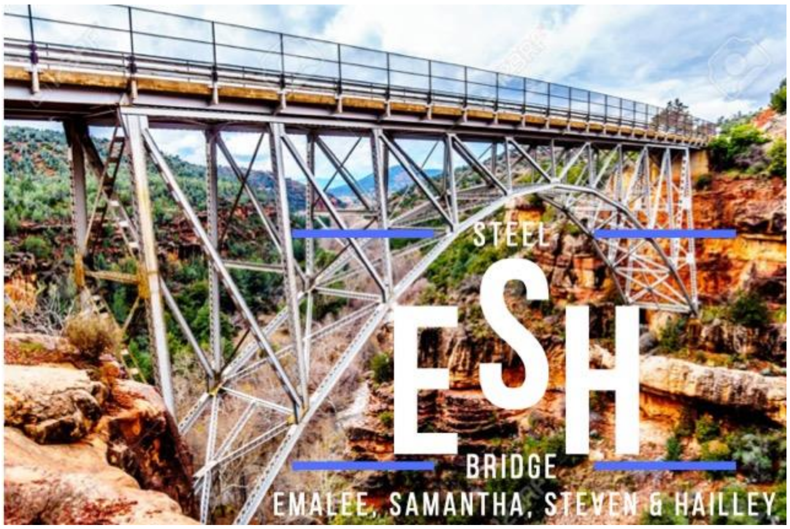
College of Engineering, Informatics, and Applied Sciences




Steel Bridge Competition Team 2020

ESH

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**90% Design Report**

**April 16, 2020**

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# Acknowledgements

The 2020 Steel Bridge Team would like to acknowledge all individuals that assisted with their time, knowledge or service. To our professional sponsors that provided a service, Copper State Nuts and Bolts, Page Steel, K-Zell Metals and Mingus Welding Team. The team acknowledges Thomas Nelson, Sabrina Ballard, Mark Lamer, Dr. Joshua Hewes, Byron Browning and Van Peterson for their technical knowledge.

# 1.0 Project Introduction

The purpose of this project is to design, analyze, fabricate and construct a scaled steel bridge and compete in the 2020 Steel Bridge Competition. The bridge will be a multi-modal skewed 1:10 scaled bridge. The provided problem statement from the American Institute of Steel Construction Student Steel Bridge Competition 2020 Rules details that the bridge is being used for Katy Trail State Park in Missouri [1]. The bridge is to act as a pedestrian, bicycle, and equestrian passage over the Missouri River. The competition rules detail a specific envelope that constrains the dimension of the entire bridge and individual bridge members.

The competition is held annually in conjunction with the American Society of Civil Engineers Pacific Southwest Conference. The team will accompany the Northern Arizona University chapter to California State University Fullerton to compete in April.

## 1.1 Competition Description

Northern Arizona University (NAU) participates in the American Institute of Steel Construction (AISC) Student Steel Bridge Competition (SSBC) annually. This is a regional and national competition; additionally, NAU is participating on a regional level at the Pacific Southwest Conference (PWSC) on April 1-4, 2020 at California State Fullerton University. The team will compete in aesthetics, construction speed, lightness, stiffness, display, construction economy and structural efficiency. To advance in the competition, the team must excel and score the lowest in each category, which equates to the highest rank.

### 1.1.1 Constraints and Design Considerations

The unique design consideration for the 2020 competition is that both ends of the bridge are skewed 1’-6”. The material must be standard grade steel. Individual members cannot exceed 42” by 6” by 4” in size. The bridge itself must remain in the provided envelope in Figure A1 and A2 in Appendix A. The bridge cannot exceed 22’ in vertical length and cannot be more than 3’-8” wide. The top chord of the bridge must be at least 1’-7’’ but no more than 1’-11’’ in vertical height. The bottom chord must clear 7” off the ground. All bolts must be 3’’ long or less and be fully engaged with a nut.

The bridge will be loaded laterally and vertically. A 50-pound load will be applied laterally to measure sway. Sway cannot exceed 1 inch and will be measured from the southside of the bridge. There will be a maximum of 2500 pounds applied vertically to the bridge to test deflection. The bridge cannot deflect more than 3 inches.

### 1.1.2 Competition Categories for Scoring

The competition will score competitors based on construction speed, aesthetics, lightness, stiffness, display, construction economy, and structural efficiency [1]. These categories will account for the overall performance of the bridge; the bridge with the lowest combined score from each category will win the competition. The overall performance rating is the sum of the construction cost, structural cost and any fines incurred as violation during competition.

Bridge aesthetics includes the overall look of the bridge and excludes fabrication quality. A poster is required during display. The poster requires a bridge explanation, a scaled and dimensioned side view of the bridge, a free body diagram of the beam's stringers with shear and moment diagrams, and an explanation of the team's preparation for construction. A poster is necessary to score well in that category.

The scoring for structural efficiency is dependent on deflection, load penalties, and weight of the bridge. The categories of scoring are detailed in Section 6 of the AISC SSBC 2020 Rules [1].

## 1.2 Objectives and Deliverables

The primary objective of this project is to design, fabricate, construct and display a fully working bridge within the restrictions of the AISC rules. The team will use statics, steel design, structural design, and construction methods to complete the project. The outcome of the project is to fully understand bridge design as it pertains to the 2020 AISC Steel Bridge Rules.

The team deliverables are to create a fully functioning bridge, a poster to be used during display, a design report, and a final presentation. In completion of the bridge, the team will produce shop drawings. Shop drawings will be used to assist with fabrication and construction. These drawings will include necessary details for the function and explanation of the bridge.

### 1.2.1 Technical Considerations

The Steel Bridge Team will be doing structural analysis and design, material selection, fabrication, connection design and construction to create a competitive product for competition. The competition will have lateral and vertical loading, so deflection in both directions will be considered.

# 2.0 Bridge Type Selection and Design

The team considered two bridge designs upon reading the 2020 AISC Student Steel Bridge Rules, a beam and truss bridge based on the provided envelope and structural needs. A beam bridge is simple for the necessary loading and requirements. Beam bridges endure compressional stress on the top side and tensional stresses on the bottom. This load distribution can lead to the top to buckle and the bottom side to snap. Beam bridges need two supports at each end and a horizontal structure that rests on the end supports which does not align with the AISC SSBC Rules, as all members must be connected by a fully engaged nut and bolt connection. With this knowledge the team figured that the bridge would need to be able to support a deck. Additionally, a bridge that can be uniform and symmetrical with the skew was considered in design for ease in construction.

Truss bridges utilize multiple triangles to support heavy loads. Truss bridges are stronger and more efficient because of the triangle design. The triangle design is important to the strength and integrity of the bridge. The truss structure effectively manages compression and tension by spreading out the load through the structure. Trusses are built on a system of connections, and one weak connection or member can affect the overall behavior of the bridge. Truss bridges can become complicated because of the number of connections and pieces necessary. It can also become very heavy with the number of connections and member pieces. However, a truss bridge can also be a lighter option as it allows forces to be transferred over a long span better. Bridge weight was considered a part of competition and constructability. The team considered deck bridges based on the given AISC Rules and the length and height required for the bridge. The Warren, Pratt, and Howe deck trusses were considered and designed.

A Warren bridge can be designed with or without verticals like Figure 2-1 and 2-2 below. The Warren Truss has parallel chords and alternating diagonals. The Warren Truss without verticals is simple to design to understand and follow the movement of loads. The Warren Truss is designed with equilateral triangles; it is typically useful for distributed loads over a long span and spreads the loads evenly between members. Concentrated point loads do not distribute well across members and typically perform poorly. The compressive loads are usually at the top chord and in the center while tension is usually on the bottom chord.

