NAU STEEL BRIDGE

Final Project Report

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Joshua Lamphier, Tatianna Smith, Aadil Farried,
M. Eric Barton, Emma Keiser

SB Engineering Co.

SB Engineering Co.

Table of Contents

List of Figures

List of Tables

List of Equations

List of Abbreviations

YLS: Yield Limit Strength
FLS: Fracture Limit Strength

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Plasma Cutting
Andrew Lamer & Mingus Welding, Cottonwood AZ

Welding
Eddie Byron, Phoenix AZ

Equipment
Robin Tuchscherer, PhD, PE, SE, Flagstaff AZ

1.0 Introduction

The goal of this project was to analyze the bridge designed by the 2019-2020 NAU Steel Bridge Team and predict the maximum amount of weight it could hold in six loading scenarios. New connections were designed to outperform these numbers and allow the bridge to hold more weight. For this project, new connection designs were engineered to fit the existing bridge structure, and no changes to the bridge's overall geometry were made. The new capacity of the bridge was calculated based on calculated capacities of the newly designed connections. Full fabrication and reconstruction of the bridge was completed.

To finalize this project, the team will load the new bridge until yielding is observed. Failure data will be collected from the field testing. This data will be compared to the predictions and the findings will be summarized in this report. This project is a combination of steel design and performance studies. The summary of these studies are outlined in this report.

2.0 Technical Work

2.1 Existing Bridge Design Analysis

The existing bridge designed by the 2019 - 2020 NAU Steel Bridge capstone team was analyzed to determine its load capacity and theoretical failure points. This data established a baseline of improvement for this project. The project's focus is to pinpoint failure in the existing connection plates and improve each connection design and capacity. The existing bridge plan set is shown in Appendix A. This plan set contains the overall bridge structure design and connection designs. It is referenced throughout this report and during analysis procedures completed for this project.

2.1.1 Loading Scenarios

For this project, six loading scenarios were analyzed. These loads are a replica of the loads planned for use in the cancelled 2020 and 2021 AISC steel bridge competitions. Each consists of a distributed load over two 4'x4' footprints. These are available in Appendix B. Failure was predicted in each of these loading locations. One of these six scenarios will also be used in the actual loading process of the final bridge design. A dice will be rolled to select one loading point, and the team will replicate the load on the constructed bridge until failure occurs.

2.1.2 Existing Connection Capacities

Each existing connection was analyzed to determine the ultimate tensile capacity, bearing and tearout capacity, and the tensile and shear strength of the bolts. The existing connections are provided in Appendix A. The following table shows the calculated controlling strength capacity for each mounting point of each connection. In the case where there is more than one mounting hole, the hole is assigned a number from 1 to 3. These bolt hole labels are available in the Existing Connections drawing in Appendix A.

Table 1: Capacities for Each Existing Connection

Connection	Controlling Strength Capacity, kips	Description
NONE	0	No design
A1	8.96	Bolt hole tearout
A2	8.96	Bolt hole tearout
B	8.96	Bolt hole tearout
C1	12.66	Bolt hole tearout
C2	8.44	Bolt hole tearout
C3	8.96	Bolt hole tearout
D	5.625	Tensile Fracture
E1	10.02	Bolt hole tearout
E2	8.96	Bolt hole tearout
F	5.625	Tensile Fracture

2.1.2.1 Analysis of Previous Year's Connections

For analysis purposes, each individual connection was paired with its relative member attachment point. This information is available in the Bridge Legend in Appendix A. This figure relates all of the members to their end connections. Initial insight into the existing drawings revealed dimensional discrepancies and incomplete consideration for controlling connection strengths. The connections were redrawn based on as-built specimens in order to properly calculate the predicted capacity of each design. In Appendix A, the dimensions highlighted in red represent the corrected dimensions. On the existing plans, these were either incorrect by the previous team or left blank. The as-built dimensions were used within the equations in the following sections to determine the capacity of each connection and each mounting point. These calculations were done using the load and resistance factor design (LRFD) requirements represented in the following sections.

