Student Steel Bridge Competition

Revised 5/7/24

Location

Garland Gregory Hide-A-Way Park

Ruston, Louisiana 71270

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Table of Contents

1.0 Project Introduction 7
1.1 Problem Statement 7
1.1.1 Project Objective 7
1.1.2 Relevance 7
1.1.3 Project Location 7
2.0 Project Background Research 9
2.1 Competition Overview 9
2.2 Bridge Dimension Limits 10
2.3 Material and Component Constraints 11
2.4 Construction Regulations 11
2.5 Load Requirements 12
2.6 Bridge Design Research 14
2.6.1 Bridge Type Research 14
2.6.2 Stringer Design Research 14
2.7 Connection Design Research 15
3.0 Preliminary Analysis and Design 15
3.1 Member Design and Analysis 15
3.1.1 Stringer Design and Analysis 15
3.1.2 Truss Design and Analysis 17
3.2 Material Specifications 18
3.3 Connection Design 18
3.4 Final Design Recommendation 19
4.0 Final Analysis and Design 20
4.1 Design Modeling 20
4.2 Strength Analysis and Design 21
4.3 Serviceability Analysis and Results 25
4.4 Connection Analysis and Design 27
4.5 Final Design and Analysis Summary 30
5.0 Bridge Production 30
5.1 Shop Drawings 30
5.2 Fabrication Coordination 32
5.3 Mentee Collaboration 33
6.0 Assembly 33
6.1 Initial Bridge Assembly 33

6.2 Design Modifications 34
6.3 Competition Assembly Practice 34
7.0 Competition 36
7.1 Competition Display 36
7.2 Competition Results 36
7.3 Lessons Learned 36
8.0 Impact Analysis 37
9.0 Summary of Engineering Work 39
9.1 Proposed Schedule vs Adjusted Schedule 39
9.2 Cost of Work Scheduled vs Cost of Actual Work Performed 41
10.0 Conclusion 43
11.0 References 45
Appendices 46
Appendix A: Connection Hand Calculations 46

Table of Tables

Table 2-1 Load Combinations [5] 12
Table 3-1 Category Descriptions (Stringer Design) 15
Table 3-2 Symbol Descriptions 16
Table 3-3 Qualitative Decision Matrix (Stringer) 16
Table 3-4 Category Descriptions (Bridge Design) 17
Table 3-5 Qualitative Decision Matrix (Bridge) 17
Table 3-6 Material Standard Sizes 18
Table 4-1 Design Load Placement Summary 21
Table 4-2 Member Loading Summary 21
Table 4-3 Material Specifications 23
Table 4-4 Lateral Bracing Internal Force Summary 24
Table 4-5 Vertical Deflection Summary 25
Table 4-6 Lateral Deflection Summary 26
Table 4-7 Connection Analysis Summary 30
Table 4-8 Final Design Summary 30
Table 7-1 Competition Results 36
Table 8-1 Impact Analysis Summary 38
Table 8-2 Impact Analysis Scoring 39
Table 9-1 Proposed Hours 41
Table 9-2 Actual Hours 41
Table 9-3 Proposed Cost of Work 42
Table 9-4 Actual Cost of Work 43

Table of Figures

Figure 1-1 Project Location Map [1] 8

Figure 1-2 Construction Location Map [2] ... 8
Figure 2-1 ISWS Bridge Construction [3] ... 9
Figure 2-2 Vertical Load Test 2009 Regional Competition [4] ... 9
Figure 2-3 Bridge Envelope Profile View [5] ... 10
Figure 2-4 Bridge Envelope Front View [5] ... 10
Figure 2-5 Stringer Template [5] ... 10
Figure 2-6 Construction Site Plan View [5] ... 11
Figure 2-7 Construction Zone and Staging Yard Details [5] ... 12
Figure 2-8 Vertical Load Test Plan and Profile View [5] ... 13
Figure 2-9 Lateral Load Test Plan View [5] ... 13
Figure 2-10 Bridge Type Considerations [7] ... 14
Figure 2-11 Stringer Configurations [7] ... 14
Figure 3-1 Stringer Location [7] ... 15
Figure 3-2 Plate-on-Plate Connection ... 18
Figure 3-3 Angle-on-Angle Connection ... 18
Figure 3-4 Elevation View of 2-D Sketch ... 19
Figure 3-5 Side View of 2-D Sketch ... 19
Figure 3-6 Top View of 2-D Sketch ... 19
Figure 4-1 Initial RISA Model ... 20
Figure 4-2 Final RISA Model ... 20
Figure 4-3 Member Callouts ... 22
Figure 4-4 Tension Forces ... 22
Figure 4-5 Compression Forces ... 22
Figure 4-6 RISA 3D Bridge Plan View ... 23
Figure 4-7 Axial Behavior Result ... 24
Figure 4-8 RISA 3D Vertical Deflected Shape ... 25
Figure 4-9 Lateral Deflected Shape ... 26
Figure 4-10 RISA 3D Code Check ... 26
Figure 4-11 Eccentric Reaction and Demand Concept- ... 27
Figure 4-12 Connection Locations ... 28
Figure 4-13 Stringer Connection Illustration ... 29
Figure 4-14 Column/Lateral Bracing Connection Illustration ... 29
Figure 4-15 Universal Substructure Connection Illustration ... 29
Figure 5-1 Bridge Plan and Profile View ... 31
Figure 5-2 Example Detail Drawing ... 31
Figure 5-3 Example Part Drawing ... 32
Figure 5-4 Stringer Fabrication Process [9] ... 33
Figure 6-1 Completely Constructed Bridge ... 34
Figure 6-2 Construction Site CEMCEE FIELD Station ... 35
Figure 6-3 Substructure Construction Illustration ... 35
Figure 7-1 Adjusted RISA Model ... 37
Figure 9-1 Proposal Schedule ... 40
Figure 9-2 Actual Schedule ... 40

Appendices

Appendix A: Hand Calculations 42

Acknowledgments

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We also wanted to acknowledge our grading instructor Robin Tuchscherer. Dr. T had always been ready to meet with our team regarding any matter we had, even outside the scope of the reports and presentations. He would always give us helpful hints and ideas that we would ponder on when discussing the design of our bridge. As well as this he always pushes our team to think deeper than we originally thought we knew. Some meetings were demoralizing but whenever our team met with Dr. T, we always knew the next steps for the project. Thank you, Dr. T!

We also wanted to acknowledge our client, Mark Lamer. However, for the sake of the project he is considered our client. As a faculty advisor, he was willing to meet and begin setting up next steps for our team to begin the technical work for the project. He has also provided the team with a lot of input from the previous year's team which helped the team understand what needed to be fixed to make a better bridge this year. Thank you, Mark!

We also wanted to acknowledge our lab manager, Adam Bringhurst. Adam was a critical component in getting access to the CECMEE field station. Within a matter of weeks, he had helped us finish our lab safety binder, got our keys to go to the field station, and begin working on the bridge. Without his help we could not have gotten to the fabrication stage of the project as quickly as we did. Thank you, Adam!

Finally, we wanted to acknowledge all our sponsors, Page Steel, and the welding class of Flagstaff High School. Page steel was gracious enough to donate all the steel to the project and the welding class at Flagstaff High School was willing to put in the extra time to help us weld our bridge. Without their help we would not be able to build our bridge at all. Thank you, Page Steel, and the welding class of Flagstaff High School.

1.0 Project Introduction

1.1 Problem Statement

The Ponderosa Steel Jacks have been tasked with developing and constructing a 1:10 scale model bridge. The development of this scale model bridge is intended to represent a 1.25-mile bridge that will be constructed in the future at Hideaway Disc Golf Course. The bridge will be designed as a pedestrian bridge with added use for utility vehicles. With this information in mind the Ponderosa Steel Jacks will be competing in American Institute of Steel Construction's (AISC) feasibility study to identify the best design for the new bridge. The scale model of the bridge will need to be erected over a staging area that will demonstrate the strength-to-weight ratio of the bridge, rapid construction time, and design versatility. During the design, the Ponderosa Steel Jacks will need to keep the surrounding environment in mind as the bridge must match the surrounding aesthetics of the park.

1.1.1 Project Objective

The Ponderosa Steel Jacks aim to develop and construct a scale model of the pedestrian bridge to compete in the AISC feasibility study. The bridge will be designed to support a pre-determined vertical and lateral load evaluated during the competition. The bridge design will support its self-weighted and additional loading applied to parts of the bridge at the ISWS competition. The competition will be hosted by the American Society of Civil Engineers (ASCE) for the Intermountain Southwest Symposium (ISWS) held at Utah State University from Aprill 11^{th}–Aprill 14^{th}.

1.1.2 Relevance

This project provides students with an intercollegiate challenge with practical and firsthand steel design and construction. The Student Steel Bridge Competition (SSBC) team will be responsible for designing a bridge under aesthetic, safety, fabrication, and serviceability concerns. This scale model will be used as a reference for full-scale construction, and to represent the potential success of a full-scale model design. The competition also aims to give each team real-world experience in structural engineering.

1.1.3 Project Location

The full-scale bridge design is intended to be erected in Ruston, Louisiana as shown in Figure 1-1 below. As the full-scale model will not be addressed, the scale model will be constructed at Utah State University to undergo different load testing and construction speed competition. The construction location is shown in Figure 1-2.

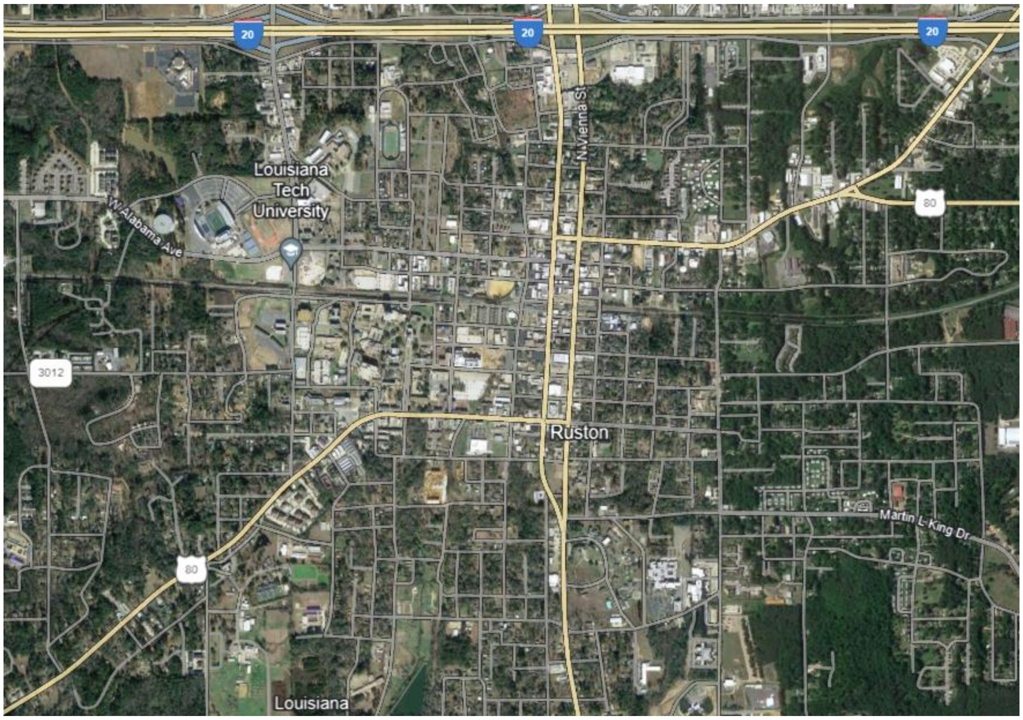


Figure 1-1 Project Location Map [1]

Figure 1-2 Construction Location Map [2]

2.0 Project Background Research

2.1 Competition Overview

The competition will last two days. The first day of competition will involve the teams assembling their bridges to display for the judges. Additionally, during this day, the teams will present a poster board that display a variety of information including a critical member demonstrating the moment capacity of the member, project sponsors, bridge design decisions, etc. The second day of the competition will include a timed construction and load testing. The timed construction and load testing will occur in a sequential order in which one team will begin then the following team will begin upon the previous team's completion of construction. Loading will occur in the same format directly after the bridge's construction. Figure 2-1 shows a picture of the construction day at the 2023 competition hosted by the University of Nevada, Reno. Figure 2-2 illustrates what is to be expected of the vertical load testing during competition.

Figure 2-1 ISWS Bridge Construction [3]

Figure 2-2 Vertical Load Test 2009 Regional Competition [4]

2.2 Bridge Dimension Limits

The envelope for the scale model bridge has been defined by the AISC rules as 21-ft. long and 2.5-ft. high. Within the construction zones footings must be maintained within a 1-ft. by 1-ft. square. Further dimensions can be identified in Figures 2-3, 2-4, and 2-5 below. The figures identify the bridge envelope to show the bounds of the dimensions allowed. Another constraint regards dimensions for individual members which have been limited to fit within a 3.5-ft. long, 6-inch tall, and 4-inch-wide box [5].