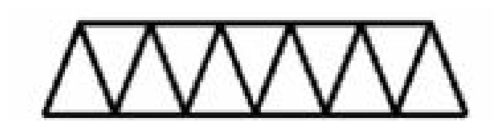


*Figure 2-1: Warren Truss without Verticals Design [2]*

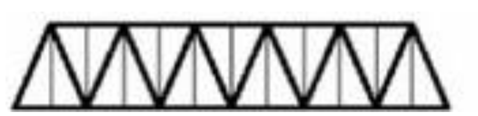


*Figure 2-2: Warren Truss with Verticals Design [2]*

The Pratt truss in Figure 2-3 has diagonal members in tension and the vertical members in compression. The diagonal members create a more efficient design because less steel is necessary. This design was considered because it can be lighter. This design is more useful when the applied load is predominately in the vertical direction but horizontal loads excel as well. It also can be a simple and up-front design to create.

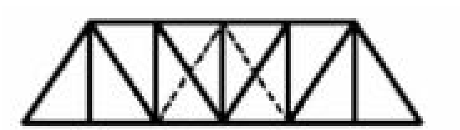


*Figure 2-3: Pratt Truss Design [2]*

The Howe truss is the opposite of the Pratt truss in geometry. It is diagonal bracing goes in opposite directions and changes the way loads are distributed. The diagonal members are in compression and the vertical loads are in tension. The Howe truss in Figure 2-4 below is as effective as the Pratt truss but the load is distributed differently. The Pratt truss can have more unloaded members in comparison to the Howe truss.

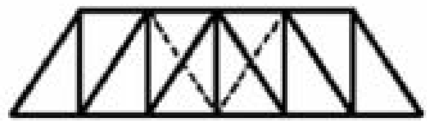


*Figure 2-4: Howe Truss Design [2]*

To begin designing, the team used the RISA program to render different truss bridge designs or underslung truss designs using these four truss bridge types as a guide.

# 3.0 RISA

## 3.1 Computer Modeling

The Steel Bridge Team used the RISA 3D structural software to design and analyze the bridge. This model was created using the truss bridge design with members connected by either a fixed or a pinned connection. Fixed connections represent welded pieces and pinned connections represent connections like plates with nuts and bolts. The fixed connections are used for the vertical webbing and the columns of the bridge as these members would have gusset plates welded onto them. The pinned connections are for the top chord, bottom chord, angled webbing, and bracing members as these members need to have a nut and bolt connecting the member to the gusset plate. As an additional design tool, the unity factor was used to determine if the bridge structure would hold. The unity factor depicts the experienced stress of the member over the allowable stress of the member. The unity check aided in selecting member cross sections as well as member grade for the final design.

To ensure that the model being designed could withstand all applied load location variations at competition, each load combination was entered into the RISA model. Once the model was finished being designed, the model was solved with all six vertical load combinations and the lateral combinations. Following this, the bridge was examined to see if any members were failing or were highly stressed. If a member was failing or highly stressed, i.e. had a unity factor of 0.97, the cross section of the member was increased or the thickness was increased.

RISA was essential to understanding the design and performance of the final bridge design. Over thirty iterations of the bridge design were completed. Changes to member location, material selection, member length, truss design, and member cross sections were made. In addition to the iterations for specific members, iterations for the shape of the bridge fitting within the competition constraints were done.

## 3.2 Load Combination

The loading combination was taken out of the AISC SSBC 2020 rules. There is one lateral loading condition and six vertical loading conditions. The lateral loading condition has the lateral load being placed on a decking unit who's left edge is 9’ from the east end of the bridge on the south side. The vertical loading conditions are placed in accordance with Table 3-1 with the distance being determined by a dice roll at competition. Location 1 and 2 is measured from the east end and south side of the bridge, respectively, to the left edge of the decking unit.

*Table 3-1: Location Determination by dice roll provided by AISC [1]*

| N (dice roll) | Location 1 | Location 2 | Lateral Location |
|---|---|---|---|
| **1** | 8’-0” | 3’-0” | 9’-0” |
| **2** | 10’-0” | 4’-0” | 9’-0” |
| **3** | 11’-0” | 7’-0” | 9’-0” |
| **4** | 12’-0” | 3’-6” | 9’-0” |
| **5** | 12’-6” | 6’-0” | 9’-0” |
| **6** | 13’-0” | 8’-5” | 9’-0” |

Calculations were performed when a design was completed to see if the design could withstand the applied load. To completely account for all possibilities in loading, all six possible loading cases were applied to the bridge, along with the lateral load case. Each bridge was then solved under the designed loading conditions. Finally, after designing multiple iterations of bridges with the design considerations, the bridge that did not fail with the least stressed members, lowest weight, and smallest deflections was selected as the final bridge design. All calculations for the final bridge design are compiled into the RISA report located in Appendix E.

## 3.3 Material Selection

In conjunction with the selection of the final bridge design, a uniform steel grade of A513 was selected. The bottom chord, webbing, and columns are all made out of 1”x1”x0.065” HSS steel with lengths selected to fit into the RISA design. The top chord is made of 2”x1”x0.083” HSS members, and bracing is 0.75"x0.75”x0.065” HSS steel.

The tubing for the top and bottom chord is different because when modeling the bridge in one of the iterations, when both tubing sizes were the same the bridge failed. When the top chord was

2”x1”x0.065” the bridge failed the loading test conditions. However, with the 2”x1”x0.083” cross section, the bridge no longer failed any of the loading test conditions.

# 4.0 Connection Design

## 4.1 Connection Type Options

There are many ways to join pieces coming together at a node. The type of connection you design will impact the RISA results, as a welded connection between the member and the node will yield different results than a bolted connection between 2 parts. The team considered slip connections and gusset plate connections, and ultimately went with gusset plate connections due to their simplicity. The plates had to be custom designed to fit the pieces at each node. Due to the symmetrical nature of the bridge, a total of 6 different plates could be used for the entire bridge, with all pieces being used multiple times. The plan set in Appendix C shows all 6 connections with their dimensions, and the welding drawings show exactly how they will be connected to each piece. Additionally, tabs needed to be welded onto the interior facing top chord connections to support the lateral bracing on the bridge. The team went with welded tabs due to their simplicity, despite there being a few other ways to attach the horizontal pieces to the left and right side of the bridge.

## 4.2 Plate Design

The connections were designed by rendering connection ideas in AUTOCAD. The renderings were completed while being mindful that the connections can only be welded onto members if they were within the 42”x6”x4” box. This thought process enabled the team to have many connections welded on to other members, and therefore cut down on construction time. Multiple iterations had to be done to narrow down the design for the connections. The connection design not only impacts the RISA results, but also the strength of the piece to withstand the forces acting on the bridge when it gets loaded at competition.

Analysis was completed to design the connections of the bridge. The connections were solved by taking the shear strength, stress, and tensile strength given by the RISA report calculations, and comparing them to the calculations performed in Microsoft Excel. Properties such as the steel yield and ultimate strength, thickness of the plates, hole size, bolt placement, number of planes in shear, and height of the rupture plane were taken into consideration for the calculations. Once done, each connection was modified based on the results obtained. For example, if the connection was under designed, then another plate could be welded on and 1 more plate was added on in the calculations.

### 4.2.1 Tension Capacity

The tension capacity of each connection was calculated by using the AISC Steel Construction Design Manual and referencing the equation in Section D1. The yield strength of the steel, the gross area of the shear plan, the number of plates the plane is going through, and the phi factor for safety (0.75 for LRFD) were multiplied together.

### 4.2.2 Bolt Shear Capacity

The bolt shear capacity was found by using equation J3-4 in Section J3 of the manual. The area of the bolt hole, along with the phi factor (1.00) and the planes of shear (2) were multiplied together.

### 4.2.3 Block Shear

The block shear for the failure paths on the connections were calculated by referencing Section J3 in the Steel Construction Design Manual. For some connections there are many rupture paths, and as such, if one was failing then it did not matter what the other paths were as the failing path would govern. The shear and tension paths were established, along with the pre-established properties of bolt placement, hole size, and plate thickness, and equation J4-5 was used to find the block shear capacity. This was done in conjunction with the safety factor of 0.75 for LRFD.