2.1.2.2 Tensile Strength

To analyze the tensile strength of the connections, Chapter D of the AISC Steel Construction Manual was used to determine the yield limit strength (YLS) and the fracture limit strength (FLS). Equation 1 shows the YLS design tensile strength and equation 2 shows the FLS design tensile strength. Since these both relate to the design strength of the connections plates, the smaller of the two governs the design.

Equation 1: Tensile Yielding in the Gross Section (AISC SCM 15th ed. eq. D2-1)

$$\phi_t P_n = (0.75) F_y A_g$$

ϕ_t = Strength reduction factor for LRFD, 0.75
F_y = Specified minimum yield stress, ksi
A_g = Gross area of member, in^2

Equation 2: Tensile Rupture in the Net Section (AISC SCM 15th ed. eq. D2-2)

$$\phi_t P_n = (0.75) F_u A_e$$

F_u = Specified minimum tensile strength, ksi
A_e = Effective net area, in^2

2.1.2.3 Bearing and Tearout Strength

Bearing and tearout strength at each bolt hole connection was determined in reference to section J3.10 in the AISC Steel Construction Manual. Bearing strength and tearout strength were determined separately using equations 3 and 4 respectively. As done before, the smaller of these two values governs the bearing and tearout strength of the connection.

Equation 3: Bearing: when deformation at the bolt hole at service load is not a design consideration. (AISC SCM 15th ed. eq. J3-6b)

$$\phi_t R_n = (0.75) 3.0 d t F_u$$

ϕ_t = Strength reduction factor for LRFD, 0.75
d = Nominal fastener diameter, in
t = Thickness of connected material, in
F_u = Specified minimum tensile strength of the connected material, ksi

Equation 4: Tearout: when deformation at the bolt hole at service load is not a design consideration. (AISC SCM 15th ed. eq. J3-6d)

$$\phi_t R_n = (0.75) 1.5 l_c t F_u$$

l_c = Clear distance, in direction of force, between the edge of the hole and the edge of the material

2.1.2.4 Tensile and Shear strength of Bolts and Threaded Parts

Tensile and shear strength of bolts and all threaded parts for each bolt hole connection was determined in reference to section J3.6 in the AISC Steel Construction Manual. Due to inaccurate records of the previous team's bridge design, some assumptions about the bolts were made. These assumptions include that the bolts are made of Grade 8 material and that the threads would not be included in the shear planes. Using this information, the value of nominal tensile strength (Fnt) was determined to be 90 ksi from table J3.2. Tensile and shear strength were determined separately using equations 5 and 6 below respectively.

Equation 5: Design Tensile Strength (AISC SCM 15th ed. eq. J3-1)

$$\phi_t R_n = (0.75) F_{nt} A_b$$

ϕ_t = *Strength reduction factor for LRFD, 0.75*

F_{nt} = *Nominal tensile strength, ksi*

A_b = *Nominal unthreaded body area of bolt or threaded part, in*^{2}

Equation 6: Design Shear Strength (AISC SCM 15th ed. eq. J3-1)

$$\phi_t R_n = (0.75) F_{nv} A_b$$

ϕ_t = *Strength reduction factor for LRFD, 0.75*

F_{nv} = *Nominal shear strength, ksi*

A_b = *Nominal unthreaded body area of bolt or threaded part, in*^{2}

2.1.3 Determination of Theoretical Failure Using RISA Software

Modeling was conducted using RISA software to determine bridge failure for each of the six possible load combinations. For the purpose of this project, **failure** is defined as when a connection reaches its capacity and breaks apart beyond bending or deforming. Analysis consisted of increasing the total load on the bridge in RISA until axial loading on a member came within 0.50% of its connection's theoretical capacity. The axial loading data was taken from RISA and compared to the connection capacities in Appendix B. This process was done for each scenario to identify the failure location, the failure type, and the corresponding ultimate load capacity.

The following assumptions were made to perform this analysis:

1. Failure will occur in a connection. This neglects tensile failure or buckling in members.
2. The RISA model is simply supported - pinned on one end and only restricted in the vertical direction on the other. This is the most realistic representation of the real-world support conditions.
3. Any connection mounting locations that are loaded in compression will not fail due to sufficient material backing each bolt hole in the compressive direction, and the stoutness (length to thickness ratio) being very small.