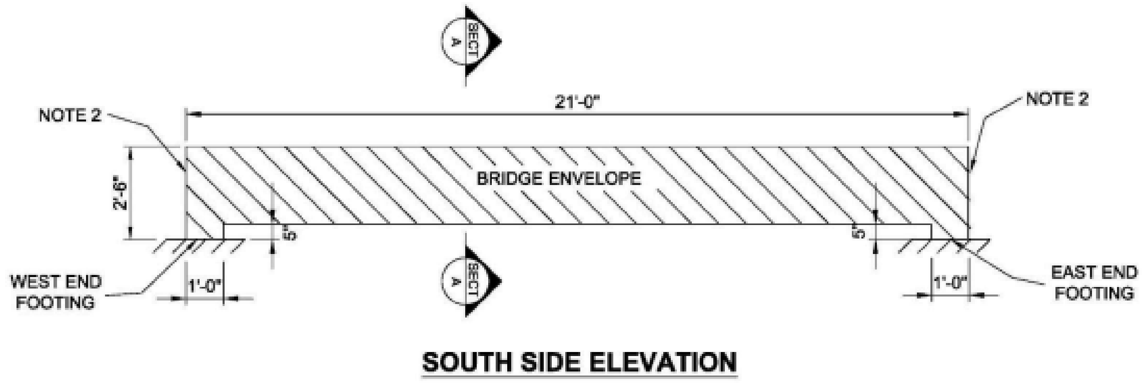


Figure 2-3 Bridge Envelope Profile View [5]

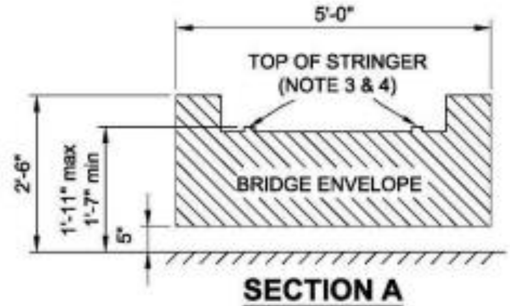


Figure 2-4 Bridge Envelope Front View [5]

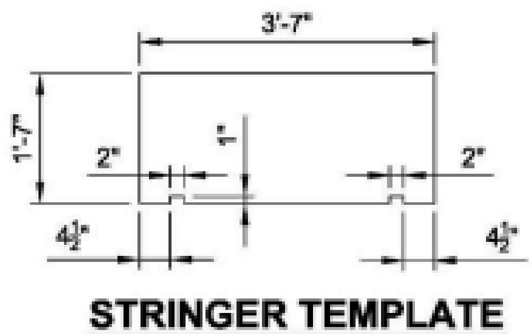


Figure 2-5 Stringer Template [5]

Notes for Figures 2-2,2-3, and 2-4

1. DRAWINGS ARE NOT TO SCALE
2. NO PART OF THE BRIDGE SHALL EXTEND AWAY FROM THE RIVER BEYOND THE CONSTRUCTION ZONE BOUNDARIES
   TOPS OF STRINGERS SHALL BE AT LEAST 20 FT. LONG AND AT MOST 21 FT. LONG
3. BRIDGE SHALL PROVIDE A STRAIGHT, CLEAR DECKING SUPPORT LOCATION AND PASSAGEWAY

2.3 Material and Component Constraints

The bridge must be made entirely of steel. According to the rules, the steel must be "strongly magnetic." Additionally, the bridge must be made of different structural components. The bridge must have two stringers. This will be used to place the 3 ft. 6 in. decking that will be provided at the competition and used to apply load to the bridge. The decking will not be anchored, and the structural design must not inhibit the placement of the decking [5].

2.4 Construction Regulations

During construction, the Ponderosa Steel Jacks will be supervised and evaluated on the ability to construct within the construction site plan along with various limitations with associative violations. The construction regulations determine placements and capabilities of three main positions including "Captain," "Builders," and "Barges." The builders include the entire team working on the construction, a captain is specially delegated to be the point of contact for the judges, and the barges are two builders delegated to begin and end at the dock but during construction must remain in the river portion. A site plan as well as plan details have been identified in Figure 2-6 and Figure 2-7, respectively [5].

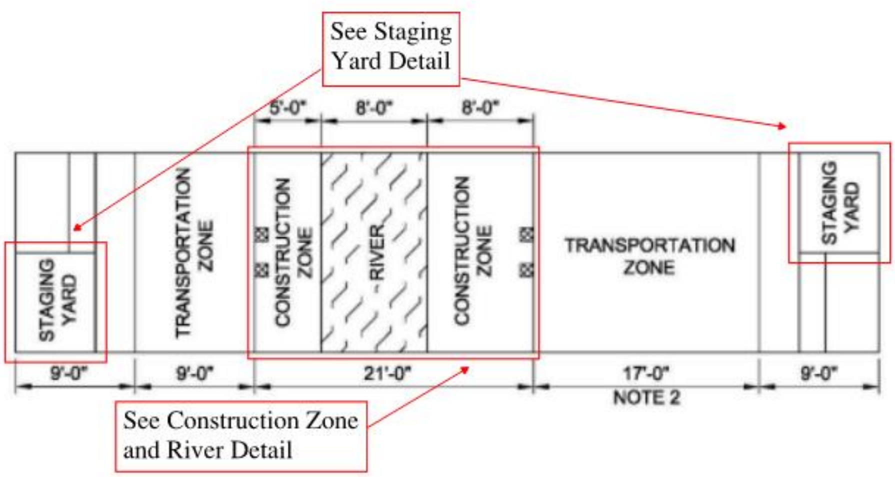


Figure 2-6 Construction Site Plan View [5]

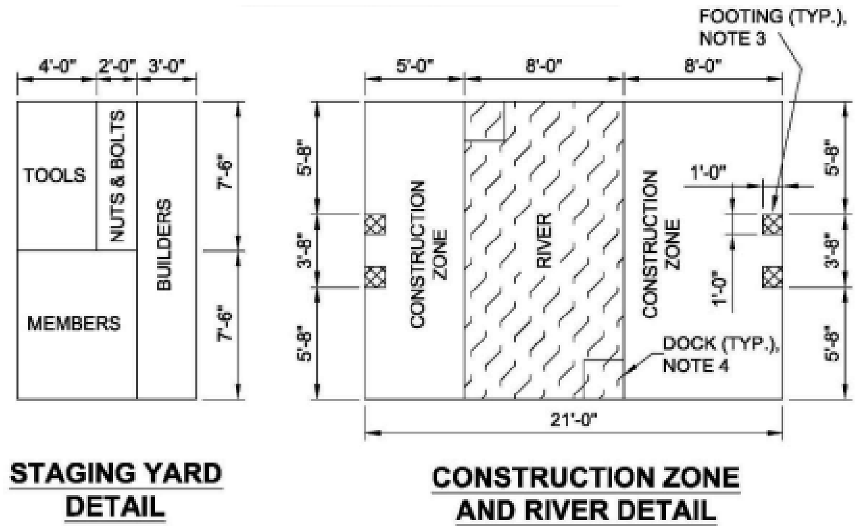


Figure 2-7 Construction Zone and Staging Yard Details [5]

Notes for Figures 2-5 and 2-6:

1. DRAWINGS ARE NOT TO SCALE
2. DIMENSION MAY BE REDUCED TO FIT LOCAL CONDITIONS
3. ALL FOUR FOOTINGS ARE 1' BY 1'
4. ALL DOCK AREAS ARE 2' BY 2'

Section 10 of the AISC rules determines the general regulations, pre-construction conditions, safe construction practices, accidents, construction site layout, construction start practices, time factors, and construction finish procedures [5]. The SSBC team will be responsible for defining and identifying easy and safe practices to ensure a successful time competition. This competition will show that the bridge will be easily constructed when a full-scale model is developed.

2.5 Load Requirements

Load testing is a procedure conducted at the competition to evaluate the vertical and lateral stability of the bridge. The test will be randomly conducted on the bridge dependent on a dice roll. The matrix for that roll and the combination of dimensions for the location of the tests can be seen in Table 2-1 below [5]. The L1, L2, and S dimensions can be seen on the lateral and vertical load test plans in Figures 2-8 and 2-9 below.

Table 2-1 Load Combinations [5]

N	L1	L2	S
1	4'-6"	9'-0"	7'-6"
2	6'-0"	12'-0"	9'-0"
3	7'-0"	13'-0"	9'-0"
4	7'-6"	11'-6"	9'-0"
5	8'-6"	12'-6"	10'-6"
6	10'-0"	14'-0"	10'-6"

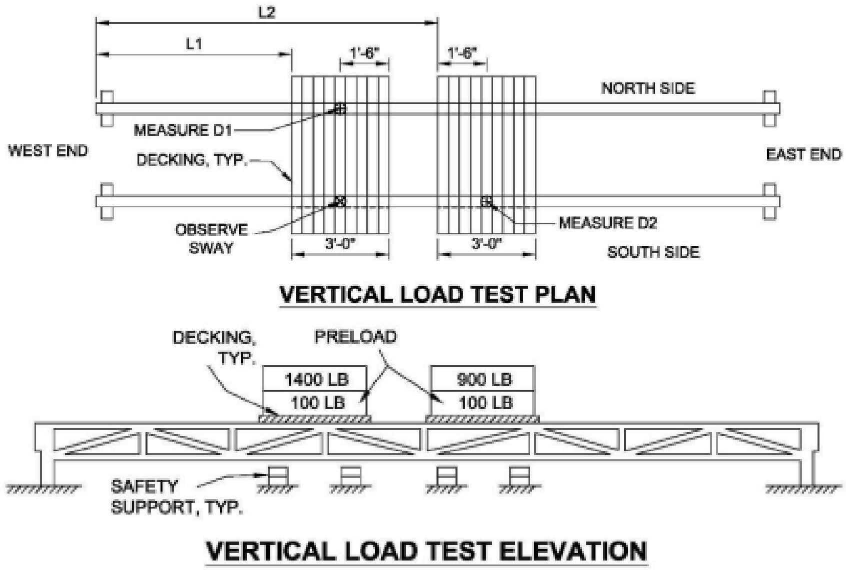


Figure 2-8 Vertical Load Test Plan and Profile View [5]

Note for Figure 2-7:

1. DRAWINGS ARE NOT TO SCALE
2. DECKING LOCATIONS “L1” AND “L2” ARE DETERMINED BASED ON THE DIE ROLL AND ARE THE SAME FOR ALL BRIDGES
3. DECKING LOCATION “L1” AND “L2” ARE MEASURED FROM THE WEST END OF THE NORTH SIDE STRINGER
4. SAFETY SUPPORTS ARE REQUIRED UNDER BOTH DECKING UNITS AT ALL TIMES
5. THE 100 LBS. OF PRELOAD ARE PLACE FIRST ON EACH DECKING UNIT, FOLLOWED BY INITIALIZATION OR INITIAL READINGS OF DEFLECTION AND SWAY MEASURING DEVICES
6. THE PRELOAD REMAINS IN PLACE, AND 1400 LBS OF LOAD IS PLACED ON THE DECKING UNIT LOCATED AS “L1”, FOLLOWED BY 900 LBS OF LOAD PLACED ON THE DECKING UNIT LOCATED AT “L2”
7. LOACTIONS OF DEFLECTION AND SWAY MEASURNMENTS ARE SPECIFIC TO THE NORTH AND SOUTH SIDES
8. DEFLECTIONS D1, D2, AND SWAY ARE MOINITIORED CONTINUOUSLY

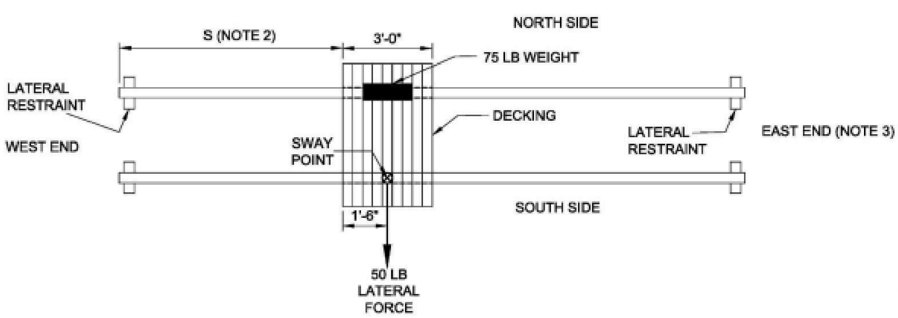


Figure 2-9 Lateral Load Test Plan View [5]

Notes for Figure 2-8:

1. DRAWING NOT TO SCALE
2. DECKING LOCATION "S" IS DETERMINED BASED ON THE DIE ROLL AND IS THE SAME FOR ALL BRIDGES
3. EAST END OF THE BRIDGE IS DETERMINED BASED ON A RANDOMIZING PROCESS FOR EACH BRIDGE
4. LOCATIONS OF LATERAL PULL, LATERAL RESTRAINT, AND SWAY MEASUREMENTS ARE SPECIFIC TO THE NORTH AND SOUTH SIDE OF THE BRIDGE

2.6 Bridge Design Research

2.6.1 Bridge Type Research

The research related to the truss design is based on the premise of using either an under-truss (arch) or a typical truss bridge instead of a beam bridge. At previous ISWS competitions multiple teams had used a beam bridge in which the three podium finishers from the 2021 regional competition had all used a beam bridge [6]. Northern Arizona University's (NAU) team last year had constructed a truss bridge. This bridge had failed due to excessive swaying. However, with this in mind the team believed that the previous team's ideas should be taken into consideration. An arch bridge had also been taken into consideration due to its ability to transfer the vertical load into horizontal loads which would feed into the structure's columns. Figure 2-10 illustrates the bridge types under consideration.

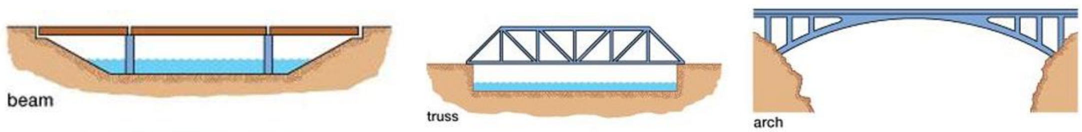


Figure 2-10 Bridge Type Considerations [7]

2.6.2 Stringer Design Research

In consideration with the stringers the team had primarily focused on using stringers that had worked for the teams from the 2021 competition. Many of the teams at the 2021 competition had used beam bridges while the top 3 finishers had all used a beam bridge. The use of beam bridges means the force being applied had to be supported by the stringers without assistance from an additional frame. With this information in mind the team elected that using the previous winners' stringer configuration would provide the best results for this upcoming competition. The primary stringer frames used in the 2021 competition included a through warren truss, a quadrangular warren truss, and a camel back truss. Figure 2-11 shows the types of stringer configurations.

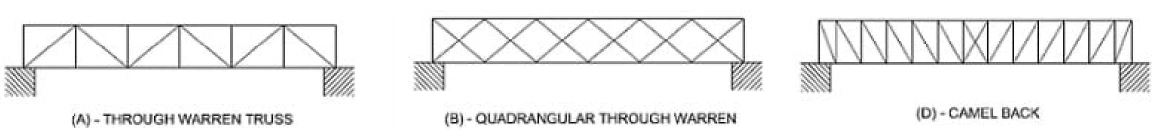


Figure 2-11 Stringer Configurations [7]

2.7 Connection Design Research

The team began researching connections through the AISC rules and constraints. In previous years, teams used slotted connections to cut down on connection time and resist the internal moments applied to the bridge. However, this year AISC had included rules prohibiting connections that used interlocking mechanisms, camshafts, dovetails, etc. [5]. The AISC rules state that interlocking connections are prohibited and that any bolt must go through at least one faying surface but no more than two faying surfaces [5]. The team proceeded to connection design understanding that simple connections using two plates were the most probable connections to be used. Alongside this, after communicating with the team's client, a rough estimate of 30 seconds should be added for each connection location. With this information in mind the team aimed to limit the number of connections throughout the bridge.