### 4.2.4 Tensile Rupture Strength

The tensile rupture strength for the connections were calculated by referencing the equations in Sections D2 and D3.  The net effective area had to first be calculated before it could be multiplied by the ultimate strength of the steel and the safety factor of 0.75 for LRFD.

### 4.2.5 Bolt Bearing Strength

Section J3-6a in the Steel Construction Design Manual was used to find the bolt bearing strength of each connection. The properties of the connection such as diameter of bolt holes, thickness of the plate, and ultimate strength of the steel in ksi were multiplied together.

# 5.0 Final Design

## 5.1 Completion of RISA 3D Modeling

The final design decided by the 2020 NAU Steel Bridge team used an underslung truss design that closely models the Howe truss. The difference in the design and the Howe Truss is the bottom chord has an extra horizontal piece at the ends that connects to the columns. In order to relate the entire bridge lateral bracing was necessary. This was done to prevent lateral sway and deflection; running the RISA model showed lateral bracing is essential in preventing deflections during the lateral testing at competition. Figure 5-1 depicts the isometric view of the bridge from RISA 3D.

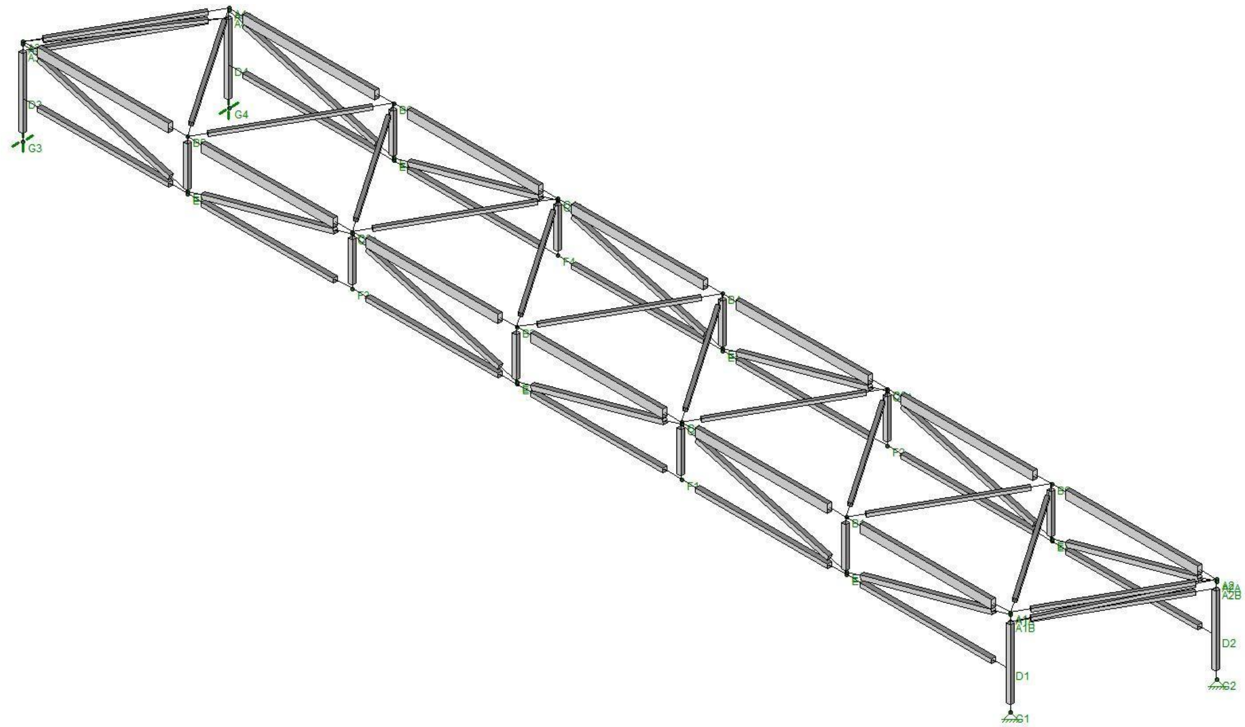



*Figure 5-1: Isometric View of Final Design from RISA 3D Model*

The final design has a total length of 21’-7” from end to end, a height of 1’-10”, and a width of 2’-9”. The underslung truss design is 10’’ from the top chord and bottom chord.  The bottom chord has a ground clearance of 8.5”. There are 25 members on each end and 17 lateral bracing members. At the end of each side the three lateral bracing members will be welded together and combined into one member. The shop drawings in Appendix C detail the full scale and design of the bridge.

After the final design was made, all calculations were compiled into the RISA report. Upon completion of the final bridge design, a uniform steel grade of A500 was selected. The bottom chord, webbing, and columns are all made out of 1” x 1” x 0.065” HSS steel with lengths selected to fit into the RISA design. The top chord is made out of 2” x 1” x 0.083” HSS steel, and bracing is 0.75” x 0.75” x 0.065” HSS steel.

RISA analyzed our final bridge design with a unity check. This unity check took the worst-case scenario for each loading combination at a 1.1 scale and took the self-weight at a 1.25 scale. The scales were placed specifically to ensure the bridge accounted for the weight of all the gusset plates, nut, and bolts. The results of the bridge unity check were that the bridge did not fail and all the members were not severely stressed. The final bridge, however, did have different stresses from other members. The two columns on the right side of the bridge are both stressed at 56%

and 70% respectively. In addition to this, the two-center bracing on the bottom chord is stressed in the mid 50% range. All four middle top chord members are stressed between 67% and 89%. The rest of the bridge is stressed between 0% and 50%. Over all, these members are stressed, but not overly stressed or affecting the stability of the bridge.

Using the same inputs and looking at each loading case individually, RISA 3D calculated the bridge deflections. Our bridge is designed to have a lateral deflection of 0.3 inches and a vertical deflection of -0.83 inches. This is the worst case scenario result, and it is within the requirements of the AISC rules of 1-inch lateral deflection max and 3-inch vertical deflection max.

## 5.2 Connection Calculations

The finalized calculations for each connection design are presented in Appendix F. Connection E and F were under-designed when modeled for two plates. To rectify the issue, three plates are needed; two plates need to be welded together on the exterior facing side and one plate is needed at the interior facing side. Welding two plates together causes them to act at about 85% more strength than just one plate.

## 5.3 Shop Drawings

Shop drawings were made with dimensions, labels, angles, connections, bolts, and welds to ensure an understanding for construction and fabrication. The shop drawings also include material labels and the amount required of each item placed in schedule tables. The 60% shop drawings plans are shown in Appendix C. Shop drawings were done to communicate the necessary cuts and bolt holes for the subcontractor, K-Zell Metals. Another set was done for the Mingus Welding team with specific weld instructions.

## 5.4 Mingus Drawings for Welding and Assembly

The drawings for welding and assembly are located in Appendix C. These drawings were rendered in AutoCAD, using the existing plan sets. The welding plans considered what members could be welded and fit inside the box. The members that were small enough to fit in the box were the webbing members BE and CF, and column AA. Members BE, CF, and AA were rendered with the gusset plate fixed. When the members and connections are welded together they are considered one member, so these members were dimensioned where welding would occur in AutoCAD. Following this, the member assembly was indicated on the welding plans with an isometric view of the bridge.

# 6.0 Fabrication

To begin fabrication for the bridge design, the team began by cutting the steel to the appropriate size. Then, the steel was cleaned by deburring the edges and degreasing the surface with acetone. Lastly, welding was done according to the plan sets.

## 6.1 Cutting Steel

To begin cutting the steel the team measured the correct size using a tape measure. The measurement was marked with soapstone at an appropriate angle with the use of a triangle. After the steel was perfectly marked, the cutting material was prepared. Two types of cutting materials were used: the hand grinder and the chop saw. The chop saw was prepared by clamping down the machine to the table and clamping down a piece of metal to the table at the correct distance, then the metal was cut. This ensured a precise and fast way of cutting the metal. The hand grinder was prepared by clamping down the steel in the vice and putting on the correct grinder disk to cut the steel. Both the hand girder and chop saw cut the steel just outside the measured line indicated with soapstone. This method was used to cut the steel to the correct measurement and not slightly under.