The original bridge was designed to withstand 2500 lbs. Results from analysis found that the bridge would uphold this weight in all six scenarios. Analysis concluded that in all six scenarios, the governing connection was predicted as connection F. Connection F was predicted to experience tensile fracture. Table 2 shows a summary of each predicted yielding point and respective max load for each scenario. These values are the minimum required values that the new connection designs must withstand.

Table 2: Predicted Max Capacities for Existing Bridge, Load Cases 1 - 6

Load Case	RISA Label of Member Associated with Failure	Overall Bridge Capacity (lbs)	Governing Connection	Connection % Loaded
LC1	M64A or M44	3200	F	99.53%
LC2	M62A	3125	F	99.43%
LC3	M62A	2875	F	99.06%
LC4	M42	3500	F	100.27%
LC5	M42	3250	F	100.14%
LC6	M42	3075	F	100.11%

2.2 New Connection Designs

2.2.1 Solutions to Existing Connection Design Flaws

Per the analysis, the connections that yielded first did so due to tensile fracture of the connection material. The next possible failure method is from bolt hole tearout. Tensile fracture is the tearing apart of a steel plate due to necking. Bolt hole tearout is a result of bolt hole proximity to the edge of the connection plate. The following figure represents the methods of failure predicted to occur in the existing connection designs. Tensile fracture is represented by #1 and bolt hole tearout is represented by #2.

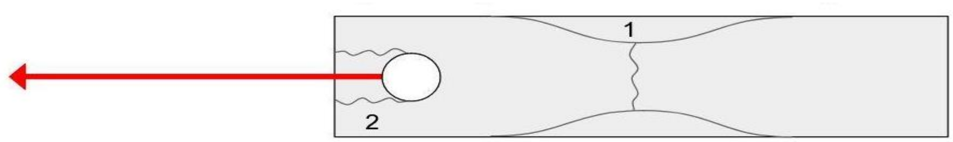


Figure 1: Causes of Failure in Existing Connections

A solution to the tearout strength deficiency would be to add more material around the bolt holes. By adding material between the edge of the plate and the bolt holes, the tearout strength would increase directly proportional to the increase in l_c as seen in Equation 4. In order to increase the tensile strength of the plates, more cross-sectional area must be added or a new material with a higher yield stress must be selected. Cross sectional area can be added by increasing the thickness of the plate, or by simply adding more material to the width of the plate. Each of these solutions also has a direct impact on the overall strength of the connections.

2.2.2. Designing to Withstand Minimum Loading for Each Scenario

Using the previous year's team's connection designs and loading cases, each connection location was analyzed to determine the forces acting in the top and bottom chords as well as the forces in each of the cross members. The previous bridge was also analyzed with the loading cases to see how each connection from their design failed. These two factors were then implemented into the new connection design to withstand the loads that occur in each specific location. It was determined that certain design features were the most feasible for use in the new connection designs. The use of gusset plates and bolt grids were determined to be the most feasible and effective overall.

2.2.3 Designing to Outperform Existing Bridge Performance

It should be noted that all connections were redesigned to maintain within repeatability and constructability engineering standards. Although the existing connection F was the only one predicted to fail, the other existing designs had similar designs that were problematic. New connections were designed to eliminate cases of bolt hole tearout and tensile fracture, and to incorporate industry steel design standards. These include maintaining a minimum distance from bolt holes to the edge of a member or plate, and including more than one bolt for mounting purposes. Last year's connections were all plates that resisted shear forces. See Appendix A for new and existing connection designs along with the overall bridge design. These shear plates experienced axial forces from web members and bending and tensile forces on the top and bottom chords. The new connection designs were considered individually, and the resulting designs depend on the forces exerted by each specific member on the entire connection. The selected designs resist the applied forces and distribute them more effectively. Connection designs were limited by the available space between the members. This was overcome in part by decreasing bolt sizes.

