

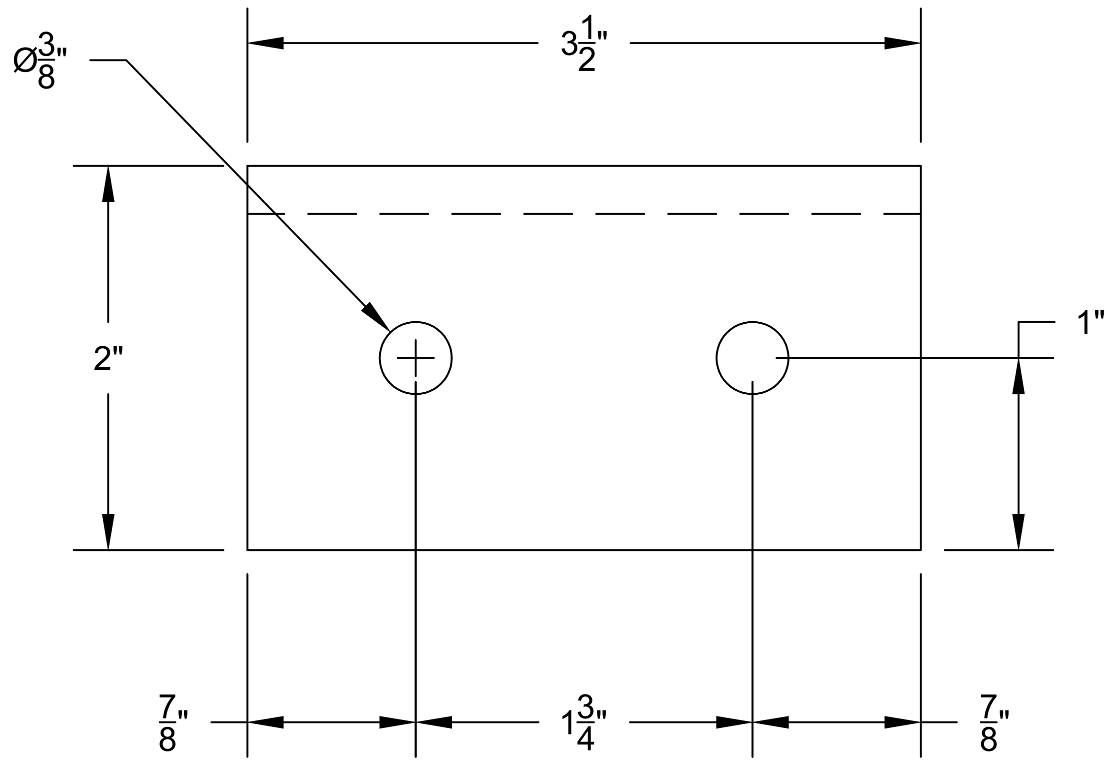
SCALE: 1" = 1'