## 6.2 Preparing Steel

After cutting, the steel was cleaned. Two cleaning methods were performed in preparation for welding. The first method for cleaning was deburring the steel. Deburring was completed by using the table girder and hand grinder. The edges of the steel were grinded to deburr and make the steel less sharp. Wire grinders were chosen to deburr because the steel would not have the potential of decreasing in length as it would with a regular stone grinder. Following grinding, degreasing was completed. Steel needs to be degreased before welding because it can cause a weak weld and the potential for breakage. Degreasing was completed using an acetone-based nail polish remover.

## 6.3 Welding Steel

Welding began after the steel cutting and cleaning. The welding followed the welding plans located in Appendix C. The team dimensioned and marked with a sharpie all locations where the welding should occur. The welding plans were used during the welding process as a quick reference and to answer questions the Mingus Welding team had. The welding required for the bridge was on the webbing, columns, and connections.

The connections E and F required two connections to be welded together to form a single connection. The webbing members labeled AB and CF are to have connections welded to both the front and back side of the tube. The connections welded to AB are connection B to the top and E to the bottom. Bracing CF only had connection F welded to the bottom, while connection C would be held in place with a bolt and nut. Additionally, the column members labeled AA had connections A welded to the top and D welded below. Finally, the tabs were welded on the internal-facing top chord connections.

# 7.0 Engineering Schedule

The assigned schedule for the team has been altered since it was made in the proposal. The proposal schedule is located in Appendix D1. Task 3’s duration has been changed from 2 days to

39 days. It also was completed on the 31st of January while the proposed completion date was the 27th of November. Task 4.1 was moved from being started on 28th of November to starting on the 3rd of February. The new completion date was scheduled to be the 7th of February with steel being returned on the 4th of March. Task 4.2 was moved from starting on the 23rd of January to starting on the 6th of March. Task 5 moved from starting on 12th of March to the 10th of March. Task 5's duration also increased from 4 days to 9 days. The new schedule can be seen in Appendix D2.

The team lost approximately a month when it came to the connection design and the connection calculations due to errors in process. However, the team did complete the welding earlier than anticipated with this delay. Completion of Task 4.2 took 4 days and gave the team back two weeks. Thus, the team will be able to devote extra time to Tasks 5 and 6 in preparation for the AISC SSBC competition on April 1st, 2020.

# 8.0 Engineering Cost

The total project cost is based on two major items. First, the cost for personnel to work on the design is broken up by the hours spent on tasks and the pay rate of the engineer performing the work. Second, the cost for supplies, materials, travel, and subcontracting accounts for the remainder of the cost.

## 8.1 Personnel Cost

The summarized billing rates for the personnel working on the project are presented in Table 8.1 below. The cost per hour for each person will be multiplied by the number of hours worked to achieve the cost for professional service.

*Table 8-1: Billing Rates*

| Personnel | Billing Rate ($/hr.) |
|---|---|
| SENG | 200 |
| ENG | 137 |
| EIT | 72 |
| LAB | 90 |
| AA | 67 |

As of Thursday April 16th, the team has worked 1,143 hours on the project. Appendix B1 outlines the number of hours worked so far in each of the major tasks. Appendix B2-B5 details each team members' individual hour contribution.

### 8.2 Material Cost
The cost for buying supplies, materials, having work performed by subcontractors, and travel need to be accounted for to create a holistic summary of the cost of the project.

The team has ordered steel from a supplier who was willing to work with us for free. Thus, the steel pieces and sheets did not cost any out of pocket money. The travel to and from Phoenix, AZ to drop off and pick up the steel was approximately $120.23.

The Mingus welding team was also able to work with us for free, so the cost for their services was $0.00. However, the steel bridge team offered to buy food and beverages as a gratitude for their service. The food and drinks were approximately $55.72. The cost of gas to travel to Cottonwood, AZ for welding was approximately $80.00.

Certain supplies have been bought to assist with fabrication of the bridge. Markers, duct tape, and stickers have amounted to approximately $52.01 in material cost.

The total cost incurred in the project thus far is approximately $307.96.

## 9.0 Exclusions
The team self-performed all structural related work. The team has completed design and analysis of a small-scale bridge. For fabrication, the cutting and welding of most members was completed by the subcontractors K-Zell Metals and Mingus Welding. The bridge is only a model and therefore, traffic analysis and planning, geotechnical soil analysis and foundation analysis, surveying, and footing design is excluded in the work. Steel cutting and welding will be completed within the context of the project.

## 10.0 Conclusion
This project focuses on analyzing a section of the Katy Trail State Park that needs a new bridge to replace their existing one. A bridge design must be analyzed for feasibility to withstand the loads being applied to it, and still be considered efficient in terms of built time, weight, stiffness, and construction economy.

The final design for the bridge has been completed using RISA 3D, the connections have been designed and analyzed to exceed the demand, and the welding has been completed. The remaining tasks are to make final improvements.

# 11.0 References

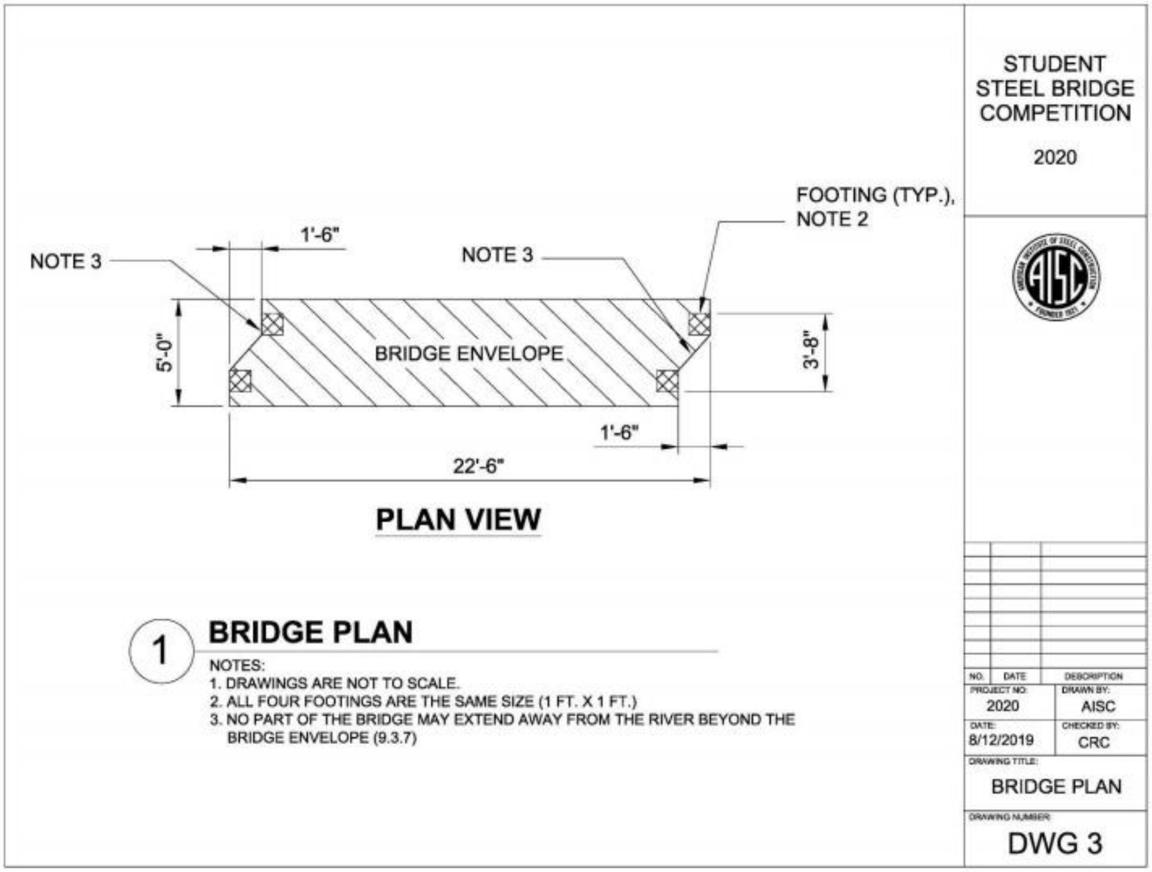
[1] AISC, “Student Steel Bridge Competition 2020 Rules,” American Institute of Steel Construction, 15-Aug-2019. [Online]. Available: https://www.aisc.org/globalassets/aisc/university-programs/ssbc/ssbcrules.pdf. [Accessed: 11-Feb-2020]