3.0 Preliminary Analysis and Design

3.1 Member Design and Analysis

3.1.1 Stringer Design and Analysis

The initial process for determining the bridge design was developing a qualitative analysis for each part of the bridge. The team first began by developing a qualitative analysis for the stringer portion of the bridge. This portion would consist of the main body of the bridge as seen in Figure 3-1.

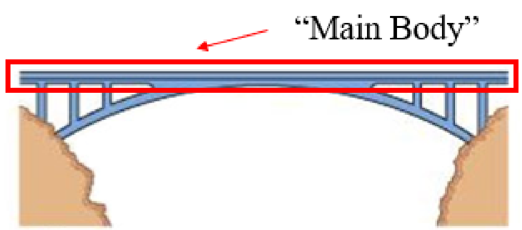


Figure 3-1 Stringer Location [7]

The qualitative analysis would consider three categories structural strength, ease of fabrication, and aesthetics. Then to determine how effective each design would be comparatively to one another a "+," "-", or neutral symbol was used. Further descriptions of the categories and symbol descriptions can be seen in Table 3-1 and 3-2.

Table 3-1 Category Descriptions (Stringer Design)

Category	Description
Structural Strength	The stringer design will be able to resist the internal reactions applied by the external loading. The design will also reduce the total aggregate deflection caused by the loading.
Ease of Fabrication	The truss design will be able to be easily replicated in which the cuttings and welding of the stringer can be done in mass with little to no design modification.
Aesthetics	The truss design will be pleasing to the eye and demonstrates visual structural integrity.

Table 3-2 Symbol Descriptions

Rating	Description
+	The bridge aspect, by visual inspection, can perform better than the other bridge designs.
	The bridge aspect, by visual inspection, is adequate in performance. The bridge aspect can perform better than some bridges but worse than others.
-	The bridge aspect, by visual inspection, does not perform as optimally as the other two bridges. The bridge aspect may be able to perform the task similarly to the other bridge aspects, however it is less adequate by comparison.

The qualitative decision matrix in Table 3-3 below shows the analysis of the three stringer designs.

Table 3-3 Qualitative Decision Matrix (Stringer)

Category	Camelback Truss	Through Warren Truss	Quadrangular Truss
Structural Strength	+	-	
Ease of Fabrication		+	-
Aesthetics	+	-	
Total	++	-	-

The bridge will primarily be designed around vertical loading. Based on the bridge's demands, the team expects the top of the stringers to be in compression while the bottom of the bridge in tension. Based on the design of a camelback truss the internal supports can be adjusted to support either the top or bottom of the bridge depending on where the demand is in the stringer. Alongside this the team determined that the fabrication would be easier than the quadrangular truss as less cutting would be required however the camelback would require more fabrication time than the through warren truss due to the incorporation of the vertical members. Finally, aesthetically the team believed the camelback truss provided a clean and symmetrical shape the visually looked structurally sound. Based on this information and the qualitative decision matrix the team will be moving forward with the camelback truss for the stringer design.

A qualitative decision matrix was used to determine the stringer configuration. Category descriptions for the decision matrix's criteria are in Table 3-4. This decision matrix uses the same point values as described in Table 3-2. The qualitative decision matrix can be found in Table 3-5.

Based on the decision matrix the team has decided to use a camelback truss for the stringer design. The bridge will experience a vertical live load which will put the stringers in tension. The team decided that the camelback truss would work best. This is because the vertical loading will cause additional loading in the bottom members of the stringer which will be in tension. The camelback truss will resist the internal forces and reduce the overall weight of the structure. The camelback truss will also be easy to fabricate as the pieces can easily be replicated. The truss as well is aesthetically pleasing and provides a sense of structural integrity.

3.1.2 Truss Design and Analysis

The initial process for determining the bridge design was developing a qualitative analysis to determine the design the team would pursue. This qualitative analysis would consider three categories: ease of construction, aesthetics, and structural design. Further descriptions of the categories can be seen in Table 3-4. The team used the same symbol notation as in Table 3-2.

Table 3-4 Category Descriptions (Bridge Design)

Category	Description
Ease of Construction	This category describes the ability of the bridge to be constructed as quickly as possible. The bridge shows that the possible number of connections is limited, and the connections created can be easily duplicated. The bridge itself will also have a lower number of members to reduce the construction.
Aesthetics	The bridge is pleasing to the eye and fits in with the aesthetics of the project location.
Structural Design	The bridge from a structural integrity view can withstand the anticipated applied load effectively. The bridge will also decrease the internal force applied to each member.

The Qualitative decision matrix in Table 3-5 below shows the analysis of three bridge types.

Table 3-5 Qualitative Decision Matrix (Bridge)

Category	Beam Bridge	Arch Bridge	Warren Bridge
Ease of Construction	+		-
Aesthetics	-	+	
Structural Design	-		+
Total	-	+	

When looking at the bridges the team had chosen to use an arch bridge. For the ease of construction, the team felt that the arch bridge would be more difficult to construct than the beam bridge however due to the ability to build the bridge from the bottom up the arch bridge would be easier to construct than the warren bridge. Alongside this the team believed the bridge looked the most aesthetically pleasing and would fit more into the project location compared to the other bridges. Finally, the team believed that the warren bridge would offer the best structural design as the stringers would be put into tension while the top of the bridge would be put into compression. Understanding this, the team also determined that the arch bridge could offer more support than the bream bridge. After reviewing the decision matrix, the team has decided to use an arch bridge design.

3.2 Material Specifications

The material availability for this project is limited to strongly magnetic steel. The team determined that the steel used cannot contain additional zinc, nickel, or austenite. The team had chosen to use Hollow Structure Sections (HSS) tubing and piping as well as stainless steel plates. Stainless steel is not typically considered a strongly magnetic metal steel however depending on the concentration of iron in the steel, stainless steel can be considered strongly magnetic. The following materials should be compliant with the AISC rules. Table 3-6 lists the standard sizes for the materials chosen by the team. Any modifications made to the chosen materials will be made evident in section 5.

Table 3-6 Material Standard Sizes

Material	Standard Sizes
HSS Rect. Tubing	1/2"x1”x.188” to 12”x20”x1/2”
HSS Square Tubing	1"x1”x.188” to 20”x20”x1/2”
Galvanized Steel	4”x8” to 8”x20”
HSS Piping	1/4” to 24” Outside diameter

3.3 Connection Design

Based on the rules set in place by AISC, the team had developed two ideas for the connections. The first idea was a simple plate on plate connection. This connection would have holes drilled into each side which would then be connected using a steel bolt and nut. The team initially thought there would be some construction issues that would arise due to the use of the plate-on-plate connections. This then prompted an alternative design which would use angle-on-angle connections. Using this connection, the team determined that there would be some plates welded to a member while an angled piece would stick out of the member. An example of the connection concepts can be seen in Figure 3-2 and Figure 3-3. After closer inspection of the AISC rules the team determined the plate-on-plate connection was feasible without limitation from the AISC rules. However, this will be further expanded upon in section 4.

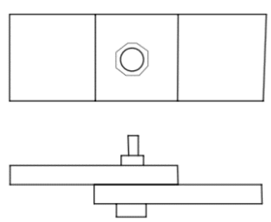


Figure 3-2 Plate-on-Plate Connection

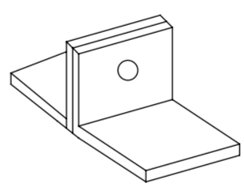


Figure 3-3 Angle-on-Angle Connection

3.4 Final Design Recommendation

Based on the preliminary analysis and design task, the team selected an arch bridge with stringers made of camelback trusses for final analysis and design, details of which are presented in Section 4. The internal supports for the stringers are anticipated to be welded while the end of the stringers will be connected. The final design should be considered a string of connected members than an actual truss system. For the connections, the team had decided to begin designing and analyzing angle on angle connections with the assumption that the plate-on-plate connections were not feasible. A preliminary 2-D design of the bridge can be seen in Figure 3-4, Figure 3-5, and Figure 3-6 below with a more fine-tuned design determined during Final Analysis and Design.

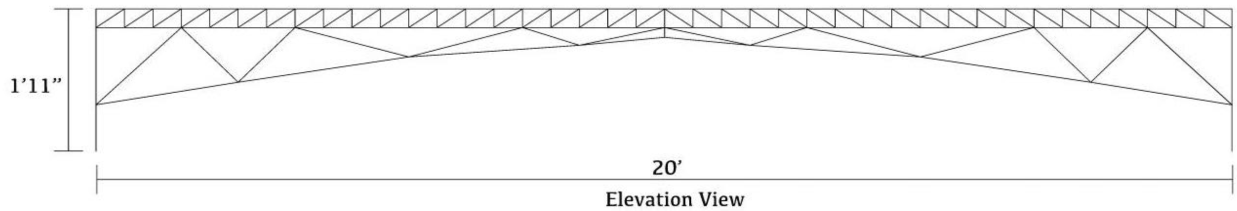


Figure 3-4 Elevation View of 2-D Sketch

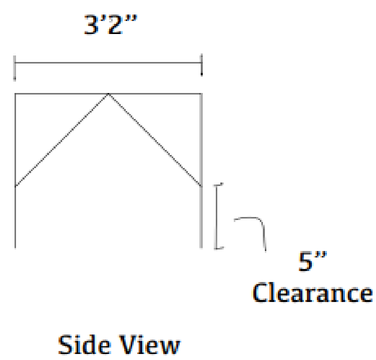


Figure 3-5 Side View of 2-D Sketch

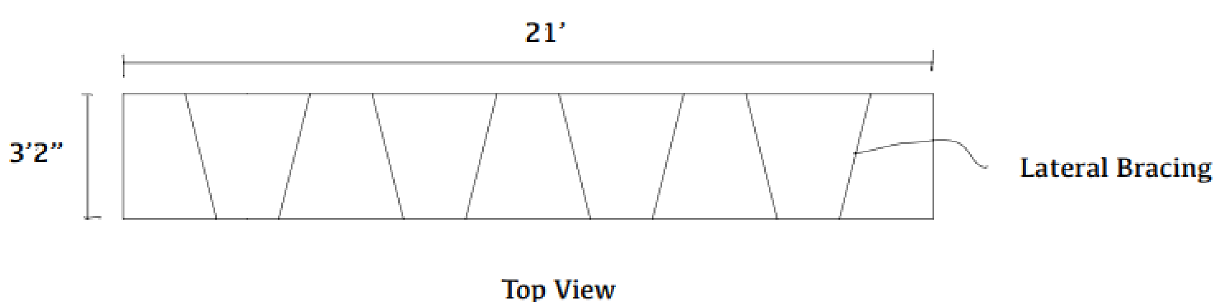


Figure 3-6 Top View of 2-D Sketch

4.0 Final Analysis and Design

4.1 Design Modeling

To begin designing the bridge, the team developed an initial model using RISA-3D. Figure 4-1 illustrates the initial model. As seen from the figure, the initial model is quite large and closely reflects what a typical arch bridge would look like. The initial design determined the feasibility of the bridge and identified any major deficiencies in the design.

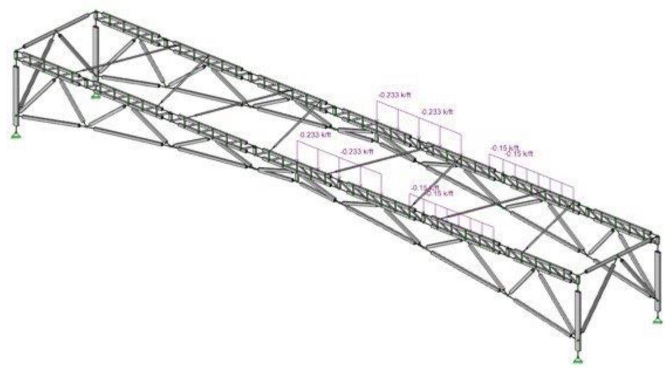


Figure 4-1 Initial RISA Model

Upon review of the initial design, the team adjusted the bridge and the connections within the design. The initial model had I and J releases at each connection point which released the moment at each end of the members. By releasing the moment at each connection point, this modeled typical pin connections that would release moment in the y direction. With this I and J release methodology the model became unstable, and after attempting to solve in RISA 3D multiple stability errors were produced. After reviewing the model, the team's technical advisor identified certain degrees of freedom which should have been "fixed" and these were changed accordingly. After reviewing the bridge envelope, the team determined that the initial model was too large. The team then went in and reduced the height of the bridge which required adjusting the lengths of several members in the substructure. Figure 4-2 illustrates the final bridge design.

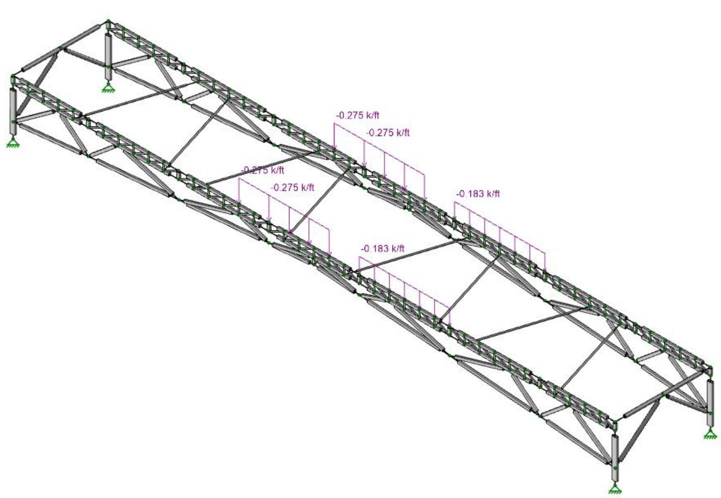


Figure 4-2 Final RISA Model

In the final design, the bridge was modeled using frictionless hinge connections. Although the model does not accurately reflect the roller behavior anticipated at the competition, the team continued to use pin supports throughout the modeling process. Further elaboration of this error can be found in Section 7.

Another element of the analysis included creating different load cases for the bridge. The load cases are applied at various locations along the bridge's length as per Table 2-1. The load cases were also set up to analyze the bridge using Load and Resistance Factor Design (LRFD). In addition to the load combination, load factors of 1.2 and 1.6were applied to the dead load and live load. The specific LRFD load combination and factors chosen produced the greatest demand scenario. Ten load cases were produced for both vertical and lateral loading. After setting up the load cases for both the vertical and lateral loading a, “batch solution” was solved for. The batch solution solved all load cases applied to the bridge all at once.