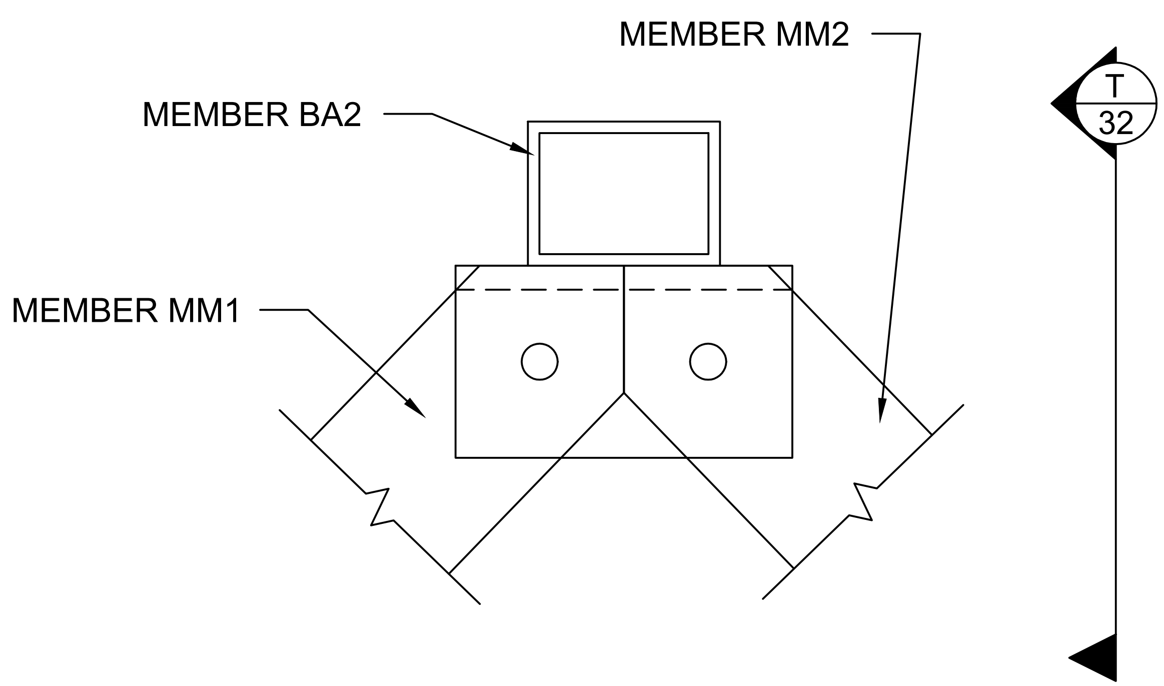
FRONT VIEW - CONNECTION L

SCALE: 1" = 1'

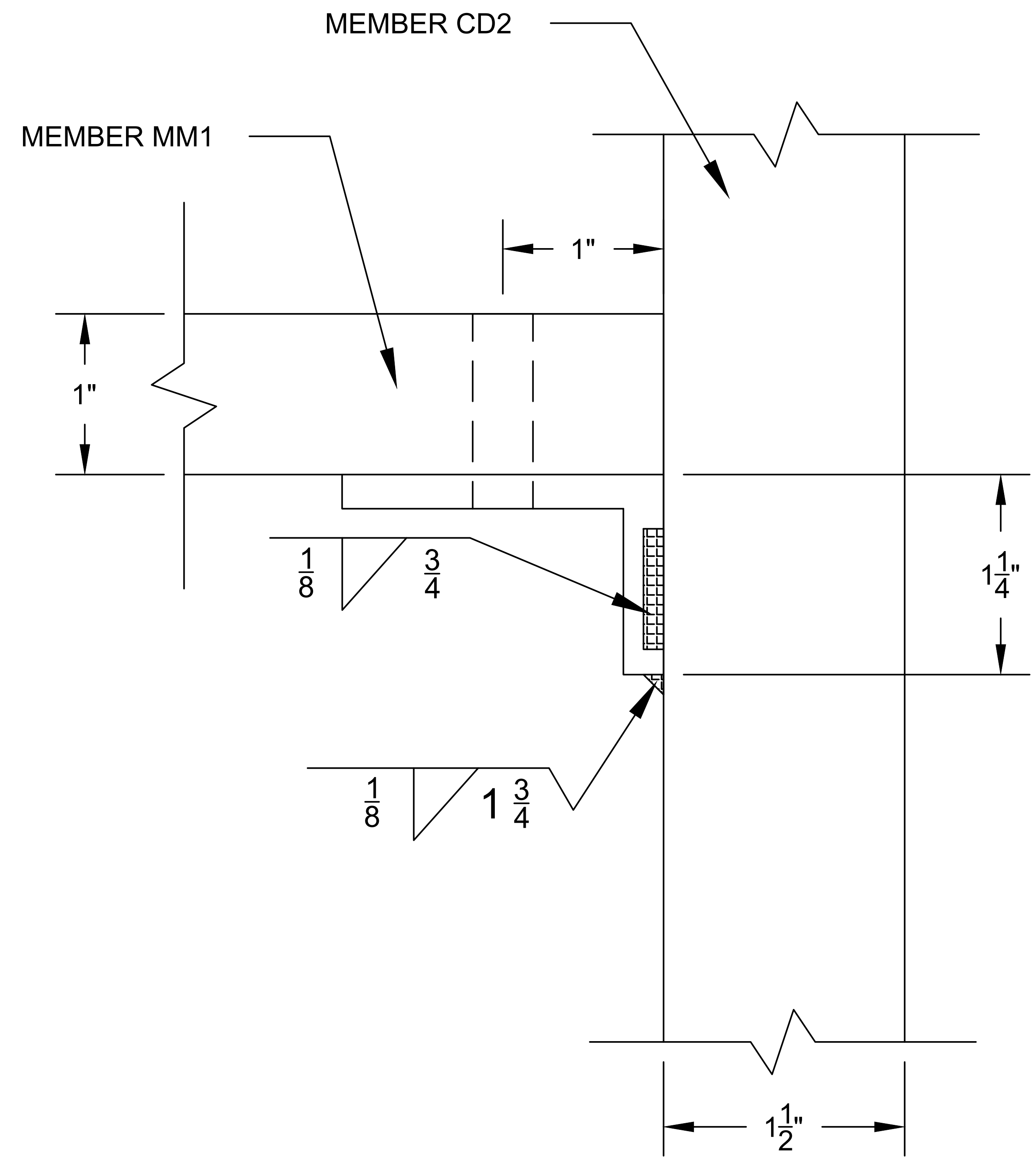
Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03	
	CONNECTION DETAIL L	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 31	OF: 34
			DATE: MARCH 2026		