[2] North Carolina Department of Transportation, “Truss Bridges” , 7-Jun-2019. [Online] Available: https://www.ncdot.gov/initiatives-policies/Transportation/bridges/historic-bridges/bridge-types/Pages/truss.aspx [Accessed 8-Feb-2020]

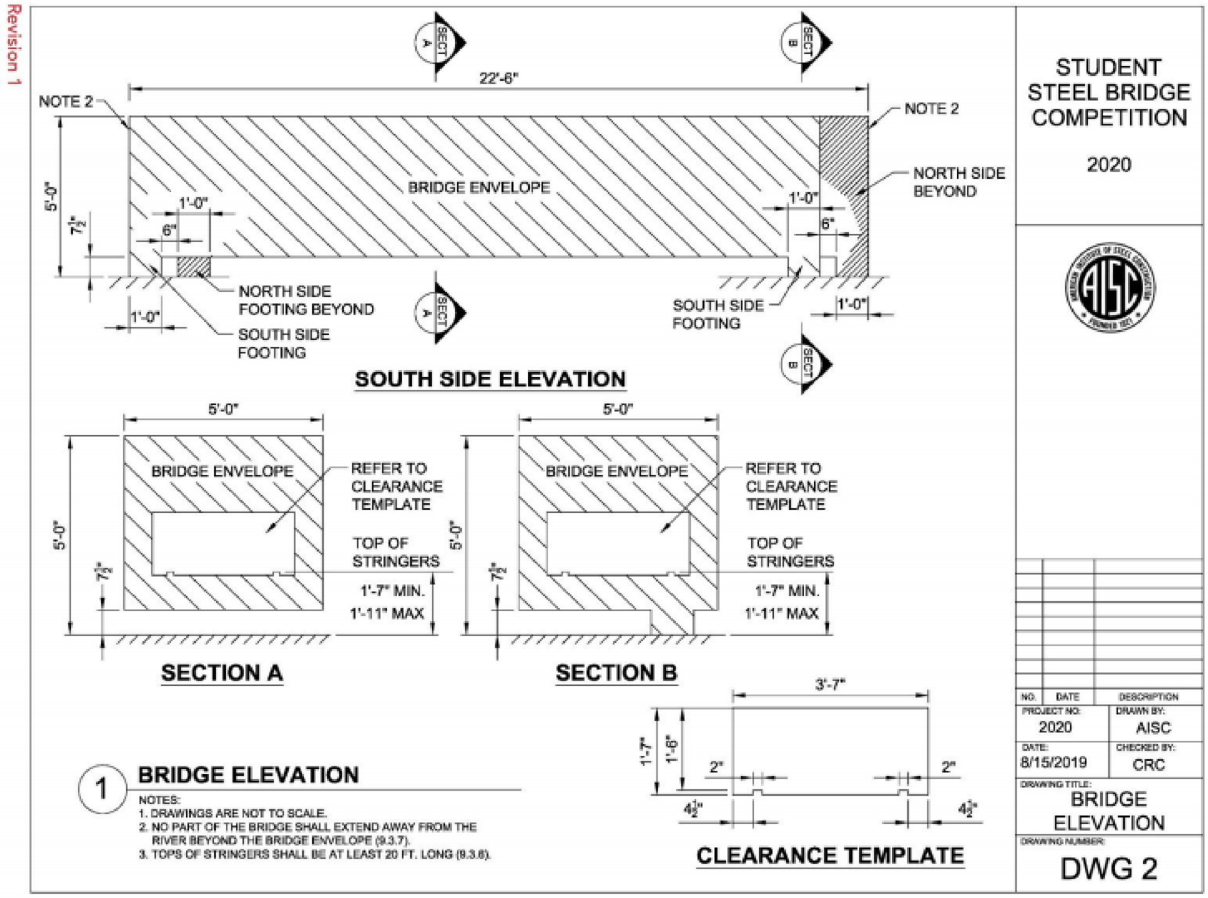
# 11.0 Table of Appendices

## Appendix A: AISC Envelope Requirements

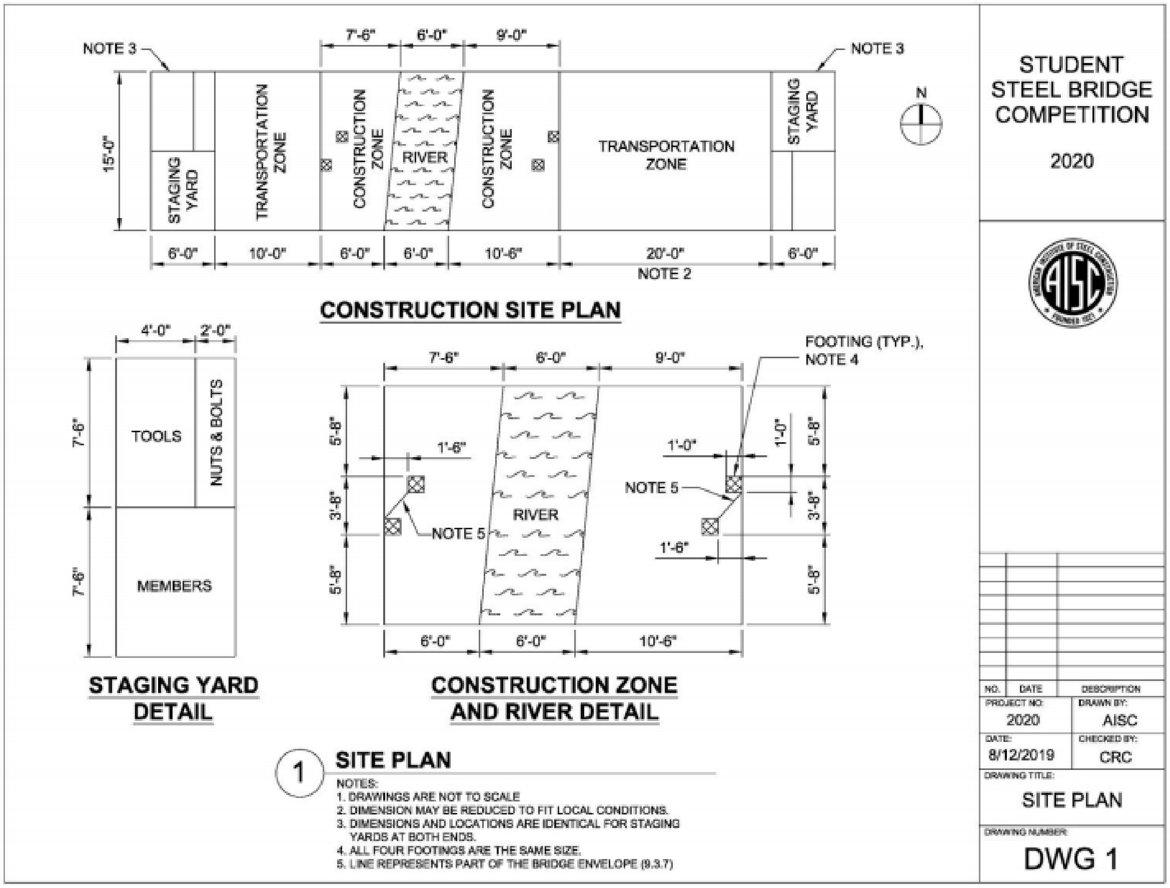
### A1: Plan View of AISC Envelope



## A2: Elevation View of AISC Envelope



## A3: Site Plan


CONSTRUCTION SITE PLAN
NOTE 3
NOTE 2
STAGING YARD
TRANSPORTATION ZONE
CONSTRUCTION ZONE
RIVER
STAGING YARD DETAIL
TOOLS
NUTS & BOLTS
MEMBERS
CONSTRUCTION ZONE AND RIVER DETAIL
FOOTING (TYP.), NOTE 4
NOTE 5
SITE PLAN
NOTES:
1. DRAWINGS ARE NOT TO SCALE
2. DIMENSION MAY BE REDUCED TO FIT LOCAL CONDITIONS.
3. DIMENSIONS AND LOCATIONS ARE IDENTICAL FOR STAGING YARDS AT BOTH ENDS.
4. ALL FOUR FOOTINGS ARE THE SAME SIZE.
5. LINE REPRESENTS PART OF THE BRIDGE ENVELOPE (9.3.7)
STUDENT STEEL BRIDGE COMPETITION
2020
PROJECT NO: 2020
DRAWN BY: AISC
DATE: 8/12/2019
CHECKED BY: CRC
DRAWING TITLE: SITE PLAN
DRAWING NUMBER: DWG 1