After searching through the node deflection results, the team found that load case 4 would produce the highest deflection in the bridge. Load case 4 produced the greatest vertical deflection. The team also found that the greatest lateral deflection would occur at load case 1. A summary of these load placements can be found in Table 4-1. The red text indicates the load placement location that creates the greatest vertical and horizontal deflection.

Table 4-1 Design Load Placement Summary

N	L1	L2	S	Design Deflection
1	4’-6”	9’-0”	7’-6”	Horizontal
3	7’-0”	13’-0”	9’-0”	Vertical

4.2 Strength Analysis and Design

After developing a working RISA-3D model, the team began analyzing the data to determine which members had the highest internal forces. Table 4-2 shows a summary of the highest calculated internal forces for the bridge members. Within the table the demand to capacity ratio is listed. Under the demand to capacity ratio any value less than one was deemed safe. Additional imagery for the loading relating to the compression and tension code checks will be provided in Section 4.3. The superstructure includes the stringers while the substructure includes the columns and members below the stringers. Figure 4-3 identifies these members.

Table 4-2 Member Loading Summary

Superstructure				
Load Type	Member	Load (lb.)	Demand (lb.)	D/C
Compression	M332	5,400	10,400	.52
Tension	M409	1,450	24,500	.06
Substructure				
Load Type	Member	Load (lb.)	Demand (lb.)	D/C
Compression	M361	3,000	10,400	.29
Tension	M362	2,700	24,500	.11

D/C = demand to capacity ratio where a value less than 1 satisfies code requirements

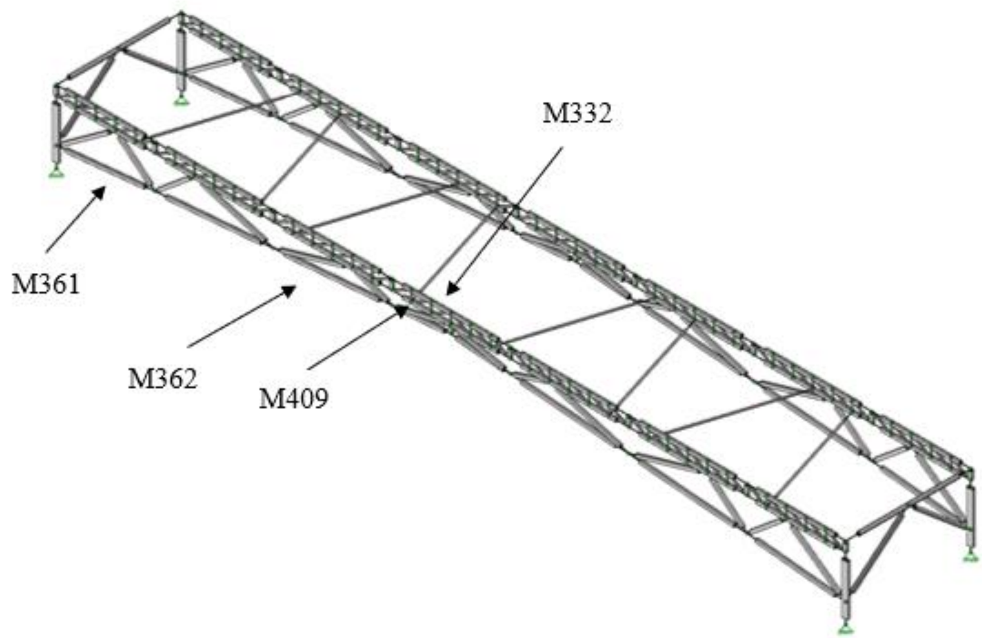


Figure 4-3 Member Callouts

From the information seen in Table 4-1, the highest compression force on a member was in member M332. This indicated that compression mostly occurred in the top of the stringer. Coupling this, the highest tension force in the stringer was in M409. Alongside this the compressive force was much higher than the tension force meaning the internal supports were placed in a way that added additional compressive support to the stringer. Based on the observed data, the middle stringer had bored the greatest load, with the wing stringers experiencing progressively lighter loading as they extended farther from the bridge's center. This concept is visualized in Figures 4-4 and 4-5.

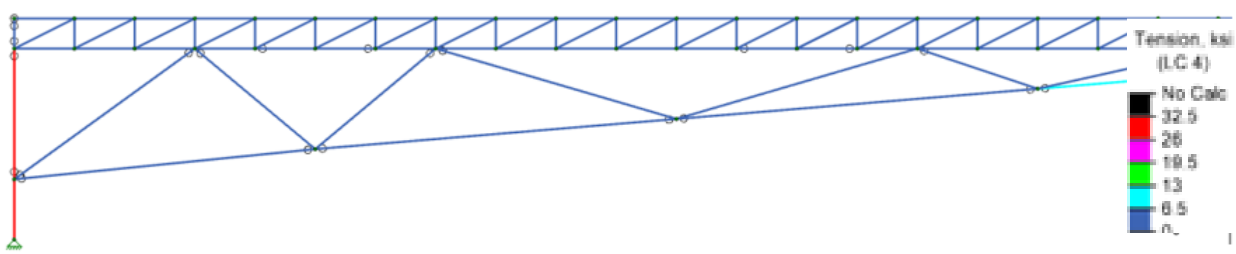


Figure 4-4 Tension Forces

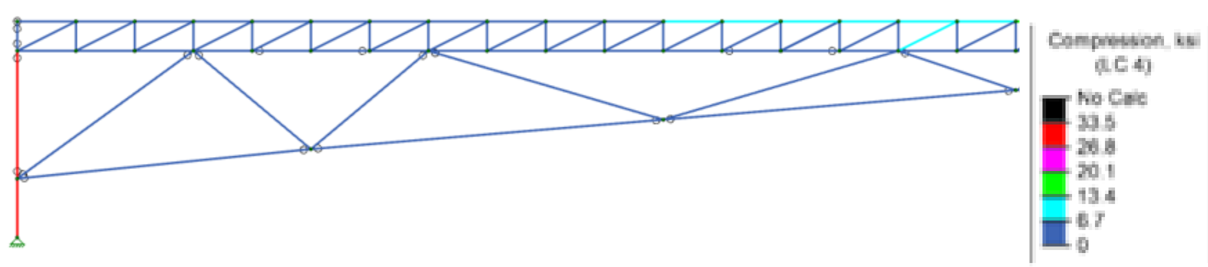


Figure 4-5 Compression Forces

Expanding on Figures 4-4 and 4-5, the figures show the internal loading throughout one side of the bridge. Since the bridge is symmetrical, the worst of the two sides was analyzed and would be replicated to the other side. This was done because the team will not know which side will be loaded until competition day as a north direction is declared during the initial construction of the bridge. These loadings will then be used later to determine the connection demand and identify critical locations.

Another unexpected observation was the supports being in both tension and compression. After looking through the model and experimenting with load placements, the team believes this occurrence is caused by the loads being placed farther from the support. After reviewing the information with the team's technical advisor, it was determined that this was caused by the swaying of the structure. Due to the load placements the bridge would sway in the direction of the clockwise moment. As such, the support farthest from this load was pulled and put it into tension. The team decided that since the support conditions during competition would both be rollers it was more likely the bridge would bow outwards rather than sway.

After finalizing the support and connection conditions, the team then began adjusting the material dimensions to reduce the overall weight of the structure. The team used an iterative process in which material size dimensions were incrementally reduced until the structure could withstand loading within constraints relating to capacity and deflection. The team checked the minimum cross-sectional area required for the highest internal force per the AISC Steel Construction Manual. After some simple stress calculations, the minimum cross section had to be at least 0.003in2. With this information the team began substituting the smallest standard size materials available into the model. Upon final review, the team determined the materials used for the bridge, shown in Table 4-3 below.

Table 4-3 Material Specifications

Component	Material	Weight (lb./ft)	Cross Section Area (in^2)
Stringer/Member	HSS1x1x1/8 Tubing	1.44	.49
Column	HSS 2x1x1/8 Tubing	2.25	.61
Bracing/Stringer Supports	HSS1/2x1/8 Pipe	0.24	.07

Figure 4-6 shows a top-down view of the bridge model and Table 4-4 shows a summary of the internal forces for the lateral bracings.

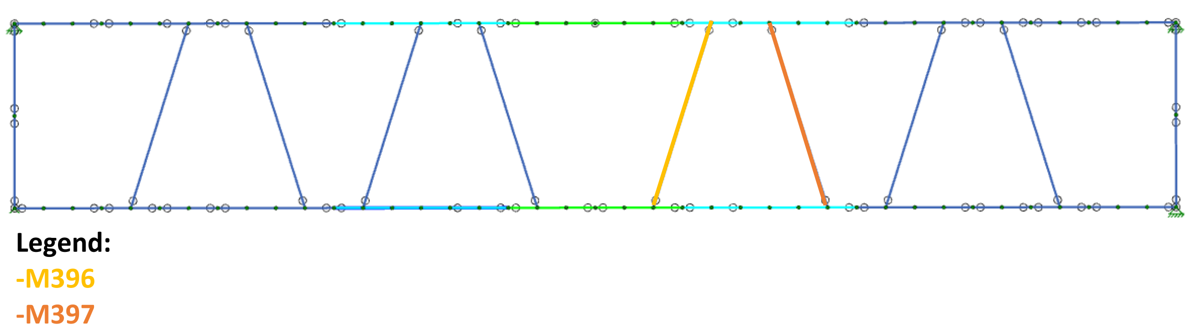


Figure 4-6 RISA 3D Bridge Plan View

Table 4-4 Lateral Bracing Internal Force Summary

Load Type	Member	Demand (ksi)	Yield Strength (ksi)	D/C
Compression	M397	1.1	25	.04
Tension	M396	.7	50	.01

D/C = demand to capacity ratio where a value less than 1 satisfies code requirements

From Table 4-4 and Figure 4-6, the bracings right from the center of the bridge experienced the greatest loading. The team then repeated the same methodology mentioned previously for the other members. Since the internal loadings were small, the team determined that the material size could be significantly reduced as bracings are less likely to deform. As such the team identified the smallest possible material size of 1/4in. seamless piping.

Finally, the columns were chosen to be made of HSS 2”x1”x1/8” to increase the surface area so that the bridge would resist lateral movement during loading. As mentioned, the bridge support conditions are considered roller supports and cannot resist the bridge's movement laterally.

The overall result from the axial behavior can be seen in Figure 4-7 below. This shows how the team anticipates the bridge to behave during loading.

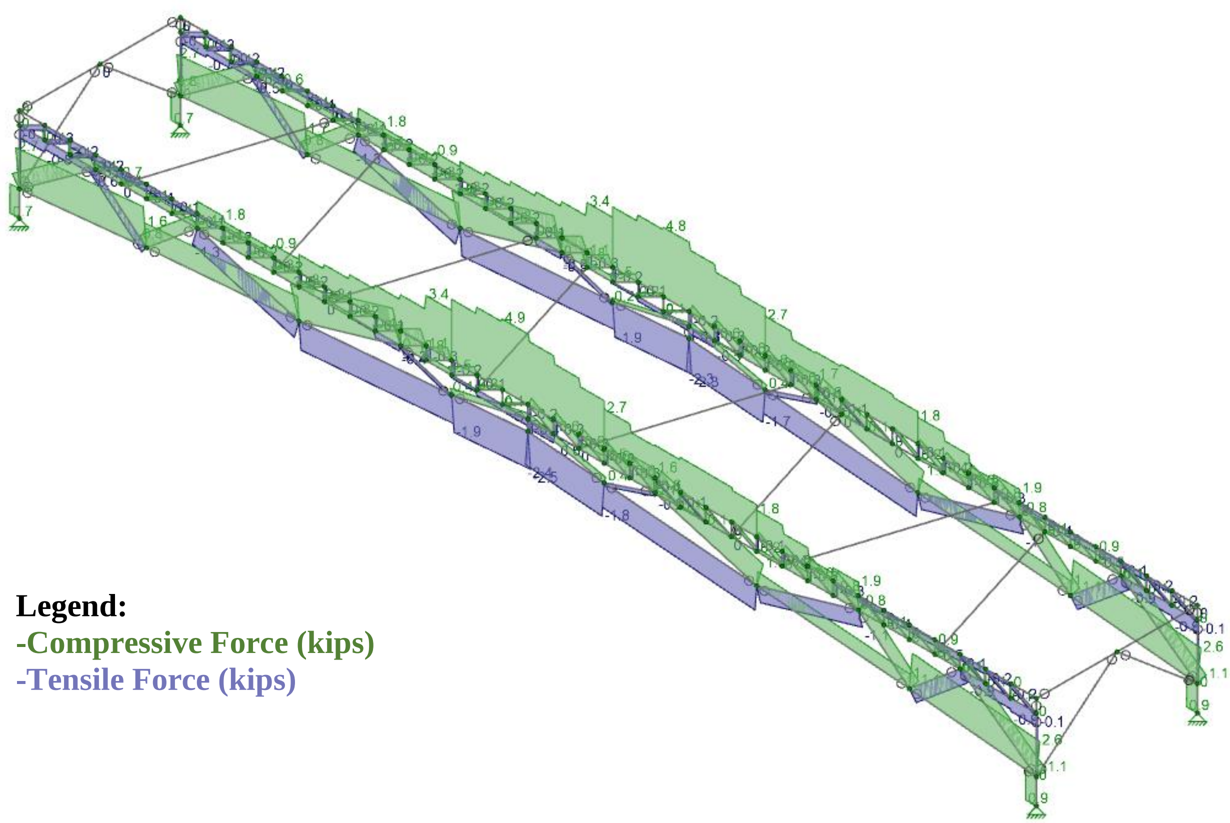


Figure 4-7 Axial Behavior Result

4.3 Serviceability Analysis and Results

The serviceability analysis of the bridge focused on two main concerns: the aggregate deflection measured at the bridge and the LRFD code checks produced by the RISA model. To determine the aggregate deflection of the bridge, two deflection points are measured along the bridge's span. These two points are measured directly under the mid-span of the decking placed on the bridge with the maximum aggregate deflection 3 inch. This 3-inch deflection is the maximum deflection before disqualification. The team's technical advisor advised for modeling purposes to limit the deflection to 1inch to account for potential fabrication errors. Table 4-5 shows the vertical deflection at each measured deflection point and the total aggregate deflection from each load case. Figure 4-8 illustrates the vertical deflected shaped to a 1:10 scale of the greatest aggregate deflection observed by the team. The distances denoted in the tables and their placement can be referenced in Section 2.5.