TOP VIEW - CONNECTION M
SCALE: 1" = 1'

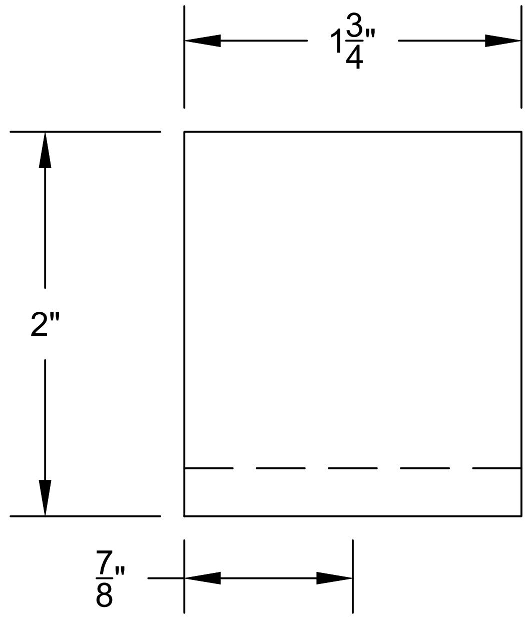


TOP VIEW - CONNECTION M
SCALE: 6" = 1'



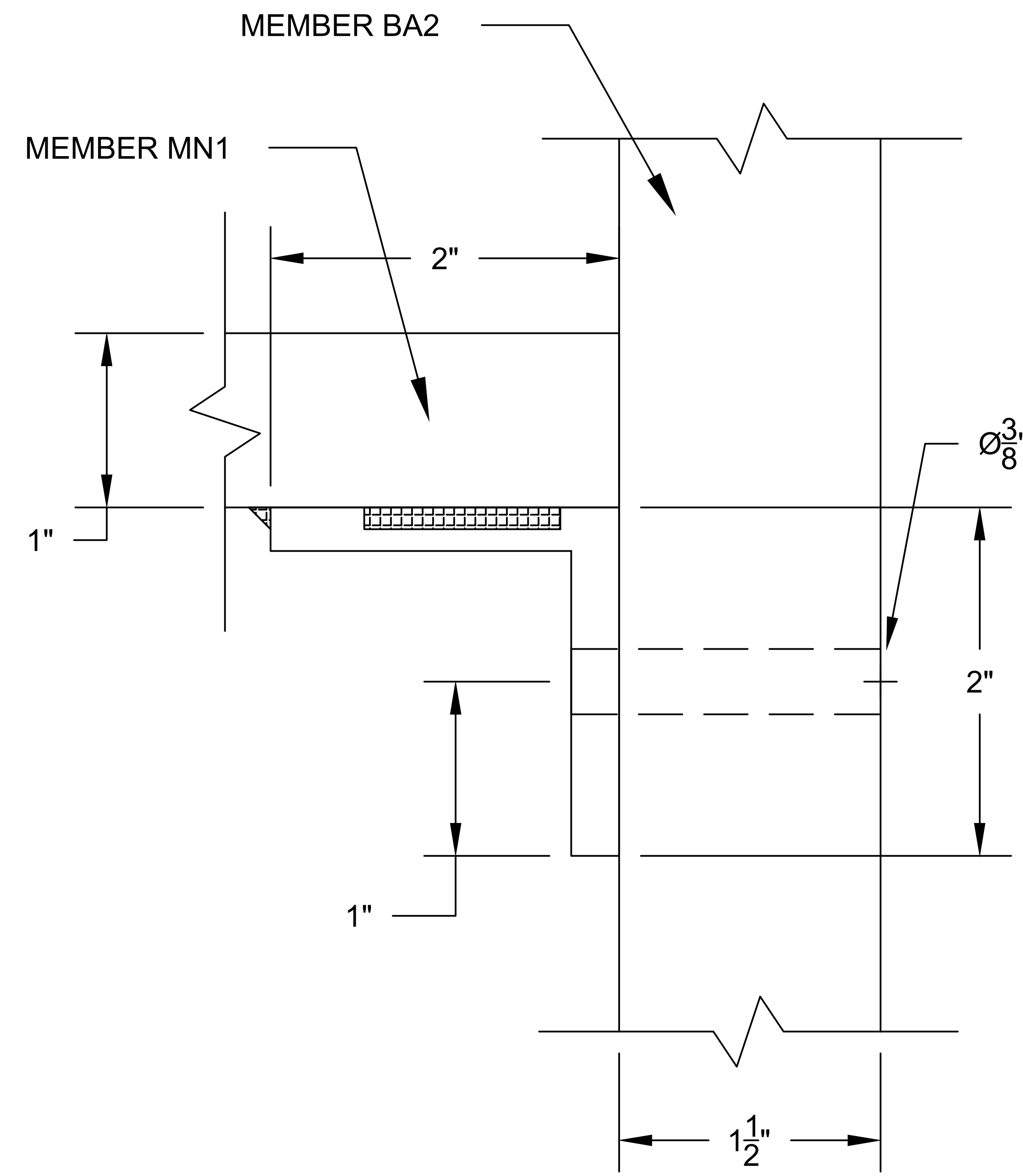
DETAIL T
SCALE: 1" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03
	CONNECTION DETAIL M	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 32 OF: 34
			DATE: MARCH 2026	