# Appendix B: Sum of Hours

## B1: Total Sum of Hours

| | SENG | ENG | EIT | Lab | AA | Sum of Hours |
|---|---|---|---|---|---|---|
| **Task 1: Research** | 1 | 40 | 211 | 0 | 0 | 254 |
| **Task 2: Design and Analysis** | 10 | 61 | 242 | 8 | 0 | 286 |
| **Task 3: Shop Drawings** | 6 | 32 | 110 | 20 | 2 | 116 |
| **Task 4: Fabrication Management** | 13 | 18 | 44 | 24 | 0 | 78 |
| **Task 5: Final Product Improvement** | 0 | 2 | 5 | 2 | 0 | 9 |
| **Task 6: Competition Preparation** | 1 | 0 | 0 | 0 | 0 | 0 |
| **Task 7: AISC Competition** | 1 | 3 | 4 | 2 | 0 | 10 |
| **Task 8: Project Management** | 21 | 63 | 140 | 17 | 34 | 390 |
| Personnel Hours | 52 | 219 | 756 | 73 | 36 | 1143 |

### B2: Hailley Sum of Hours

| | SENG | ENG | EIT | Lab | AA | Sum of Hours |
|---|---|---|---|---|---|---|
| **Task 1: Research** | 0 | 8 | 15 | 0 | 0 | 23 |
| **Task 2: Design and Analysis** | 0 | 10 | 16 | 0 | 0 | 26 |
| **Task 3: Shop Drawings** | 4 | 20 | 0 | 0 | 0 | 24 |
| **Task 4: Fabrication Management** | 6 | 26 | 24 | 6 | 0 | 62 |
| **Task 5: Final Product Improvement** | 0 | 8 | 15 | 4 | 0 | 27 |
| **Task 6: Competition Preparation** | 0 | 4 | 0 | 0 | 0 | 4 |
| **Task 7: AISC Competition** | | | | | | 0 |
| **Task 8: Project Management** | 10 | 55 | 55 | 4 | 24 | 148 |
| Personnel Hours | 16 | 65 | 77 | 0 | 3 | 314 |

## B3: Emalee Sum of Hours

| | SENG | ENG | EIT | Lab | AA | Sum of Hours |
|---|---|---|---|---|---|---|
| **Task 1: Research** | 0 | 14 | 101 | 0 | 0 | 115 |
| **Task 2: Design and Analysis** | 0 | 14 | 101 | 0 | 0 | 80 |
| **Task 3: Shop Drawings** | 0 | 4 | 76 | 0 | 2 | 28 |
| **Task 4: Fabrication Management** | 2 | 4 | 25 | 0 | 0 | 10 |
| **Task 5: Final Product Improvement** | 0 | 0 | 0 | 0 | 0 | 0 |
| **Task 6: Competition Preparation** | 0 | 0 | 0 | 0 | 0 | 0 |
| **Task 7: AISC Competition** | | | | | | |
| **Task 8: Project Management** | 0 | 17 | 0 | 16 | 10 | 103 |
| Personnel Hours | 2 | 53 | 303 | 16 | 12 | 336 |

### B4: Steven Sum of Hours

| | SENG | ENG | EIT | Lab | AA | Sum of Hours |
|---|---|---|---|---|---|---|
| **Task 1: Research** | 0 | 8 | 86 | 0 | 0 | 94 |
| **Task 2: Design and Analysis** | 10 | 20 | 82 | 0 | 0 | 112 |
| **Task 3: Shop Drawings** | 0 | 2 | 19 | 0 | 0 | 21 |
| **Task 4: Fabrication Management** | 5 | 4 | 5 | 10 | 0 | 24 |
| **Task 5: Final Product Improvement** | 0 | 2 | 4 | 0 | 0 | 6 |
| **Task 6: Competition Preparation** | 0 | 0 | 0 | 0 | 0 | 0 |
| **Task 7: AISC Competition** | | | | | | |
| **Task 8: Project Management** | 4 | 5 | 17 | 1 | 3 | 30 |
| Personnel Hours | 19 | 41 | 213 | 11 | 3 | 287 |

## B5: Sam Sum of Hours

| | SENG | ENG | EIT | Lab | AA | Sum of Hours |
|---|---|---|---|---|---|---|
| **Task 1: Research** | 1 | 10 | 9 | 0 | 0 | 20 |
| **Task 2: Design and Analysis** | 0 | 20 | 43 | 8 | 0 | 71 |
| **Task 3: Shop Drawings** | 4 | 20 | 15 | 20 | 0 | 59 |
| **Task 4: Fabrication Management** | 2 | 4 | 10 | 14 | 0 | 30 |
| **Task 5: Final Product Improvement** | 0 | 0 | 1 | 2 | 0 | 3 |
| **Task 6: Competition Preparation** | 1 | 0 | 0 | 0 | 0 | 0 |
| **Task 7: AISC Competition** | | | | | | |
| **Task 8: Project Management** | 7 | 3 | 81 | 0 | 18 | 109 |
| Personnel Hours | 15 | 60 | 163 | 46 | 18 | 302 |

## Appendix C: Shop Drawings

# Appendix D: GANTT Chart
## D1: Proposed Schedule from Scope of Work

## D2: Current Schedule

## Appendix E: RISA Report

The RISA Report is approximately 115 pages. To save paper, it will not be printed for the purpose of this report.

# Appendix F: Connection Calculation Tables

## F1: Connection A Calculations

Connection A

| Property | Strength | Units | Section of Co | Demand |
|---|---|---|---|---|
| Tension Capacity | 29.53125 | kip | D1 | 5.67 kip |
| Bolt Shear Capacity | 15.075963 | kip | J3.4 | 3.639 kip |
| Block Shear | 12.164063 | kip | J6.3 | 3.639 kip |
| Tensile Rupture | 59.0625 | kip | J6.2 | 5.67 kip |
| bolt bearing strength | 40.5 | kip | J3-6a | |

| Tension Capacity | Value | Unit |
|---|---|---|
| T_n | 32.8125 | kip |
| F_y | 50 | ksi |
| A_g | 0.4375 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2 | plates |

| Specs of Plate | Value | Unit |
|---|---|---|
| Thickness | 0.125 | in |
| Depth | 3.5 | in |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| Bolt Hole Diamet | 0.4375 | in |
| A_g | 0.4375 | in^2 |
| A_net | 0.3281 | in^2 |

| Reduction Of Area | Value | Unit |
|---|---|---|
| T_n | 29.53125 | kip |
| F_u | 60 | ksi |
| A_n | 0.328125 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2 | plates |

| Specs of Bolt | Value | Unit |
|---|---|---|
| Diameter | 0.375 | in |
| A(NO THREAD) | 0.1104 | in^2 |
| Thread Height | 0.0469 | in |
| A(THREAD) | 0.1726 | in^2 |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| L | 2 | in |
| F_uv | 91 | ksi |