Table 4-5 Vertical Deflection Summary

Vertical Deflection						
Load Case	Deflection Measuring D1 (Dist. 1) (ft)	Deflection (in)	Deflection Measuring D2 (Dist. 2) (ft)	Deflection (in)	Aggregate Deflection (in)	Deflection Limit (in.)
1	6	.35	10.5	.49	.84	1
2	7.5	.41	13.5	.38	.79	1
3	8.5	.46	14.5	.32	.78	1
4	9	.48	13	.4	.88	1
5	10	.49	14	.35	.84	1
6	11.5	.46	15.5	.25	.71	1

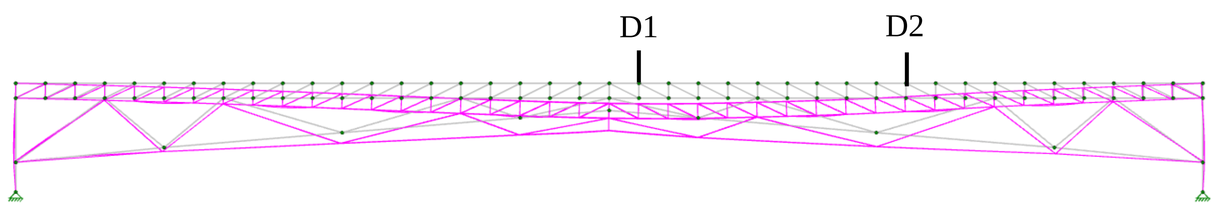


Figure 4-8 RISA 3D Vertical Deflected Shape

As required by the AISC the bridge must also undergo a lateral load test. At the competition, a 50lb. lateral load is applied to the bridge to measure the sway at varying points along the bridge span. The competition has a limit of 3/4in. of lateral deflection caused by this loading. Table 4-6 shows the lateral deflection. Figure 4-9 illustrates the lateral deflected shape to a 1:10 scale. The distances denoted in the tables and their placement can be referenced in Section 2.5.

Table 4-6 Lateral Deflection Summary

Lateral Deflection			
Load Case	Deflection Measuring Dist. (ft)	Deflection (in)	Deflection Limit (in.)
7	7.5	.28	.75
8	9	.24	.75
9	10.5	.23	.75

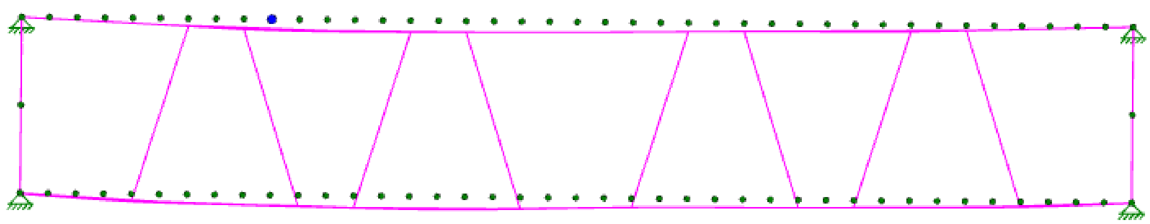


Figure 4-9 Lateral Deflected Shape

As seen in Table 4-3, load case 4 had produced the greatest aggregate deflection. The aggregate deflection was calculated as less than 1 in for the worst-case scenario. After review, the team identified 6 critical locations where connections may incur slipping due to the connected parts being connected by perpendicular plates. At each of these connection locations the team anticipated 1/16 in. of slip due to differences in the hole and bolt diameters. For the total bridge deflection, with fabrication being accounted for, it has an estimated 1.5inches of aggregate deflection assuming the load placements will be loaded under load case 4. Additionally, the team had seen 0.25 inches of deflection in all lateral load cases. Both the vertical and lateral deflections were within tolerance based on the AISC SSBC rules.

In addition to performing analysis on the aggregate deflection, the team also wanted to focus on the AISC code checks present through RISA. The team's technical advisor suggested code checks be conducted to identify critical points within the bridge structure. An illustration of these code checks can be seen in Figure 4-6 and Figure 4-7.

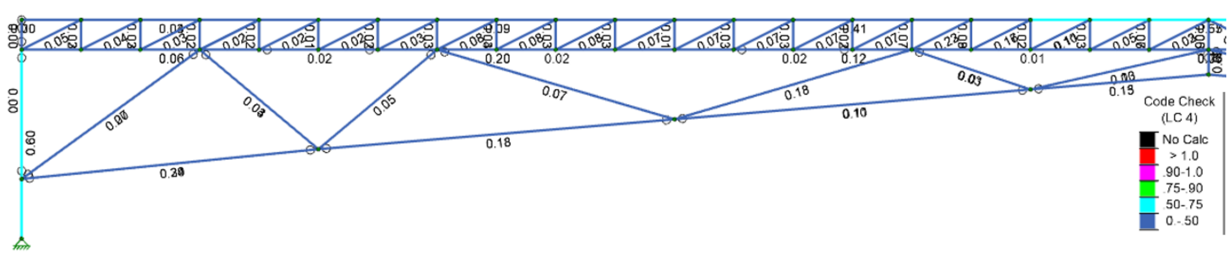


Figure 4-10 RISA 3D Code Check

As RISA outputs data using the LRFD methodology, the code check is meant to display which members are code compliant by analyzing the demand-to-capacity ratio in which a value less than 1 indicates that the member satisfies the code requirements. The code check visually ranges from a deep red color to a deep blue color. The deep red colors signify that the members are either not code compliant or have a demand close to exceeding the capacity of the member. The blue shaded colors indicate the members are code compliant and have a low demand to capacity ratio. The team individually analyzed and assessed each red shaded member to determine whether the design needed to be adjusted. From the model, the team noticed the highest demand to capacity ratio was 0.732, which occurred in the columns of all vertical load cases. This was anticipated due to the column conditions and with the expectation that all the loads would travel into the column.

4.4 Connection Analysis and Design

The connections the team had chosen to use were a variety of angle connections. Initially the team had chosen this type of connection to limit the number of faying surfaces. However, after reviewing the AISC SSBC rules, the definition of a faying surface was further clarified, and plate-on-plate connections would be feasible. When looking at both connections, however, the angle-on-angle connections provided more benefits to the team compared to the typical plated connections. The orientation of the angle connections in relations to the members resisted the moment along the x-y plane of the bridge and released the moment in the y-z plane. Figure 4-11 shows how moment is released for an angle-on-angle connection. This moment release concept was used for all the connections throughout the bridge. Figure 4-11 also shows how the demand was determined for all the angle-on-angle connections.

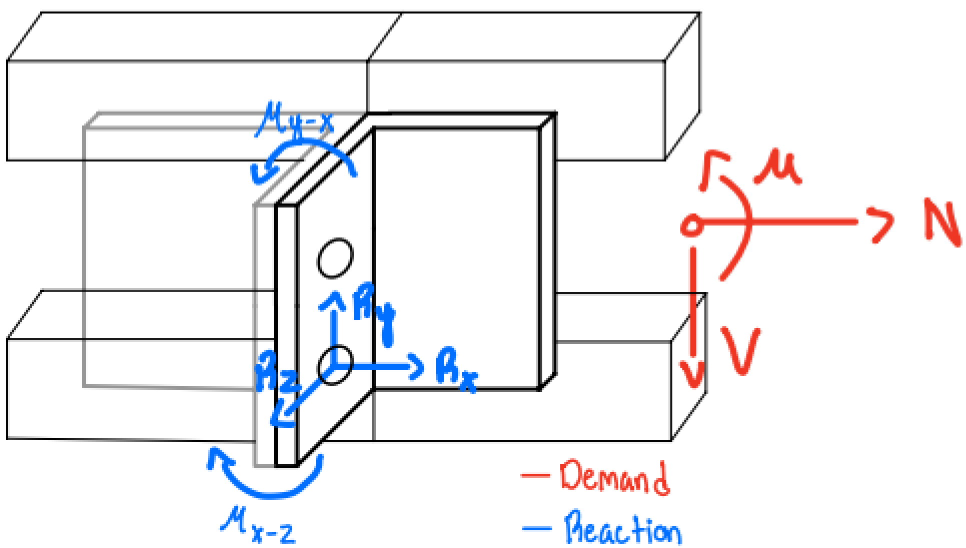


Figure 4-11 Eccentric Reaction and Demand Concept-

This additional resistance in the x-y plane showed the internal force demand produced by the members is directed into the bolt rather than into the angle. The major disadvantage to these connections is that the demand for these connections will be considered eccentrically loaded. As such there will be an increase in the demand going into the bolt. The team's technical advisor had advised that this type of connection will produce a slight increase in demand. The team decided to continue with designing the angle-on-angle connections. To begin analyzing the connections the team aimed to determine the demand at several connection locations in reference to the code checks in Section 4.3. To analyze the eccentric connections, the "Elastic Method," or the "Instantaneous Center of Rotation Method" was used to account for the eccentricity. For the analysis of this project, the connections were analyzed using the elastic method. The team determined that if the connection capacity could withstand the demand produced by the elastic method, then the connection would work for the demand solved for by either method.

Using the elastic method the demand was solved by adding the resultant forces from a concentric load and concentrated moment. The concentrated moment is solved by multiplying the moment caused by the eccentric force by the distance between the centroid of the bolt group to the center of a bolt hole. then dividing by the polar moment of inertia of the bolt group. An example of this calculation can be seen in Appendix A.

The team began individually analyzing the force going into each connection to determine which connection would experience the greatest shear force. Figure 4-12 shows the connection locations and the loading going into each connection. The connection location numbers are marked in black. The red coloring indicates which members are primarily in compression while the blue coloring indicates which members are in tension. Figure 4-12 only shows one side of the bridge as it is symmetrical as analysis done on one side of the bridge can be duplicated to the other side of the bridge. After finding the demand for points 1-12, the team found that point 3 experienced the highest overall shear force at roughly 5,100 lbs. In the effort to make the fabrication and analysis more simplified, the team decided to make a universal design for the substructure connections.

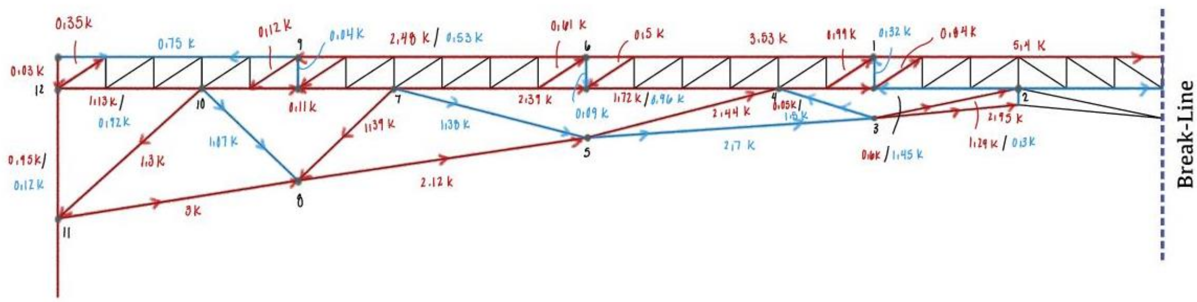


Figure 4-12 Connection Locations

Immediately after determining the connections' demand, the team checked 8 different failure modes for the connections found in the AISC Steel Construction Manual. For the analysis, each bolt was assumed to be in bearing. This assumption was made due to the construction not being able to properly tension the bolts to create slip-critical connections. Figures 4-13, 4-14, and 4-15 illustrates the connections used for the stringers, the connections used to connect the stringer to the column and lateral bracing, and the connections used to connect the substructure members. Table 4-7 summarizes the demand to capacity ratio for the connections in relation to each of the failure modes. For complete calculations refer to Appendix A.

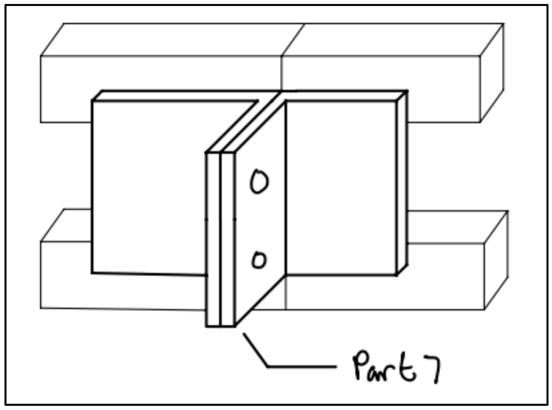


Figure 4-13 Stringer Connection Illustration

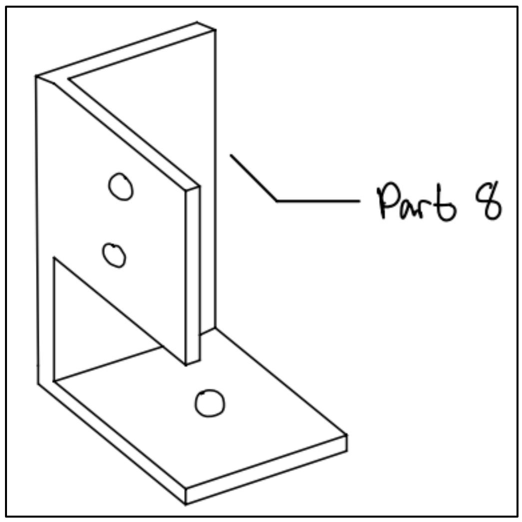


Figure 4-14 Column/Lateral Bracing Connection Illustration

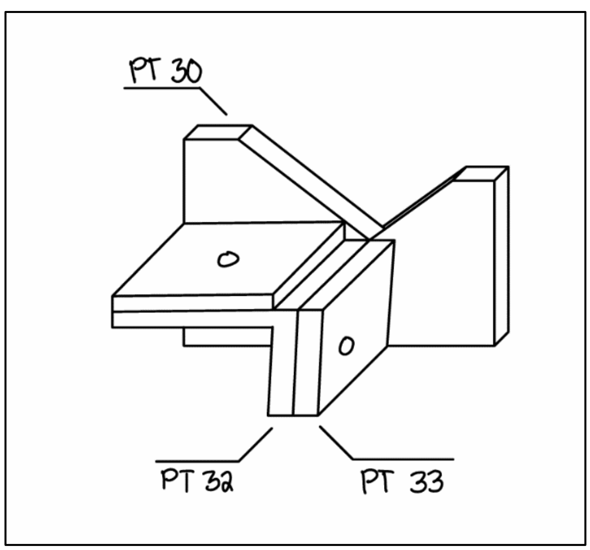


Figure 4-15 Universal Substructure Connection Illustration

Table 4-7 Connection Analysis Summary

Part #	Part 7	Part 8	Part 30	Part 32	Part 33
	D/C	D/C	D/C	D/C	D/C
Bearing at bolt hole	.2	.01	.78	.78	.78
Tearout at bolt hole	.1	.66	.43	.21	.21
Block Shear	.06	.24	.39	.08	.08
Compression Buckling	.02	.12	.42	.04	.04
Gross Section Yielding	.02	.12	.42	.04	.04
Net Section Rupture	.02	.19	.72	.07	.07
Combined Shear and Tension (Bolt)	.12	.12	.46	.46	.46
Weld Strength	.19	.1	.34	.93	.93

D/C = demand to capacity ratio where a value less than 1 satisfies code requirements

4.5 Final Design and Analysis Summary

The final recommended design can be seen in Table 4-10 below.