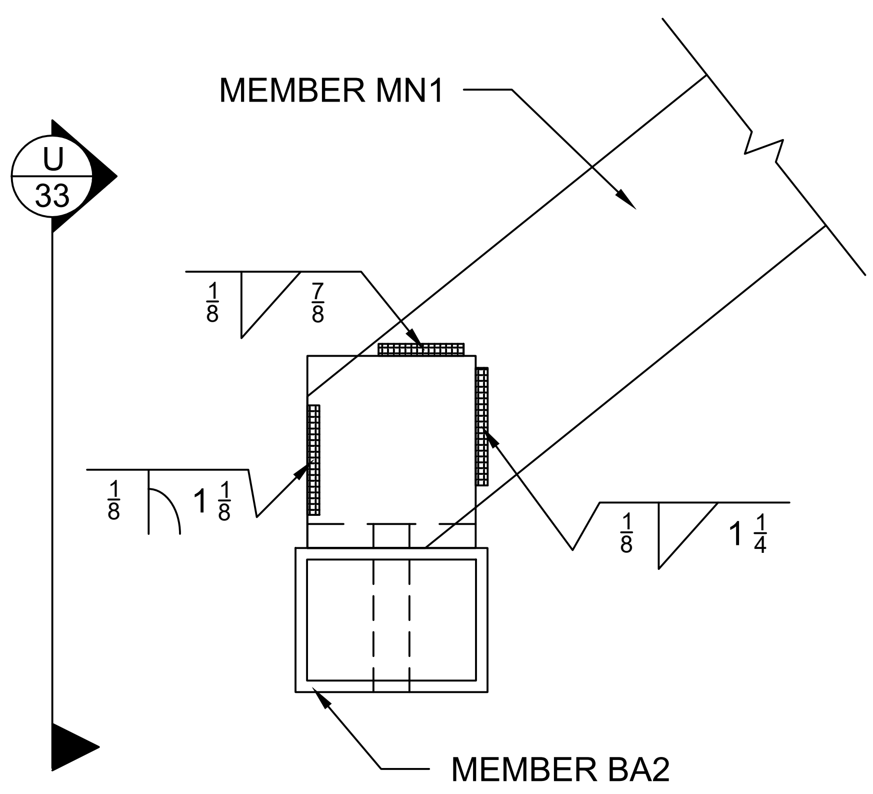
TOP VIEW - CONNECTION 0.5M

SCALE: 1" = 1'