| Bolt Shear | Value | Unit |
|---|---|---|
| P_uv | 15.075963 | kip |
| F_uv | 91 | ksi |
| A(NO THREAD) | 0.1104466 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Planes of Shear | 2 | Planes |

| Block Shear | Value | Unit |
|---|---|---|
| P_nr (1) | 12.480469 | kip |
| P_nr (2) | 12.164063 | kip |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| A_gv | 0.09375 | in^2 |
| A_nt | 0.2304688 | in^2 |
| A_nv | 0.0664063 | in^2 |
| U_bs | 1 | All Cases |
| phi | 0.75 | LRFD |

| Tensile Rupture | Value | Unit |
|---|---|---|
| P_nt | 59.0625 | kip |
| A_e | 0.328125 | in^2 |
| F_u | 60 | ksi |
| U | 1 | |
| A_n | 0.328125 | in^2 |
| A_g | 0.4375 | in^2 |
| n_b | 4 | fasteners |
| phi | 0.75 | LRFD |

| Bolt bearing capacity | Value | Unit |
|---|---|---|
| known because the bolt diamter size is not in tab | | |

| bolt bearing strength J3-( | Value | Unit |
|---|---|---|
| R_n | 40.5 | kip |
| d_t | 0.375 | in |
| F_u | 60 | ksi |
| phi | 0.75 | LRFD |

## F2: Connection B Calculations

Connection B

| Property | Strength | Units | Section of Co | Demand |
|---|---|---|---|---|
| Tension Capacity | 17.578125 | kip | D1 | 5.67 kip |
| Bolt Shear Capacity | 15.075963 | kip | J3.4 | 3.639 kip |
| Block Shear | 6.1875 | kip | J6.3 | 3.639 kip |
| Tensile Rupture | 17.578125 | kip | J6.2 | 5.67 kip |
| bolt bearing strength | 40.5 | kip | J3-6a | |

| Tension Capacity | Value | Unit |
|---|---|---|
| T_n | 18.75 | kip |
| F_y | 50 | ksi |
| A_g | 0.25 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2 | plates |

| Specs of Plate | Value | Unit |
|---|---|---|
| Thickness | 0.125 | in |
| Depth | 2 | in |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| Bolt Hole Diamet | 0.4375 | in |
| A_g | 0.25 | in^2 |
| A_net | 0.1953 | in^2 |

| Reduction Of Area | Value | Unit |
|---|---|---|
| T_n | 17.578125 | kip |
| F_u | 60 | ksi |
| A_n | 0.1953125 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2 | plates |

| Specs of Bolt | Value | Unit |
|---|---|---|
| Diameter | 0.375 | in |
| A(NO THREAD) | 0.1104 | in^2 |
| Thread Height | 0.0469 | in |
| A(THREAD) | 0.1726 | in^2 |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| L | 2 | in |
| F_uv | 91 | ksi |

| Bolt Shear | Value | Unit |
|---|---|---|
| P_uv | 15.075963 | kip |
| F_uv | 91 | ksi |
| A(NO THREAD) | 0.1104466 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Planes of Shear | 2 | Planes |

| Block Shear | Value | Unit |
|---|---|---|
| P_nr (1) | 6.5039063 | kip |
| P_nr (2) | 6.1875 | kip |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| A_gv | 0.09375 | in^2 |
| A_nt | 0.0976563 | in^2 |
| A_nv | 0.0664063 | in^2 |
| U_bs | 1 | All Cases |
| phi | 0.75 | LRFD |

| Tensile Rupture | Value | Unit |
|---|---|---|
| P_nt | 17.578125 | kip |
| A_e | 0.1953125 | in^2 |
| F_u | 60 | ksi |
| U | 1 | |
| A_n | 0.1953125 | in^2 |
| A_g | 0.25 | in^2 |
| n_b | 2 | fasteners |
| phi | 0.75 | LRFD |

| Bolt bearing capacity | Value | Unit |
|---|---|---|
| known because the bolt diamter size is not in tab | | |

| bolt bearing strength J3- | Value | Unit |
|---|---|---|
| R_n | 40.5 | kip |
| d_t | 0.375 | in |
| F_u | 60 | ksi |
| phi | 0.75 | LRFD |

## F3: Connection C Calculations

Connection C

| Property | Strength | Units | Section of Co | Demand |
|---|---|---|---|---|
| Tension Capacity | 14.765625 | kip | D1 | 5.67 kip |
| Bolt Shear Capacity | 15.075963 | kip | J3.4 | 3.639 kip |
| Block Shear | 8.6132813 | kip | J6.3 | 3.639 kip |
| Tensile Rupture | 29.53125 | kip | J6.2 | 5.67 kip |
| Bolt Bearing Strength | 40.5 | kip | J3-6a | |

| Tension Capacity | Value | Unit |
|---|---|---|
| T_n | 16.40625 | kip |
| F_y | 50 | ksi |
| A_g | 0.21875 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2 | plates |

| Specs of Plate | Value | Unit |
|---|---|---|
| Thickness | 0.125 | in |
| Depth | 1.75 | in |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| Bolt Hole Diamet | 0.4375 | in |
| A_g | 0.21875 | in^2 |
| A_net | 0.1640625 | in^2 |

| Reduction Of Area | Value | Unit |
|---|---|---|
| T_n | 14.765625 | kip |
| F_u | 60 | ksi |
| A_n | 0.1640625 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2 | plates |

| Specs of Bolt | Value | Unit |
|---|---|---|
| Diameter | 0.375 | in |
| A(NO THREAD) | 0.1104466 | in^2 |
| Thread Height | 0.046875 | in |
| A(THREAD) | 0.1725728 | in^2 |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| L | 2 | in |
| F_uv | 91 | ksi |

| Bolt Shear | Value | Unit |
|---|---|---|
| P_uv | 15.075963 | kip |
| F_uv | 91 | ksi |
| A(NO THREAD) | 0.1104466 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Planes of Shear | 2 | Planes |

| Block Shear | Value | Unit |
|---|---|---|
| P_nr (1) | 8.6132813 | kip |
| P_nr (2) | 8.71875 | kip |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| A_gv | 0.1875 | in^2 |
| A_nt | 0.0976563 | in^2 |
| A_nv | 0.1601563 | in^2 |
| U_bs | 1 | All Cases |
| phi | 0.75 | LRFD |

| Tensile Rupture | Value | Unit |
|---|---|---|
| P_nt | 29.53125 | kip |
| A_e | 0.1640625 | in^2 |
| F_u | 60 | ksi |
| U | 1 | |
| A_n | 0.1640625 | in^2 |
| A_g | 0.21875 | in^2 |
| n_b | 4 | fasteners |
| phi | 0.75 | LRFD |

| Bolt bearing capacity | Value | Unit |
|---|---|---|
| nown because the bolt diamter size is not in tab | | |

| bolt bearing strength J3-( | Value | Unit |
|---|---|---|
| R_n | 40.5 | kip |
| d_t | 0.375 | in |
| F_u | 60 | ksi |
| phi | 0.75 | LRFD |