2.2.3.1 Designed Connection Calculations

Since the same design method used for previous calculations (LRFD) was also used for the new connection designs, many of the equations remained the same. Such calculations that remained the same include the tensile YLS and FLS for the member connections and the tensile/shear strength of the threaded bolts. However, since the new connections were designed such that two bolts lie in the tension plane, block shear must be considered. The equation below shows the method used to determine block shear strength from the Steel Construction Manual.

Equation 7: Block Shear Strength (AISC SCM 15th ed. eq. J4-3)

$$\phi R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt}$$

A_{nv} = *Net area subject to shear, in*^{2}

U_{bs} = *Uniform tension stress factor*

A_{nt} = *Net area subject to tension, in*^{2}

For these calculations, the overall reduction factor (φ) was taken as 0.75 from the LRFD specifications while the nonuniform stress distribution factor was taken as 0.5 since the loads are not acting on the connections in a uniform manner. Such block shear calculations were evaluated for multiple failure paths to ensure the different failure modes were considered and to select the lowest strength. The following table shows the calculated capacities for each connection mounting location. The calculations for these capacities are in Appendix C. For connections with "NA" capacities, the ultimate compressive strength cannot be determined according to AISC due to the fact that the loads exerted on these points are compressive [2]. This is due to the complexity of such calculations and lack of available research for AISC [2]. Therefore, it is assumed that the truss members under compression will buckle before the connection plates fail.

Table 3 shows the calculated capacities for the new connections. The connection designs went through a revision process in order to make them more cost-effective and manufacturable. Although some of the new mounting capacities are smaller, these locations are not critically loaded in comparison to other locations. The locations of issue, especially in connection D, are much stronger than last year's design. By focusing the strength in critical areas, cost is saved in the other locations and the bridge is still able to hold an increased load.

Table 3: Calculated Capacities for New Connection Designs

Old Connection Name	New Connection Name	Old Capacity, kip	Revision 1 Capacity, kip	Revision 1 % Stronger	Revision 2 Capacity, kip	Revision 2 % Stronger
E2	A1	10.02	20.6	105.59%	3.87	-61.38%
E1	A2	8.96	11.67	30.25%	3.87	-56.81%
E2	A3	10.02	NA		NA	
C1	B1	8.96	10.86	21.21%	9.46	5.58%
C2	B2	8.44	13.39	58.65%	3.87	-54.15%
C3	B3	12.66	NA		NA	
B	C1	8.96	NA		NA	
B	C2	8.96	NA		NA	
F	D1	5.625	21.94	290.04%	21.94	290.04%
F	D2	5.625	NA		NA	

2.3 Modeling and Analysis of the New Design

2.3.1 SolidWorks Connection Models

Each new connection design was 3D modeled in SolidWorks. Dimensions were checked for compatibility and the final connection drawings were created from their models. Each member was also modeled in SolidWorks so that a complete bridge assembly could be modeled and drawn out for reference. Initial and revised drawings are available in the plan set in Appendix A.

2.3.2 Determination of Theoretical Failure of New Design using RISA

With the new top and bottom chords remaining as continuous members, the RISA model was changed to accommodate the distribution of forces. Ends of the top and bottom chords were modeled with reactions along their spans, instead of hinges as previously modeled.

2.3.3 Prediction of New Max Load Capacity

RISA was used to calculate the internal truss member forces, which were again compared to the capacities of each respective connection. These calculations are available in APPENDIX #. The following table shows the calculated total load based on the capacities of each connection. It was determined that the governing connection in each case was connection A at mounting location 2, which connects to the angled truss members. Block shear will occur in these plates near the ultimate load capacity in each scenario.

Table 4: Ultimate Load Capacity for New Designs

Load Case	Old Ultimate Load Capacity, lbs	New Ultimate Load Capacity, lbs	Connected Member	Governing Connection	% Increase in Strength Over Existing Bridge
LC1	3200	3569	M61	A2	42.75%
LC2	3125	4021	M61	A2	60.82%
LC3	2875	3568	M67B	A2	42.71%
LC4	3500	3723	M67B	A2	48.94%
LC5	3250	3378	M67B	A2	35.13%
LC6	3075	3161	M67B	A2	26.46%

In all cases, the new capacity of the bridge is increased from the previous design. The maximum percent increase is 60.82% and the least percent increase is 26.46%. This table shows that the goal to increase the capacity of the bridge by re-designing the connections was accomplished.