DETAIL U

SCALE: 1" = 1'



TOP VIEW - CONNECTION M

SCALE: 6" = 1'

PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03	
CONNECTION DETAIL 0.5M	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 33	OF: 34
		DATE: MARCH 2026		

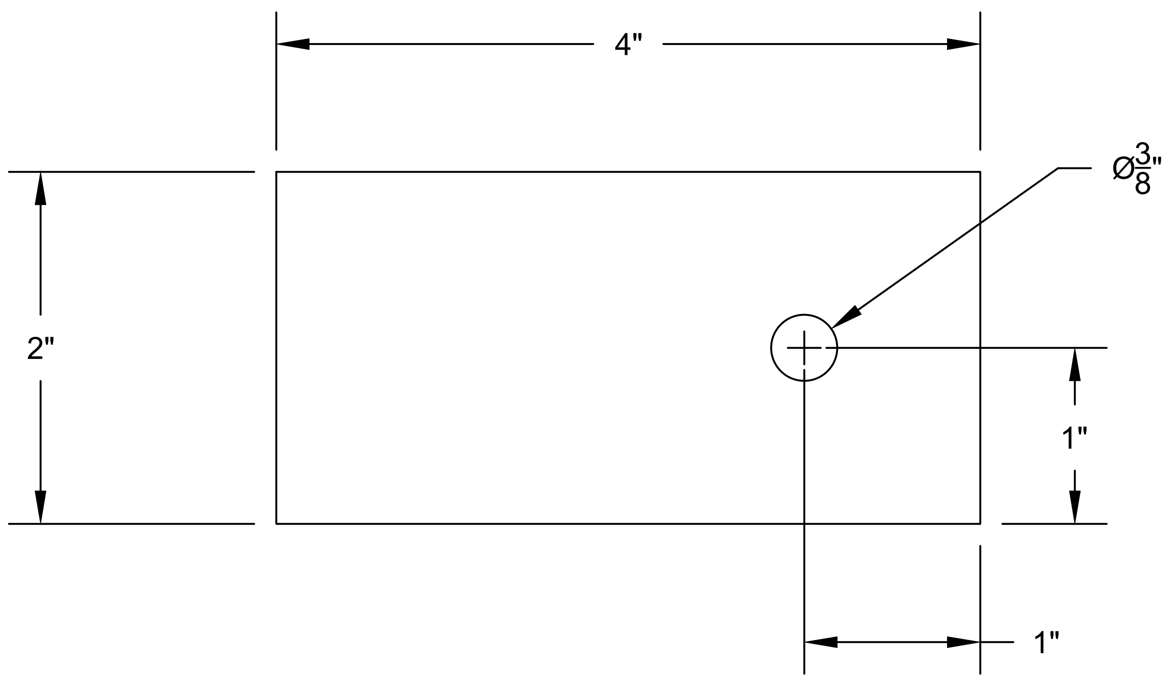
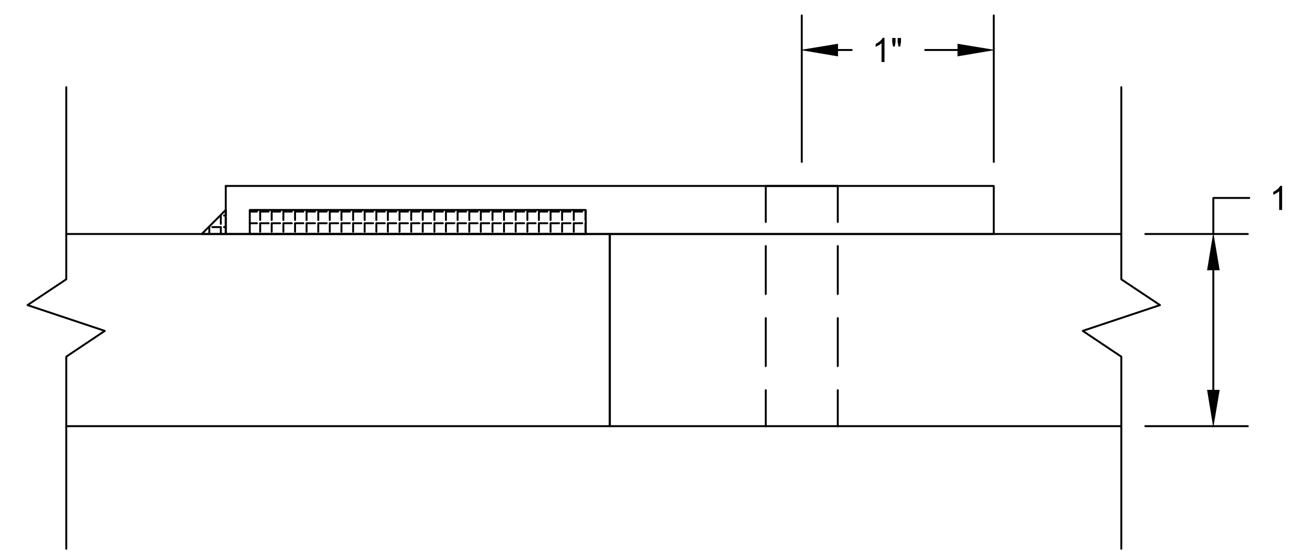