Table 4-8 Final Design Summary

Final Bridge Design	
Superstructure Design	Arch Bridge
Substructure Design	Camelback Truss
Steel Type	Lateral Bracing- HSS 0.5”x1/8” Columns- HSS 2”x1”x3_A1085 Stringers- Bracing HSS 0.5”x1/8”, Top HSS 1”x1”x1/8” Arch Members – HSS 1”x1”x1/8”
Dimensions	Length – 20'-0” Height – 1'11” Width – 3’-0"
Connection Types	Angle on Angle Steel – 1/8” Thickness Plate Connections – 1/8” Thickness *Size Varies Bolt Specifications – A490 Bolts

5.0 Bridge Production

5.1 Shop Drawings

To begin fabrication for the bridge, the team began by developing shop drawings intended for the project's fabricators. An example of the shop drawings can be seen in Figure 5-1. The shop drawings were made using an orthographic projection. The orthographic projection was used to easily dimension key features such as weld placements, weld types, hole punches, etc.

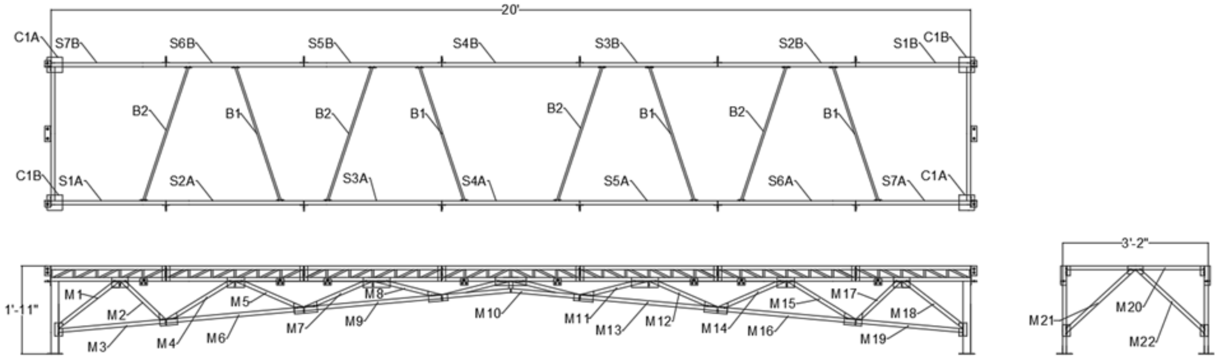


Figure 5-1 Bridge Plan and Profile View

From Figure 5-1 each member was called out in which separate detail drawings were drafted. Each detail drawing was dimensioned and called out various parts required to be welded to the members. To organize the drawings each drawing name began with the first letter of the intended parts such as “S” for stringer, “M” Members “C” for columns. Following the letter came the part number. Then if there were duplicate parts for the left or right side of the bridge the letters A or B were used. An example of this nomenclature would be S1A. S1A calls out that the drawing the fabricators are looking at is stringer 1 on the right side of the bridge. An example of a detail drawing can be seen in Figure 5-2

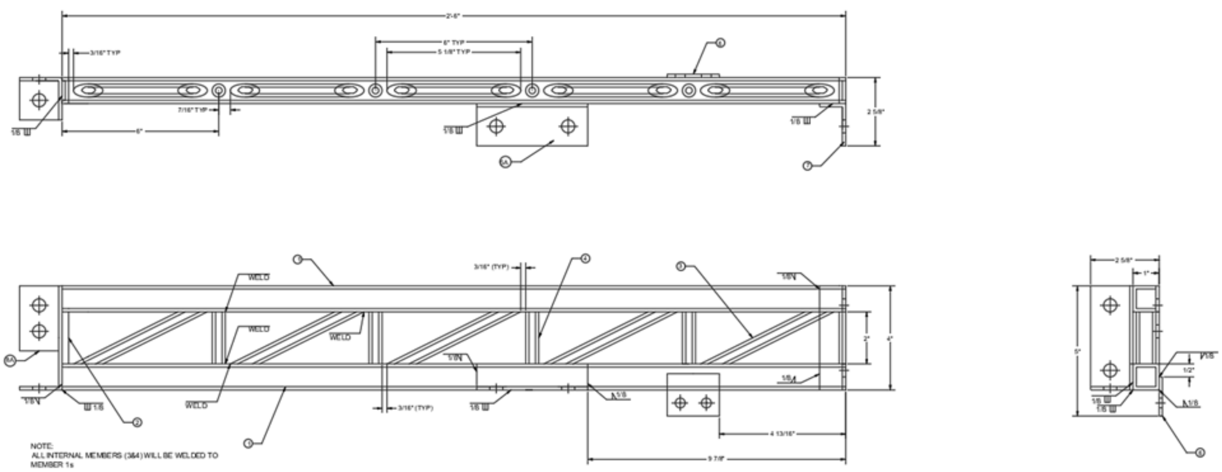


Figure 5-2 Example Detail Drawing

Finally, to finish the shop drawings the team had developed individual part drawings for all the connections. Each connection detailed a bent and unbent part. Figure 5-3 shows an example of the connection detailing.

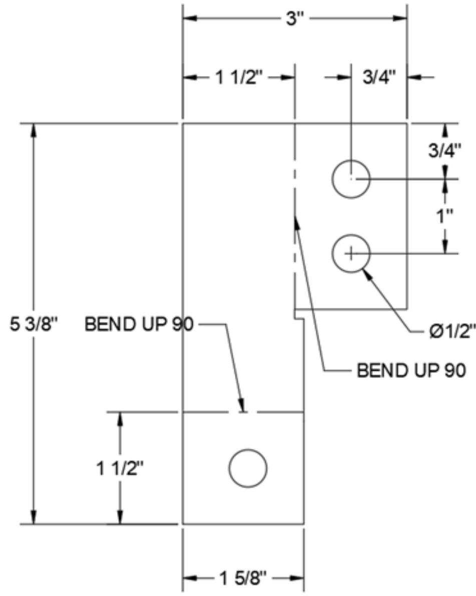


Figure 5-3 Example Part Drawing

From Figure 5-3 the connection cannot be made from a single simple plate or traditional angle. As such the team had to develop unbent part drawings and call out the locations along the plate in which the part would be bent up. To make the fabrication easy the team had developed each connection to be simply bent up 90 degrees. After finishing the part drawings, the plan set was then made and sent to the fabricators for review. After the fabricators had reviewed the drawings, additional details were added per the fabricators request.

5.2 Fabrication Coordination

The team enlisted the Flagstaff High School (FHS) welding department to weld most of the bridge's parts. Figure 5-4 shows an illustration of the work done to weld the stringers of the bridge. Part A shows an image of the FHS welding department welding a separate project from the bridge. Part B shows an image of one team member cutting the steel for the stringers to be welded. Part C shows the finished product of a single stringer after FHS was able to finish the welding task.

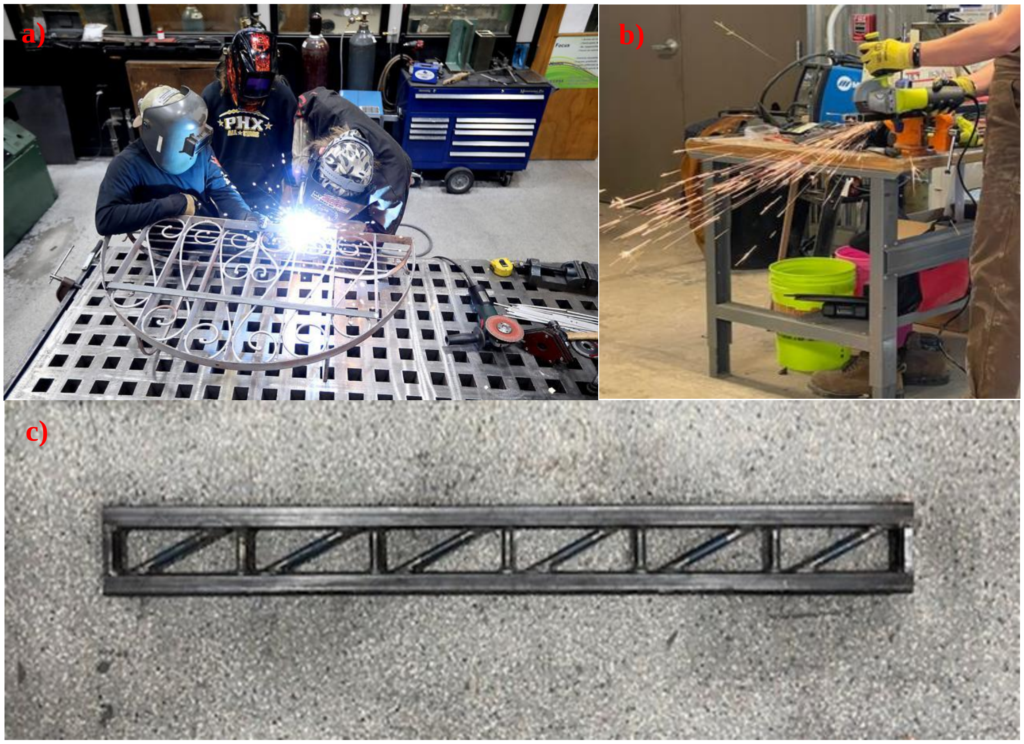


Figure 5-4 Stringer Fabrication Process [9]

An initial meeting with the FHS principal, Libby Miller, and the welding teacher, Mike Rust, was set up during December 2023. In the meeting the team had gone over the details of the drawings and the scope of the project. Upon the meeting's conclusion, it was settled that FHS could weld parts of the bridge leading up to March 29th, 2024. With this the team then began scheduling regular meetings with Mr. Rust, determining shipment dates, and anticipated deadlines. The team also worked within the welding class time to answer any questions the welding students may have had about the projects or shop drawings.

5.3 Mentee Collaboration

Prior to bridge assembly, the team began seeking mentees for the project. The goal for the mentees was to obtain aid with the assembly and prepare the following years' steel bridge team guidance on design processes. To recruit mentees, the team had spoken at an ASCE meeting during the Fall 2023 semester. During this time, the team obtained contact information for several potential students interested in the steel bridge project. The team held periodic meetings with the mentees to go over the bridge design, calculations, and fabrication at the CECMEE field station.

6.0 Assembly

6.1 Initial Bridge Assembly

The team began by assembling the bridge on each side to determine how the pieces would construct together. Initially the team had assembled the bridge without bolts to determine what pieces needed to be grinded, cut, and if filler material was needed. If any members or connection needed adjustments, the team would mark the adjustment piece.

6.2 Design Modifications

After the initial assembly, the team then went through and reviewed the marks made on the adjustment pieces then adjusted accordingly with welding filler material or grinding deformities. Additionally, the team had noticed that during competition, it may be difficult to quickly determine which members would go on which side of the bridge. As such the team then decided to paint the stringer and members to easily differentiate which members would go on the left or right side of the bridge. The team also developed a labeling scheme to help with identifying members quickly. The scheme simply followed the labeling of the members as seen in Figure 5-1. More details regarding the use of the scheme will be developed in section 6.3.

6.3 Competition Assembly Practice

Competition practice began by completely assembling the bridge without a time limit and without boundary restrictions. The team's aim for this assembly was to ensure every builder understood the procedures, construction regulations, and general assembly. The team also brainstormed the best way to approach the construction and the role each team member would have. Figure 6-1 shows an example of the bridge completely assembled with the bolts in.

Figure 6-1 Completely Constructed Bridge

From Figure 6-1, the bridge was fully constructed while also supporting its self-weight. At this point, the team determined that to accurately determine the time it would take to assemble the bridge during competition, a mock construction site had to be made. To do this, the team used tape and chalk to draw out the construction site's dimensions as seen in Figures 2-6 and 2-7. Figure 6-2 shows a picture of the construction zone specified in the competition rules.

Figure 6-2 Construction Site CEMCEE FIELD Station

After taping and drawing out the mock construction site the team scheduled construction times for the practice assembly with the team's mentees. The team had scheduled two practice assembly days on April 5th and April 7th. The team had established a goal of less than 30 minutes to complete construction. During competition, the limit for construction is 30 minutes where penalties count towards score. If the team cannot stay within the time limit, an extra 15 minutes is allotted to complete the bridge with construction restraints. If the bridge could not be constructed during this 45-minute, allotted time, the bridge would have to be completely disassembled and would be disqualified. As such, the team aimed for less than 30 minutes construction time, but it would continue practice until under 45 minutes.

The scheme used to construct the bridge was done as follows. Firstly, when looking at the bridge the numbering would increase from left to right i.e. if someone were to look at the bridge from the side view, they would read S1-S7 increasing from 1-7 and M1-M19 increasing from 1-19. This was done so that no matter which side was declared as north the team would be able to follow the same construction plan. The team would start by constructing all the stingers and the lateral bracings. From there, the team would build the substructure members in a set of 3 following a clockwise and counterclockwise pattern depending on which side of the bridge a team member was on Figure 6-3.

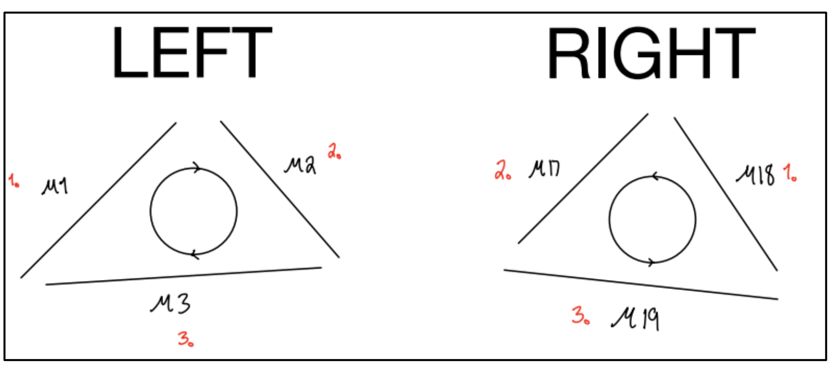


Figure 6-3 Substructure Construction Illustration

After the practice assemblies, the team achieved 36 minutes. Although this was not ideal the team was within the 45-minute time constraint to still be able to load the bridge. As such the team deemed this time to be acceptable and carried forward to competition.