## F4: Connection D Calculations

Connection D

| Property | Strength | Units | Section of Co | Demand |
|---|---|---|---|---|
| Tension Capacity | 6.328125 | kip | D1 | 5.67 kip |
| Bolt Shear Capacity | 15.075963 | kip | J3.4 | 3.639 kip |
| Block Shear | 3.375 | kip | J6.3 | 3.639 kip |
| Tensile Rupture | 6.328125 | kip | J6.2 | 5.67 kip |
| bolt bearing strength | 40.5 | kip | J3-6a | |

| Tension Capacity | Value | Unit |
|---|---|---|
| T_n | 9.375 | kip |
| F_y | 50 | ksi |
| A_g | 0.125 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2 | plates |

| Reduction Of Area | Value | Unit |
|---|---|---|
| T_n | 6.328125 | kip |
| F_u | 60 | ksi |
| A_n | 0.0703125 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2 | plates |

| Bolt Shear | Value | Unit |
|---|---|---|
| P_uv | 15.075963 | kip |
| F_uv | 91 | ksi |
| A(NO THREAD) | 0.1104466 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Planes of Shear | 2 | Planes |

| Block Shear | Value | Unit |
|---|---|---|
| P_nr (1) | 3.6914063 | kip |
| P_nr (2) | 3.375 | kip |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| A_gv | 0.09375 | in^2 |
| A_nt | 0.0351563 | in^2 |
| A_nv | 0.0664063 | in^2 |
| U_bs | 1 | All Cases |
| phi | 0.75 | LRFD |

| Tensile Rupture | Value | Unit |
|---|---|---|
| P_nt | 6.328125 | kip |
| A_e | 0.0703125 | in^2 |
| F_u | 60 | ksi |
| U | 1 | |
| A_n | 0.0703125 | in^2 |
| A_g | 0.125 | in^2 |
| n_b | 2 | fasteners |
| phi | 0.75 | LRFD |

| Bolt bearing capacity | Value | Unit |
|---|---|---|
| known because the bolt diamter size is not in tab | | |

| bolt bearing strength J3-( | Value | Unit |
|---|---|---|
| R_n | 40.5 | kip |
| d_t | 0.375 | in |
| F_u | 60 | ksi |
| phi | 0.75 | LRFD |

| Specs of Plate | Value | Unit |
|---|---|---|
| Thickness | 0.125 | in |
| Depth | 1 | in |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| Bolt Hole Diamet | 0.4375 | in |
| A_g | 0.125 | in^2 |
| A_net | 0.0703125 | in^2 |

| Specs of Bolt | Value | Unit |
|---|---|---|
| Diameter | 0.375 | in |
| A(NO THREAD) | 0.1104466 | in^2 |
| Thread Height | 0.046875 | in |
| A(THREAD) | 0.1725728 | in^2 |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| L | 2 | in |
| F_uv | 91 | ksi |

## F5: Connection E Calculations

| Connection E | | | | |
|---|---|---|---|---|
| Property | Strength | Units | Section of Co | Demand |
| Tension Capacity | 45.28125 | kip | D1 | 5.67 kip |
| Bolt Shear Capacity | 15.075963 | kip | J3.4 | 3.639 kip |
| Block Shear | 5.2734375 | kip | J6.3 | 3.639 kip |
| Tensile Rupture | 64.6875 | kip | J6.2 | 5.67 kip |
| bolt bearing strength | 5.0625 | kip | J3-6a | |

| Tension Capacity | Value | Unit |
|---|---|---|
| T_n | 60.703125 | kip |
| F_y | 50 | ksi |
| A_g | 0.578125 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2.8 | plates |

| Specs of Plate | Value | Unit |
|---|---|---|
| Thickness | 0.25 | in |
| Depth | 2.3125 | in |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| Bolt Hole Diamete | 0.4375 | in |
| A_g | 0.578125 | in^2 |
| A_net | 0.359375 | in^2 |

| Reduction Of Area | Value | Unit |
|---|---|---|
| T_n | 45.28125 | kip |
| F_u | 60 | ksi |
| A_n | 0.359375 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2.8 | plates |

| Specs of Bolt | Value | Unit |
|---|---|---|
| Diameter | 0.375 | in |
| A(NO THREAD) | 0.1104466 | in^2 |
| Thread Height | 0.046875 | in |
| A(THREAD) | 0.1725728 | in^2 |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| L | 2 | in |
| F_uv | 91 | ksi |

| Bolt Shear | Value | Unit |
|---|---|---|
| P_uv | 15.075963 | kip |
| F_uv | 91 | ksi |
| A(NO THREAD) | 0.1104466 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Planes of Shear | 2 | Planes |

| Block Shear | Value | Unit |
|---|---|---|
| P_nr (1) | 7.3828125 | kip |
| P_nr (2) | 5.2734375 | kip |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| A_gv | 0.1875 | in^2 |
| A_nt | 0.0703125 | in^2 |
| A_nv | 0.078125 | in^2 |
| U_bs | 1 | All Cases |
| phi | 0.75 | LRFD |

| Tensile Rupture | Value | Unit |
|---|---|---|
| P_nt | 64.6875 | kip |
| A_e | 0.359375 | in^2 |
| F_u | 60 | ksi |
| U | 1 | |
| A_n | 0.359375 | in^2 |
| A_g | 0.578125 | in^2 |
| n_b | 4 | fasteners |
| phi | 0.75 | LRFD |

| Bolt bearing capacity | Value | Unit | J3.10 | bearing and tearout | |
|---|---|---|---|---|---|
| known because the bolt diamter size is not in tab | | | | 42,600 | kip |

| bolt bearing strength J3-( | Value | Unit |
|---|---|---|
| R_n | 5.0625 | kip |
| d | 0.375 | in |
| t | 0.125 | |
| F_u | 60 | ksi |
| phi | 0.75 | LRFD |

# F6: Connection F Calculations

Connection F

| Property | Strength | Units | Section of Co | Demand |
|---|---|---|---|---|
| Tension Capacity | 17.71875 | kip | D1 | 5.67 kip |
| Bolt Shear Capacity | 20.101284 | kip | J3.4 | 3.639 kip |
| Block Shear | 6.75 | kip | J6.3 | 3.639 kip |
| Tensile Rupture | 12.65625 | kip | J6.2 | 5.67 kip |
| bolt bearing strength | 40.5 | kip | J3-6a | |

| Tension Capacity | Value | Unit |
|---|---|---|
| T_n | 26.25 | kip |
| F_y | 50 | ksi |
| A_g | 0.25 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2.8 | plates |

| Specs of Plate | Value | Unit |
|---|---|---|
| Thickness | 0.25 | in |
| Depth | 1 | in |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| Bolt Hole Diamet | 0.4375 | in |
| A_g | 0.25 | in^2 |
| A_net | 0.140625 | in^2 |

| Reduction Of Area | Value | Unit |
|---|---|---|
| T_n | 17.71875 | kip |
| F_u | 60 | ksi |
| A_n | 0.140625 | in^2 |
| Reduction Factor | 0.75 | LRFD |
| Number of Plates | 2.8 | plates |

| Specs of Bolt | Value | Unit |
|---|---|---|
| Diameter | 0.375 | in |
| A(NO THREAD) | 0.1104466 | in^2 |
| Thread Height | 0.046875 | in |
| A(THREAD) | 0.1725728 | in^2 |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| L | 2 | in |
| F_uv | 91 | ksi |
| | | |

| Bolt Shear | Value | Unit |
|---|---|---|
| P_uv | 20.101284 | kip |
| F_uv | 91 | ksi |
| A(NO THREAD) | 0.1104466 | in^2 |
| Reduction Factor | 1 | LRFD |
| Planes of Shear | 2 | Planes |

| Block Shear | Value | Unit |
|---|---|---|
| P_nr (1) | 7.3828125 | kip |
| P_nr (2) | 6.75 | kip |
| F_y | 50 | ksi |
| F_u | 60 | ksi |
| A_gv | 0.1875 | in^2 |
| A_nt | 0.0703125 | in^2 |
| A_nv | 0.1328125 | in^2 |
| U_bs | 1 | All Cases |
| phi | 0.75 | LRFD |

| Tensile Rupture | Value | Unit |
|---|---|---|
| P_nt | 12.65625 | kip |
| A_e | 0.140625 | in^2 |
| F_u | 60 | ksi |
| U | 1 | |
| A_n | 0.140625 | in^2 |
| A_g | 0.25 | in^2 |
| n_b | 2 | fasteners |
| phi | 0.75 | LRFD |

| Bolt bearing capacity | Value | Unit |
|---|---|---|
| nown because the bolt diamter size is not in tab | | |

| bolt bearing strength J3-( | Value | Unit |
|---|---|---|
| R_n | 40.5 | kip |
| d_t | 0.375 | in |
| F_u | 60 | ksi |
| phi | 0.75 | LRFD |