2.3.4 Prediction of New Failure Points

New yielding is predicted in all cases to be the result of block shear in the web members of the bridge truss. This information is shown in columns two and three of Table 4.

2.4 New Plan Sets

2.4.1 New Overall Bridge Plan Sets

A new drawing that represents the overall assembly of the bridge – including members and connection locations – was created using SolidWorks. This was attached to the existing plan sets and are available in Appendix A. This drawing includes new detail locations that correspond to the naming convention for the new connections.

2.4.2 New Connection Plan Sets

A new set of connection details was created for the new connection designs to show the dimensions of each connection design on the bridge and the connection details individually. These are represented in Appendix A at the end of the existing plan sets. Revision 1 was replaced with Revision 2 drawings for feasibility of manufacturing and cost effectiveness. Although Revision 2 capacities were in some cases lower than last year's connections, the critical force locations had greater capacities and thus the new overall design still theoretically held more than the original bridge as shown in Table 4.

2.5 Construction Materials

2.5.1 Steel Tubing

Field observations indicated that the existing steel members had been drilled into for bolt mounting locations. This is shown in Figures 2 and 3. The new connection designs have different dimensions and mounting styles. Existing connections were also welded to the vertical truss members, therefore new steel was required.

Figure 2: Existing Connection B Conditions

Figure 3: Existing Connection E Conditions

New steel was obtained from Page Steel to replicate the existing members and allow for new bolt holes to be drilled in the final bridge. The following table shows the bill of materials for the required square tubing to rebuild the bridge, which was provided to Page Steel.

Table 5: Bill of Materials for Steel Members

Advance Bill of Materials					
Project Name: 20-21 NAU Steel Bridge					
Job Number: Members					
Profile	**Quantity**	**Grade**	**Length**	**Length (inches)**	**Total Length (inches)**
HSS1x1x0.065	6	A500	1'-8"	20	120
HSS1x1x0.065	6	A500	3'-3 1/2"	39.5	237
HSS1x1x0.065	18	A500	3'-3"	39	702
HSS1x1x0.065	16	A500	0'-10 1/2"	10.5	168
HSS1x1x0.065	10	A500	3'-4"	40	400
HSS1x2x0.065	6	A500	3'-4 1/2"	40.5	243
HSS1x2x0.065	10	A500	3'-4"	40	400
HSS0.75x0.75x0.065	29	A500	3'-3"	39	1131

2.5.2 Sheet Steel

The final connection designs, represented by the Revision 2 drawings, were to be created from 11-gauge sheet steel. It was determined that 16 square feet of steel was required to fabricate each plate. The following bill of materials was sent to Page Steel for the required material.

Table 6: Bill of Materials for Sheet Steel

Advance Bill of Materials		
Project Name: 20-21 NAU Steel Bridge		
Job Number: Connections		
Profile	**Quantity (ft^2)**	**Grade**
1/8" Sheet Steel	16	ASTM 1011 Grade 50

2.5.3 Hardware

The following bolts were acquired from Copper State Nut & Bolt in order to fully assemble the bridge at each connection.

Table 7: Bill of Materials for Bolt Hardware

Advance Bill of Materials				
Project Name: 20-21 NAU Steel Bridge				
Job Number: Fasteners				
Quantity	**Grade**	**Length**	**Type**	**Diameter**
298	Grade 8-ASTM A490	2"	Half Threaded Bolt	¼”
298	Grade 8-ASTM A490	NA	Typical Nut	¼”
596		NA	Washer	¼”

2.5.4 All Other Miscellaneous Materials

Drill bits, cutoff wheels, and cutting oil, files, and W-40 were all purchased in order to fabricate the bridge. PPE was also purchased for fabrication.