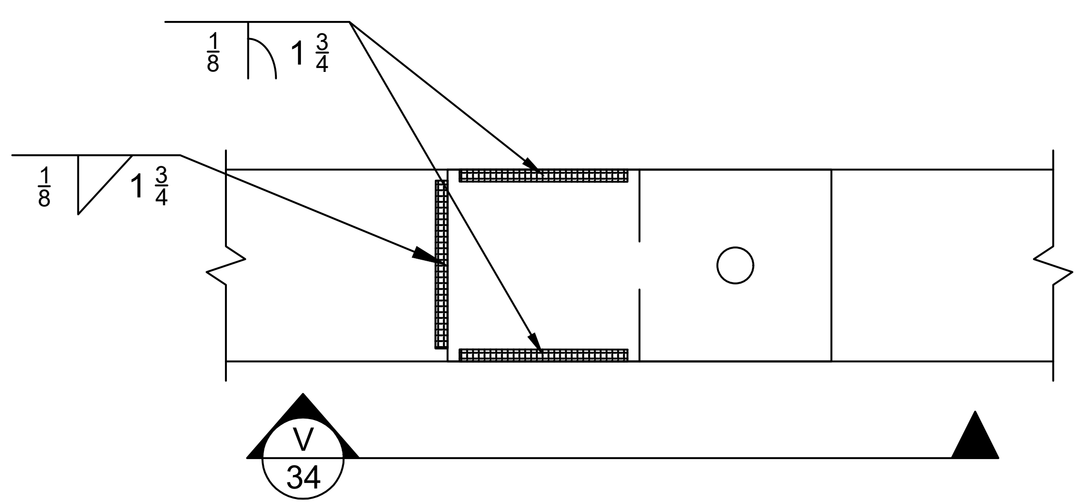
PLATE DIMENSION - CONNECTION N

SCALE: 1" = 1'



DETAIL V

SCALE: 1" = 1'



TOP VIEW - CONNECTION N

SCALE: 6" = 1'

Beyond Bridges	PROJECT: NAU 2025-2026 STEEL BRIDGE		BY: MA	REVISION: 03
	CONNECTION DETAIL N	SCALE: N/A	CHECKED BY: KJ, KF, & IK	PAGE: 34 / OF: 34
			DATE: MARCH 2026	

A-5: Competition Poster

The poster can be seen on the following page.

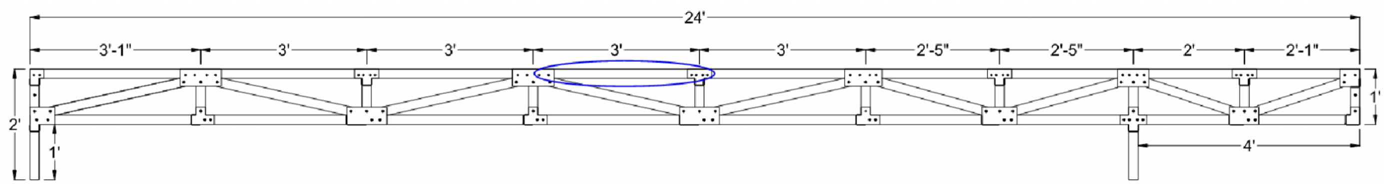
Beyond Bridges

Northern Arizona University

2026 Student Steel Bridge

Team: Isaiah Kimmerle, Kealohamailani Jacob, Megan Alexander, & Kurtis Froyd

NAU NORTHERN ARIZONA UNIVERSITY



Bridge Selection

The bridge was selected based on Risa-2D and STAAD analysis. Of the potential designs considered, this bridge had the least connections and handled the required loads with minimal deflection.