7.0 Competition

7.1 Competition Display

April 11th marked the first day of competition. In the morning, the team had set up the bridge for display alongside other university teams. During the display, the judges observed the bridge for aesthetics, the bridge poster, and asked questions about the bridge. After the end of the display, the team deconstructed the bridge and prepped the bridge for testing day. The following day started with the construction speed competition followed by the vertical and lateral load competition.

7.2 Competition Results

Competition results were given to the team on April 12th. Table 7-1 shows the competition results given to the team by the head judge based on the categories listed for judging.

Table 7-1 Competition Results

Competition Results		
Category	Result	Ranking
Assembly Time	189min*	8
Construction Cost	$171,787,500*	8
Weight	662lb*	7
Vertical Deflection	10in*	8
Structural Efficiency	$63,758,166*	8
Aesthetics	9.71	8
Overall Ranking	8	

Table 7-1 shows the competition results after penalties were acquired at competition. The team's assembly time was 39 minutes however since the team had gone over the 30min construction time an additional 150 minutes was added on to the construction time. The construction cost used a predetermined equation set by the judges that was a function of the assembly time. The team had projected a construction cost of $25,000,000. However, since the team had gone over the 30-minute construction time and dropped numerous items during construction the value became excessively inflated. The actual weight of the structure came out to be 325 lbs. During construction the team had dropped tools and after loading the structure the deflection became too great. As such penalties inquired increased the weight of the structure for the competition results. The bridge as well unfortunately did not withstand the 2500lbs and as such an automatic 10in vertical deflection was given to the team. The structural efficiency as well was another preset equation used by the judges that was calculated as a function of the vertical deflection. The team anticipated a structural efficiency cost of $5,000,000 however due to the 10in penalty this number as well was excessively inflated.

7.3 Lessons Learned

After the competition, the team went back through the bridge's modeling to determine what caused it to fail. After reviewing the model and changing the supports to pin and roller, the model became unstable which means that without all the supports being pinned the bridge deflected too much. Figure 7-1 shows the aggregate deflection after changing the pin supports to roller supports.

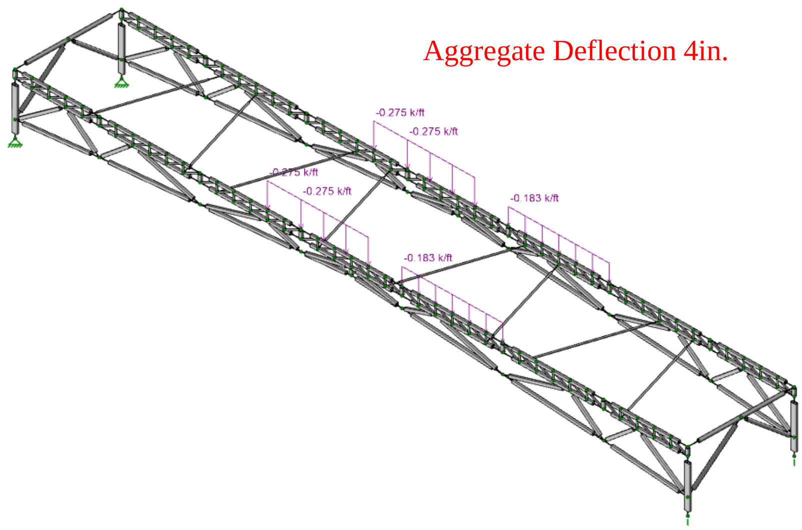


Figure 7-1 Adjusted RISA Model

The team then set up an additional meeting with the TA to determine what may have gone wrong. From the analysis there were two connection points at the base of the columns that the team had modeled as fixed connections instead of pin connections. As such this meant that the model was producing inaccurate results from the beginning. As well as this during fabrication the team encountered numerous fabrication errors with the drill press which made the holes much bigger than the oversized 1/8in that was anticipated. With these factors in mind the team was able to understand why the bridge was unable to hold up the load during competition.

Below the team has listed out lessons learned that should be taken into next year's competition.

- Use a tension-controlled bridge such as a Warren Truss or Lockport style bridge.
- Avoid using Eccentric Connections
- Triple check the bridge envelope restrictions for the difference between max stringer height and max bridge height.
- Model the bridge only using pin and pin connection to simplify the design.
- Get holes machine cut/laser cut to a 1/32nd precision then file the holes according to the bolt size.
- Use double shear connections when applicable.
- Coordinate additional time with TA to hold a mock judging assembly.
- Make construction scheme as simplified as possible for mentees.

8.0 Impact Analysis

When conducting the impact analysis for the project, the team considered two alternatives which are constructing the bridge and not constructing the bridge. Although the project task had been building a scale model of the bridge for the feasibility study, the team wanted to determine if implementing the bridge was necessary. To accomplish this, each team member had gone through using the triple bottom line methodology. Using this methodology each team member was tasked with determining positive and negative impacts for each category. The team came together and agreed on the final scoring for each alternative. Table 8-1 summarizes the impacts the team expects to have on the surrounding area for each alternative solution.

Table 8-1 Impact Analysis Summary

Impact Analysis				
		Social	**Economic**	**Environmental**
Bridge	Pros (+)	- Additional path for Hideaway Park use - Safe passage across waterways - Improved accessibility to other areas of the park - More entertainment activities - Gives additional aesthetic appeal to park	- More clientele causing increased revenue - Provide work for contractors, maintenance, and revenue for park - Increased park versatility	- Minimize foot traffic along local ecosystems - Reduce the chance for the open area to be replaced by other unsustainable infrastructure - New desire to upkeep the park for access to new amenities
	Cons (-)	- Reduced aesthetic with obstruction to potential viewpoints - Construction noise causing park disruption - Foot traffic will need to be directed away to avoid conflict with construction zone	- Construction pollution from equipment, tool, etc. - Material Fabrication - Bridge maintenance	- Steel costs to fabricate and transport - Heavy equipment usage cost - Construction disturbing environment - Construction might allow invasive species
No Bridge	Pros (+)	- Noise limited due to no construction zone - Park's existing aesthetics will be preserved	- Money can be reallocated to improve other aspects of the park - No liability due to bridge	- Conservation of local flora and fauna - No pollution from construction - No disturbance to the river or neighboring bodies of water
	Cons (-)	- Fewer activities or options for users - Park patrons crossing the river unsafely - Limited space for park users - Current tourism would remain at a standstill without a significant increase	- People will explore other opportunities due to a lack of versatility - Resources dedicated to maintaining pathways to redirect park patrons - Reduced tourism to park and local area	- Wildlife disturbance due to limited pathway options

Table 8-2 summarizes the scoring for both alternative solutions.

Table 8-2 Impact Analysis Scoring

	Social	Environment	Economic	Total	Max-Min	SI
Bridge	**70**	**50**	**80**	**200**	**30**	**170**
No Bridge	**50**	**70**	**40**	**160**	**30**	**130**

From Table 8-2, implementing the bridge had scored higher implying that the bridge would be better for the park economically, environmentally, and socially. By building the bridge there will be increased amounts of tourism and it will act as a focal point for the surrounding community and Hideaway Park Disc Golf Course visitors. Incorporating the bridge is also important to ensure park patrons do not make their own paths or cross the river unsafely. By incorporating the bridge, the team anticipates this will protect the local ecosystems as well improve the safety of all the patrons. Finally, the team anticipates the construction of the bridge to bring an increased amount of revenue to the park and local businesses. This bridge may spark new attractions causing an increase in tourism and clientele to neighboring businesses.

A few notable issues were considered when implementing the bridge. A primary issue for the team came to be the bridge's construction. During construction, there would be more pollution, disturbances to local patrons, and ecosystems. The disturbance due to noise would only be temporary during construction, meaning after construction is complete there will be no other noise disturbances caused by the bridge. Also, the team aims to do construction as quickly as possible to limit the prolonged amount of pollution. The team anticipates the bridge will be paid for by taxpayers' taxes; however, the revenue generated by local tourism will be cycled back into the local economy. Although there are negative impacts that come along with constructing the bridge, the team believes that the positive impacts offset the negative impacts substantially and advise the bridge's construction.

9.0 Summary of Engineering Work

9.1 Proposed Schedule vs Adjusted Schedule

Throughout the project the team was able to complete tasks similarly to what was planned for the proposed project schedule. The major complication the team experienced was during Task 5. In the proposed schedule, the team planned to have 20 days of practice assembly time. However, due to the bridge having to be shortened in height due to a design oversight, along with unexpected weather, the practice time was cut back significantly to 8 days. The other 12 days of time was then allotted to the design modifications which included the team redrafting shop drawings for members, connection redesign, and increased time to fabricate the bridge. Other than tasks 5.2 and 5.3, the team has stuck to the project schedule as intended. Figures 9-1 and 9-2 illustrate the proposed project schedule and the actual project schedule.

Figure 9-1 Proposal Schedule

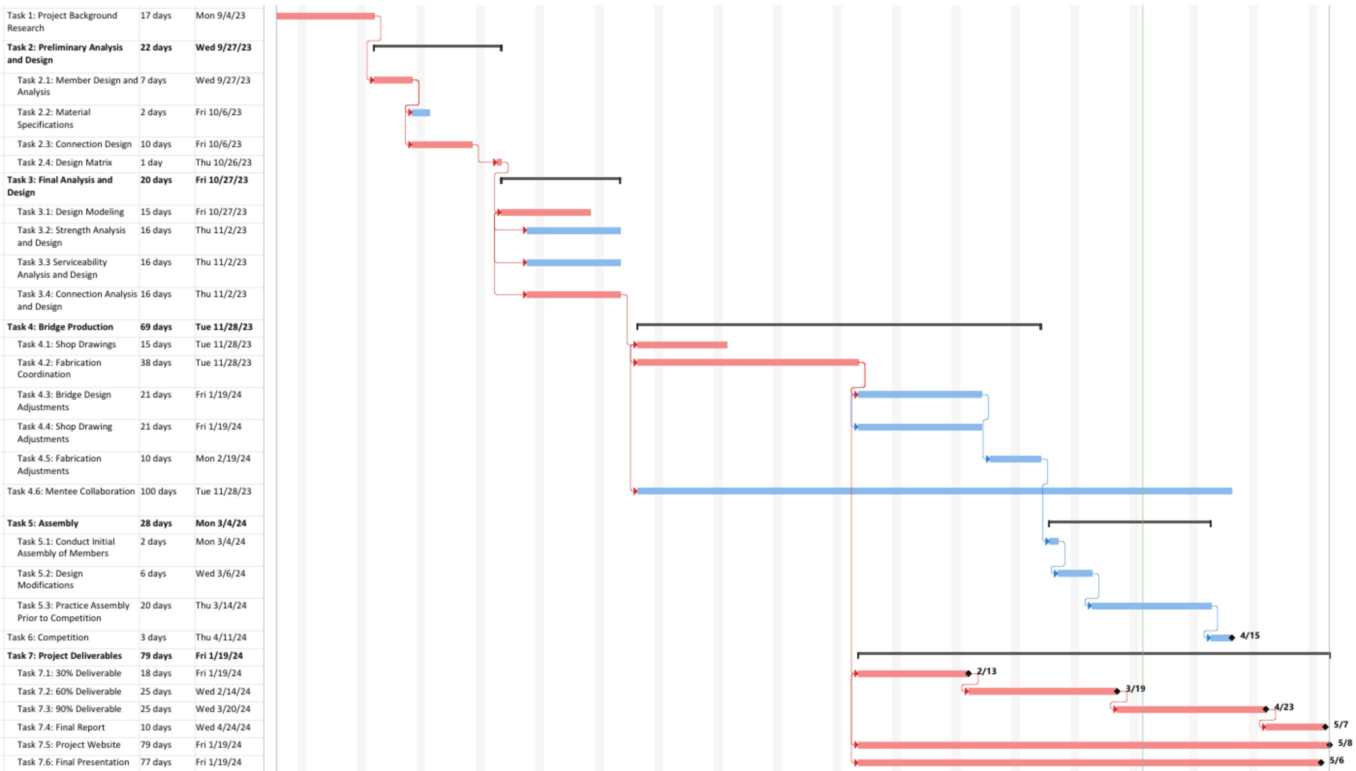
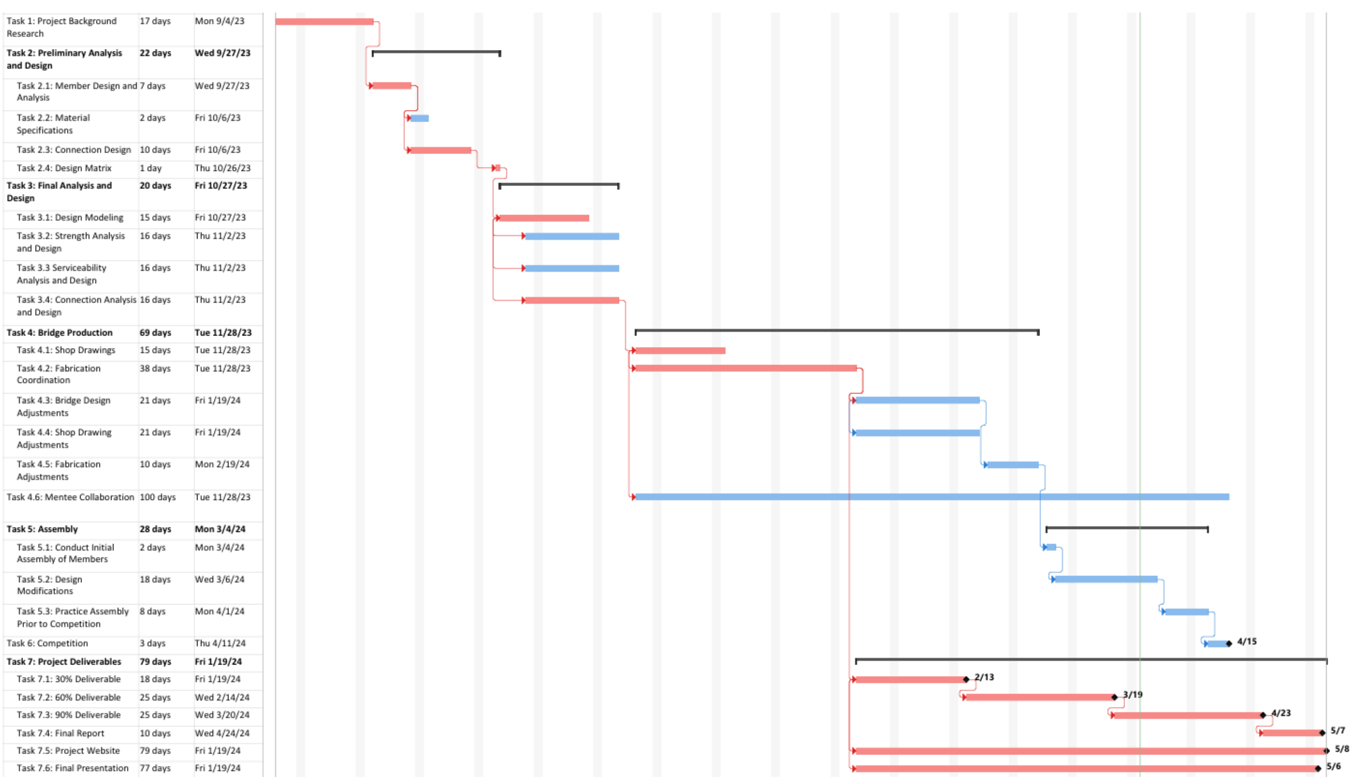


Figure 9-2 Actual Schedule



9.2 Cost of Work Scheduled vs Cost of Actual Work Performed

Throughout the project the team recorded their hours for the work performed. While this was happening, the team had aimed to stay as close to the proposed hours as possible. Tables 9-1 and 9-2 show the proposed hours worked and actual hours worked, respectively.