2.6 Fabrication

2.6.1 In-House Fabrication

Cutting of steel members occurred in-house at the NAU Farm. Jigs were created out of wood to ensure each member was cut to the correct length and angle if applicable. Bolt holes for the connection plates and members were also drilled in-house with a drill press and ¼" bit.

2.6.2 Outsourced Fabrication

2.6.2.1 Plasma Cutting

The connection plates were cut on a plasma table by Andrew Lamer with Mingus Welding in Cottonwood, AZ. Andrew also provided guidance in adding tabs to the connections to ensure the bottom plates would lock in place with the side plates and provide extra strength, as shown in Figure 4.

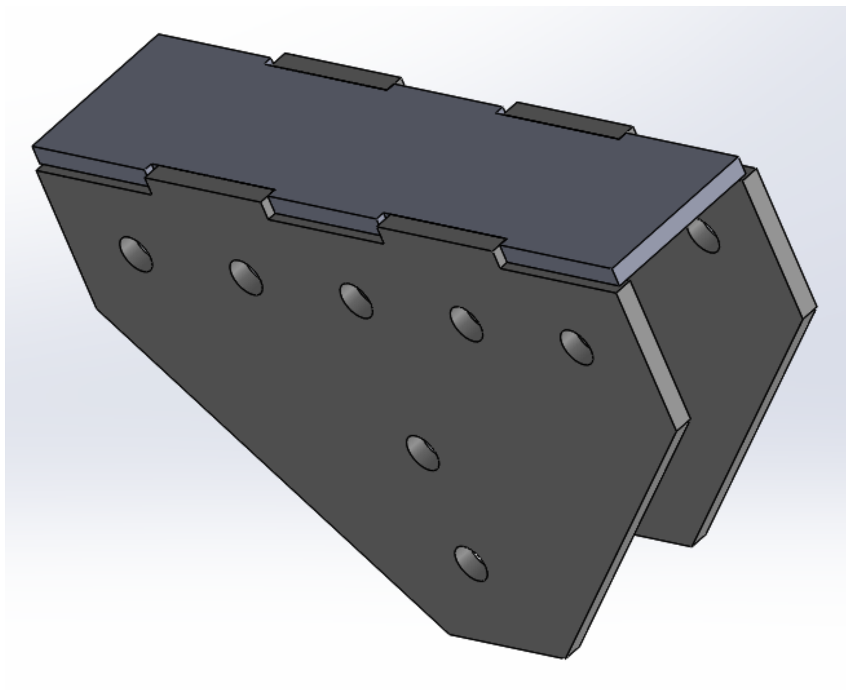


Figure 4: Manufactured Connection Design

The following table shows the part list provided to Mingus Welding. This table lists the required connection side and bottom plates to complete the entire bridge. The part numbers correspond to the Revision 2 plan set in Appendix A.

Table 8: Bill of Materials for Plasma-Cut Plates

Advance Bill of Materials	
Project Name: 20-21 NAU Steel Bridge	
Job Number: Connections	
PART NO.	QTY
A-1	12
B-1	8
0.5B-1	8
C-1	12
D-1	16
A-2	10
B-2	14
C-2	4

2.6.2.2 Welding

Welding of the connections was completed by Eddie Byron of Phoenix, AZ, who has several years of experience in the field. Welds were completed to attach the side and bottom plates as shown in Figure 4 and per the Revision 2 plan set in Appendix A. Welds were placed along the edges of the side and bottom plates everywhere except over the tab locations.

2.7 Bridge Assembly

The bridge was assembled in-house when all materials were available and cut to size. Trusses were assembled before bolt holes were drilled in order to ensure fitment. Once it was determined that all members and connections would fit together without issue, the bolt holes were cut, and the trusses were assembled.

2.8 Loading Bridge to Failure

Loading for the bridge was performed at "The Farm" on the southend of NAU's Flagstaff campus using water to simulate loading. To achieve this, two water tanks were placed at the determined locations for loading and filled until failure occurred. In order to determine the locations for loading, a single 6-sided die was rolled with the die's number matching 1 of the 6 load combinations. Load combination 2 was rolled and can be seen in the photographic evidence provided in Appendix E. Both water tanks were filled one at a time in rotation to specific volume indicators in order to mimic the load distribution between each location. Both water tanks were filled until the bridge suffered catastrophic failure with note being taken for the volume of water present in each tank and the measured vertical deflection of the bottom chord between the two loads.