Connection Selection

Once the bridge configuration was chosen, the connections were designed based on the competition's dimensional constraints and connection restrictions. Welds were placed only on connection plates so all members could fit inside the designated box, while also increased the bridge's stiffness. Bolted connections were designed to vary between a 1/4th bolt size and a 3/8th bolt size to account for the stringer template and decrease the total required bolts.

Limit State Calculation Example

Internal Stress Calculation - Method of Sections

$\Sigma F_x = 2.62+0.366*\cos(17°)+BC$

$R_{uBC} = 2.95$ kips (C)

Controlling Limit State: Bolt Shear Capacity

$\Phi R_n = 0.75 F_{nv} A_b n_b n_s$

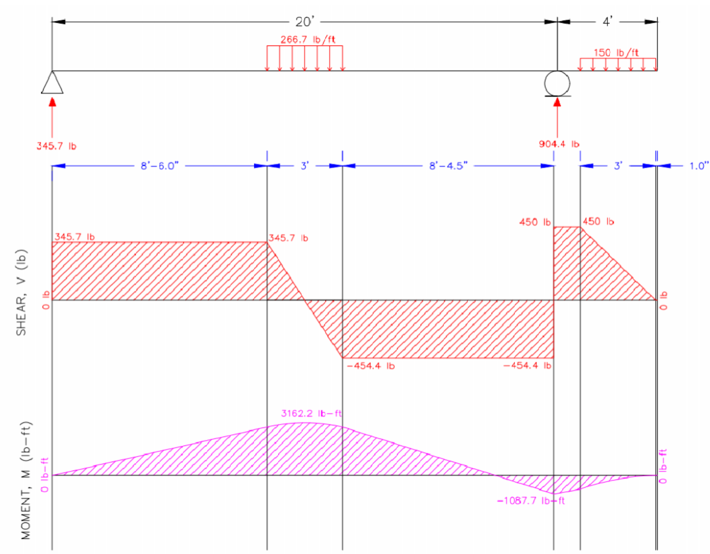
$= 0.75[0.625(54\text{ksi})(0.9)](0.049\text{in}^2)$(2 bolts)(2 shear planes)

$\Phi R_n = 4.47$ kips

$R_{uBC} \leq \Phi R_n$

Calculated Limit States

Bolts	Bolt Tension Failure, Bolt Shear Failure, Bolt Bearing & Tearout, Block Shear, Gross Section Yielding(Tension and Shear), Net Section Rupture (Tension and Shear)
Welds	Weld Capacity, Shear Yielding & Rupture, Gross Section Yielding in Tension, Net Section Rupture in Tension, Block Shear



Structural Analysis

Preliminary 2-D analysis was done through Risa-2D for three prototype designs: Pratt, Howe, and Warren trusses. The Warren truss performed best, followed by the Pratt. The Warren and Pratt trusses modeled in STAAD software for 3D analysis. The two designs were compared through a decision matrix revealed that the Warren design was optimal for sway and constructability.

Bolt and weld calculations were performed in Excel once general member dimensions were mapped in STAAD. The calculations were done according to AISC's 16th Edition Steel Manual. Calculations helped determine the required thickness of each member for welds to reach full capacity as well as bolt connection capacities. After checking capacities of three bolt diameters in the Excel calculations (1/4", 3/8", and 1/2"), 1/4"and 3/8" diameters were selected for their high capacity to overall weight ratio.

The team performed final deflection calculations with the virtual work method for the worst load case according to the STAAD software. The loads were conservatively calculated by having a 40/60 split in load dispersion between the two side panels of the bridge. Comparing the internal stresses found in the deflection calculations to the previous bolt and weld calculations confirmed that the bridge would be able to withstand the applied loads.

Acknowledgements

Dr. Robin Tuchscherer – Grading Instructor
Mark Lamer – Project Client
Sabrina Gibson – Technical Advisor
Page Steel – Donated steel members and plates
Copper State – Donated nut and bolts