Table 9-1 Proposed Hours

Task	SENG Hours	ENG Hours	TECH Hours	DRFT Hours	Task Hours
1.0 Project Background Research		15			15
2.0 Preliminary Analysis and Design	5	30			35
3.0 Final Analysis and Design	20	100	30	30	180
4.0 Bridge Production	20	20	60	80	180
5.0 Assembly	5	20	50	20	95
6.0 Competition	5		15		20
7.0 Project Deliverables	65	10	20		95
8.0 Project Management	40	15	10	10	75
Subtotal	160	210	185	140	710

Table 9-2 Actual Hours

Task	SENG Hours	ENG Hours	TECH Hours	DRFT Hours	Task Hours
1.0 Project Background Research	0	22	0	0	22
2.0 Preliminary Analysis and Design	2	23	0	0	25
3.0 Final Analysis and Design	10	108	30	15	163
4.0 Bridge Production	5	10	180	65	260
5.0 Assembly	0	78	62	55	195
6.0 Competition	2.5	0	10	0	12.5
7.0 Project Deliverables	22.5	10	10	0	42.5
8.0 Project Management	35	15	10	10	70
Subtotal	77	266	302	145	790

Table 9-2 shows that the team exceeded the proposed subtotal hours by 80 hours. Tasks 1,2, and 3 were completed within the allocated time, however, the estimates for Tasks 4 and 5 were significantly off causing major overages. The bulk of the overage occurred during Task 4, where the team underestimated the person-hours needed for fabricating the bridge. This task required extensive time for reviewing the plan sets, coordinating with the fabrication team, and ensuring compliance with design specifications. An additional complication arose from an oversight in the bridge's height, which required considerable extra fabrication work to correct the error. The oversight also impacted the assembly phase in which the engineer and technician had to make considerable design modifications. As a result, the drafter had to revise a greater number of drawings, further contributing to the hour overage. The team had attempted to make up for this overage by reducing the hours worked on Task 6.

Three of the four positions as well had different hours than what was projected. The senior engineer had worked much less than anticipated. This was due to the lack of time needed to review the engineer's work. The engineer worked more hours than anticipated, due to the oversight of the bridge envelope. Finally, the technician worked more hours than anyone on the project due to the need to refabricate. Tables 9-3 and 9-4 summarize the cost of the proposed work and the cost of the actual work performed, respectively.

Table 9-3 Proposed Cost of Work

Proposed Cost of Engineering Services				
1.0 Personnel	Classification	Hours	Rate, $/hr	Cost
	SENG	110	$216	$23,760
	ENG	230	$131	$30,130
	TECH	180	$114	$20,520
	DRFT	140	$84	$11,760
	Subtotal			$86,170
2.0 Travel	No.	Unit	Unit Cost	Cost
Rental Truck	$5	Days	$129	$645
Truck Driving Mileage	$1,232	Miles	$0	$548
Rental Van	$3	Days	$48	$145
Van Driving Mileage	$1,100	Miles	$0	$490
Hotel	$9	Person Nights	$113	$1,017
Per Diem	$18	Days	$50	$900
	Subtotal			$3,745
3.0 Subcontract				
		Hours	Rate, $/hr	Cost
Fabrication		$100	$92	$9,219
4.0 Misc.				
	No.	Unit	Unit Cost	Cost
Supplies	$1	N/A	N/A	$3,988
Equipment	$1	N/A	N/A	$192
5.0 Total Cost				**$103,313**

Table 9-4 Actual Cost of Work

Actual Cost of Engineering Services				
1.0 Personnel	Classification	Hours	Rate, $/hr	Cost
	SENG	77	$216	$16,632
	ENG	266	$131	$34,846
	TECH	302	$114	$34,428
	DRFT	145	$84	$12,180
	Subtotal			$98,086
2.0 Travel	No.	Unit	Unit Cost	Cost
Rental Truck	$5	Days	$129	$645
Truck Driving Mileage	$1,232	Miles	$0	$548
Rental Van	$3	Days	$48	$145
Van Driving Mileage	$1,100	Miles	$0	$490
Hotel	$9	Person Nights	$113	$1,017
Per Diem	$18	Days	$50	$900
	Subtotal			$3,745
3.0 Subcontract				
		Hours	Rate, $/hr	Cost
Fabrication		$100	$92	$9,219
4.0 Misc.				
	No.	Unit	Unit Cost	Cost
Supplies	$1	N/A	N/A	$3,988
Equipment	$1	N/A	N/A	$192
5.0 Total Cost				**$115,229**

From Table 9-4, the project's total cost was $115,229. This means that the project unfortunately was 12% over budget. This is by far due to the increased work that the engineer and technician had performed.

10.0 Conclusion

After the modeling step, the team projected a 1.5 in vertical deflection and a 0.25 in lateral deflection. Satisfied with these results the team had begun fabricating and constructing the scale model bridge. During the initial and practice assembly, the team noticed a sag in the middle of the bridge which could not be resolved due to time constraints. As such the team carried forward to the competition with the bridge. During the competition, the team was able to load the bridge laterally and met the 0.25 in expected deflection. However, once the team began loading the bridge vertically the bridge deflected too much, and loading had to be halted. The bridge held 200 lbs. of superimposed dead load and 425 lbs. of live load before the aggregate deflection exceeded 3 in. Reflecting on the competition the team believed that the error occurred during modeling. After going back through RISA and putting in roller supports, the model produced instability errors. The team then developed a Lockport bridge model which supported the design load using the pin and roller supports. After developing this model, the team determined that a compression-controlled bridge, like an arch bridge, would not be viable due to constraints of the competition. The team had placed 8th out of 9 competitors. Throughout the project the team was able to stick to the schedule with little setbacks where buffer or days were managed to meet critical deadlines, and the final cost of the project came out to be $115,229.

11.0 References

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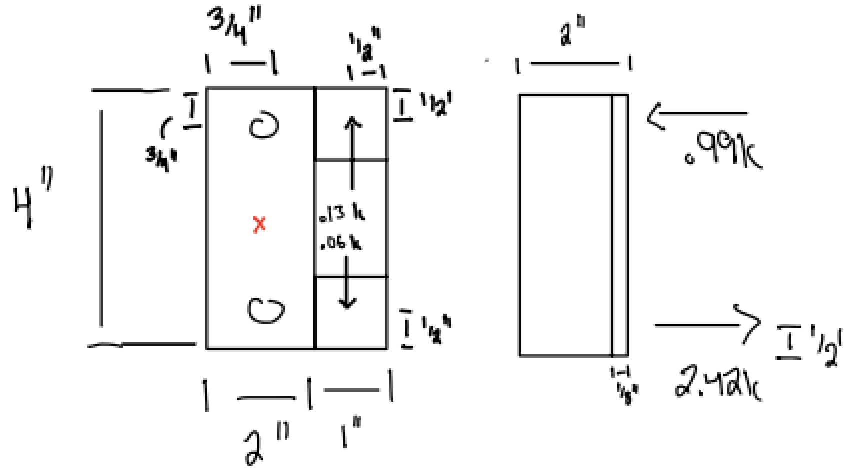
Appendices

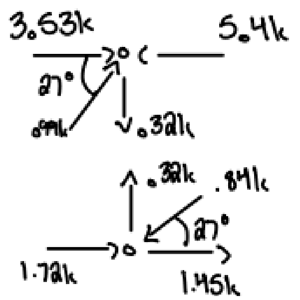
Appendix A: Connection Hand Calculations

Connection 1 Hand Calcs



L-Shape 2" x 2" x 1/8"
ASTM A490 3/8" bolts

Bolt Group In Angle



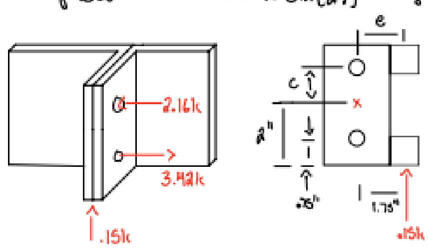
VS.

$\Sigma(F_x)_{Top} = 3.53k - 5.4k + .99k\cos(27) = -2.16k$

$\Sigma(F_y)_{Top} = .99\sin(27) - .32k = .63k$

$\Sigma(F_x)_{Bot} = 1.72k + 1.45k - .84k\cos(27) = 3.42k$

$\Sigma(F_y)_{Bot} = .32k - .84k\sin(27) = -.48k$



Elastic Method

$$\frac{150\,lb}{2\ bolts} = 75\ ^{lb}/_{bolt} = P$$

$$R_y = \frac{M_x \cdot c}{I_p} = \frac{.262k\text{-}in\,(1.25in)}{3.125in^2} = .105k/bolt$$

$M = .15k \cdot 1.75'' = .262\ k\cdot in$

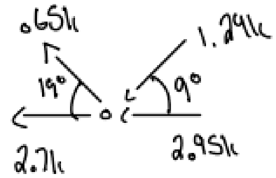
$c = 2'' - .75'' = 1.25\,in$

$I_p = \Sigma c^2$
$\quad = 2(1.25)^2 = 3.125in^2$

$R_{Total} = .105\ ^{k}/_{bolt} + .075\ ^{k}/_{bolt} = .18\ ^{k}/_{bolt}$

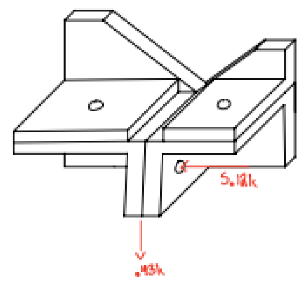
$R_u = .18\ ^{k}/_{bolt} \cdot 2\ bolts = \underline{.36k}$

Single Bolt In Angle



$\Sigma F_x = -2.7k - 2.95k - 1.29k\cos(9) - .65k\cos(19) = -5.12k$

$\Sigma F_y = .66k\sin(19) - 1.29k\sin(9) = -.43k$



Bearing at bolt hole

$R_n = 2.4 d t F_u$ [J3-6a]

$= 2.4(.5in)(.125in)(58ksi)(.75) = \underline{6.53k}$

Tearout at bolt hole

$R_n = 1.2 l_c t F_u$ [J3-6c]

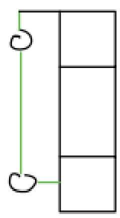
$= 1.2(.75'' - .219'')(.125in)(58ksi)(.75) = \underline{3.46k}$

center to edge

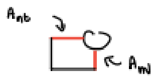
1/2 std. hole

Block Shear

Failure mode 1



Failure mode 2



controls - Less A_{nv} than mode 1

$$R_n = .6F_u A_{nv} + U_{bs} F_u A_{nt} \le .6 F_y A_{gv} + U_{bs} F_u A_{nt} \quad [J4\text{-}5]$$

$F_u = 58ksi$ $\quad U_{bs} = 1$

$F_y = 36ksi$

$A_{gv} = .125in(.75in) = .094in^2$

$A_{nv} = .094in^2 - \frac{1}{2}(.5in)(.125in) = .063in^2$

$A_{nt} = A_{nv}$

Shear

$.6(58ksi)(.063in^2) = 2.2k$

$.6(36ksi)(.094in^2) = 2k$ ← Controls

Tension

$1(58ksi)(.094in^2) = \underline{5.45k}$

Block

$.75(5.45+2) = \underline{5.59k}$

Compression Buckling

$$P_n = F_y A_g \quad [J4\text{-}6]$$

$$= 36\,ksi\,(.491\,in^2)(.9) = \underline{15.9k}$$

Tensile Yield

$$R_n = F_y A_g \quad [J4\text{-}1]$$

$$= \underline{15.9k}$$

Tensile Rupture

$$R_n = F_u A_e \quad [J4\text{-}2]$$

$$A_e = .491 - 2(.125in)(.5in) = .366in^2$$

$$= (58ksi)(.366in^2)(.75) = \underline{15.9k}$$

Shear yielding

$R_n = .6 F_y A_{gv}$ [J4-3]

$= .6(36ksi)(4in)(.125in)(1) = 10.8k$

Shear Rupture

$R_n = .6 F_u A_{nv}$ [J4-4]

$A_{nv} = .5in^2 - 2(.5in)(.125in) = .375in^2$

$R_n = .375in^2(58ksi)(.6)(.75) = 9.79k$

Combined Shear and Tension (Bolt)

$R_n = F'_{nt} A_b$ [J3-2]

$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{nv} < F_{nt}$ [J3-3a]

$F_{nt} = 113ksi$ [J3.2]

$F_{nv} = 68ksi$ [J3.2]

$\phi = .75$

$f_{nv} = 5.63ksi$ [7-1]

$F'_{nt} = 1.3(113ksi) - \frac{113ksi}{.75(68ksi)}(5.63ksi) = 134.43ksi$

$R_n = .75(134.43ksi)\left(\frac{\pi}{4}(.375in)^2\right) = 11.14ksi$