2.9 Performance Report

2.9.1 Data from Loading and Failure

Provided below in Table 9 is the calculated total load the bridge reached in capacity before failure. Variables include the volume of water in each tank in gallons, the density of water in pounds per gallon, the calculated weight for the water in each tank, the weight of the tanks dry, and total combined weight of loading at failure.

Table 9: Bridge Load

	Small Tank	Large Tank
Volume (gal)	220	500
Water Density (lb/gal)	8.345	8.345
Water Weight (lbs)	1835.9	4172.5
Tank Weight (lbs)	80	100
Total Weight (lbs)	6188.4	

2.9.2 Predicted Versus Actual Results

The team rolled a die and load case 2 was selected from testing. This scenario required two distributed loads to be placed at 8 feet and 12 feet from the end of the bridge with each distributed load spanning 3 feet. The predicted capacity for the scenario was 4021 pounds with a deflection of 1.193 inches. Failure was predicted to be block shear at connection A2 where it connects to the web member at the far end of the truss. Actual results concluded that the bridge held 6188 pounds with a deflection of 5 inches and the failure mode was block shear at connection A2 where it connects to the web member at the far end of the truss. Photographic evidence of the actual failure is provided in Appendix E.

2.9.3 Updated Design Versus the Original Design

The updated connection design significantly increased the load capacity for the entire bridge. Predicted capacity for the new connection design was 4021 pounds, which is 896 pounds more than the original design's prediction of 3125 pounds. Actual capacity exceeded predictions by holding 6188 pounds.

2.9.4 Conclusion

When looking at the initial prediction to actual results, the updated connection design was able to increase the load capacity of the bridge by 3063 pounds, resulting in a 98% improvement in load capacity. Actual results surpassed expectations as well by holding 2167 pounds more than the new connection capacity prediction, resulting in a 54% improvement. Deflection increased from the new connection prediction of 1.193 inches to an actual deflection of 5 inches, resulting in a 319% increase over prediction. Location and type of the failure were consistent with prediction, deeming the analysis of failure to be considered accurate. Differences in the modeled predictions versus actual results were likely due to conservative estimates in the model supports and the actual material strength being higher than specified. This could be determined by taking a sample of the material and determining its actual tensile strength by loading it to tensile failure in a lab setting.

3.0 Impacts

The scope of constructing a steel bridge at a 1:10 scale will incur various environmental, economic, and social impacts. The economic impacts focus directly on the companies such as Page Steel who donated material and Mingus Welding who donated time and fabricated the connections for the bridge. The sponsorship from these two avenues have been honored by the team with having their company and names labeled onto the report and presentation as a symbol of recognition. This will increase revenue and exposure for Page Steel because their name will reach a larger audience and opportunities for future services are increased. Mingus Welding students also gained experience from interaction with engineers and fabrication of connections. The donation of material and time could also be seen as a negative economic impact as Page Steel must still pay their workers for duties that do not receive payment directly in return. Other economic impacts include the cost of building a bridge, even at this scale, does cost money for materials and labor, but in turn provides ideas for a full-scale bridge that can save money in the future.

Social impacts is a category that the team experienced the most directly because each member was able to have opportunities for social interaction and create an image of Northern Arizona University that was positive. These social interactions also allowed for opportunities of education for the team to see and experience local employment facilities. The employers experienced social impacts by putting their image on the project and depending how the bridge performs will in turn influence the image of their company as well. The scaled bridge can also allow for a beneficial social impact by providing ideas for safe, long-term, durable structures that can minimize site disruption, environmental impacts, traffic congestion, and accelerated bridge construction.

Environmental impacts cover an enormously large category for this particular project. Starting with the fabrication of the steel in general, some of the negative impacts include the mining of the iron ore. It is highly intensive and causes large amounts of air pollution from the diesel generators, transportation trucks and other types of equipment. Water impacts are a major factor from the mining of materials because the heavy metals and acid drains from the mines into water sources. In order to create the steel, major amounts of energy are required to input large amounts of coal. The coal emits air pollution that causes cancer, and it creates wastewater that is highly toxic and contains large amounts of organic compounds. The team’s specific impact on the creation of steel is very minor considering the amount of steel needed to build the bridge is small, however, the team did contribute to air pollution due to the needed transportation of materials to and from facilities for manufacturing and fabrication. The use of machinery to manufacture and fabricate the steel into members and connections for the bridge does release toxins into the air as well. The scale of all these impacts are very minor considering the scale of the bridge in general, but it is important to consider these factors as it does influence future projects of bigger sizes.

4.0 Exclusions

The team was not responsible for any tasks that are outside of the project scope. Elements of engineering design that were out of the scope of this project include, but are not limited to: foundation design, geotechnical analysis, hydrology considerations, and surveying. The team was solely responsible for the design, analysis, fabrication, construction, and management of the project.

5.0 Schedule

5.1 Tasks

The major tasks for this project along with their time spans were due diligence from January 25th to February 11th, design development from February 12th to February 26th, structural analysis from March 1st to March 15th, plan sets from March 16 to March 17, fabrication from March 18th to March 29th, and project management from January 25th to April 30th. These tasks along with their associated subtasks are outlined above in section 2.0. Deliverables from the tasks above included a 30% Report and 30% Presentation on February 9th, 60% Report and 60% Presentation on March 9th, and 90% Report and 90% Presentation on April 8th, 90% website on April 12th, Final Presentation on April 15th, and Final Report and Website on April 27th.

5.2 Critical Path

The critical path for this project involved all tasks that are necessary to complete the final project. These tasks are major items that were completed in preceding order. The critical path is as follows:

1. Task 2.1: Existing Bridge Design Analysis
2. Task 2.1.2 Existing Connection Capacities
3. Task 2.2: New Connection Design
4. Task 2.3 Modeling and Analysis of the New Design
5. Task 2.4 New Plan Sets
6. Task 2.5 Construction Materials
7. Task 2.6 Fabrication
8. Task 2.7 Bridge Assembly
9. Task 2.8 Loading Bridge to Failure
10. Task 2.9 Performance Report

A delay in any of the tasks included in the critical path caused significant delay in the entire project. It was critical that the team analyzed the existing connection capacities, designed new connections, and created plan sets so the acquisition of materials and fabrication could be initiated. The team had issues with the manufacturer to provide the needed steel in time, which put the team greatly behind schedule going into the 90% report and 90% presentation.

5.3 Comparison to Proposal

The project had to be reconsidered in the beginning of January when the AISC Steel Bridge Competition was cancelled. The schedule completely changed because the team's new goal shifted from building an entire bridge to analyzing the existing connections of last year's bridge and designing higher capacity ones. The team went from having a schedule built out for the entire fall and spring semester, to having to fit the new design considerations into the spring semester.

Major tasks for this project involved all tasks that are necessary to complete the final project. A delay in any of the tasks included in the critical path caused significant delay in the entire project. The team had issues with the manufacturer to provide the needed steel in time, which put the team greatly behind schedule. The project had to be reconsidered in the beginning of January when the AISC Steel Bridge Competition was cancelled. It was critical that the team analyzed the existing connection capacities, designed new connections, and created plan sets so the acquisition of materials and fabrication could be initiated.

6.0 Staffing Plan

6.1 Required Positions

This project required the following positions to be filled by each team member. No one member was assigned a specific staffing position. Each team member took on each of the following position's responsibilities.

Senior Engineer - SENG
Engineer - ENG
Engineer in Training - EIT
Lab Technician - LAB
Administrative Assistant - AA

6.2 Team Qualifications

M. Eric Barton

Eric has worked for multiple construction companies in various project management related positions. In addition to office related work experience, he also has field experience as a working foreman which will help with the construction of the bridge for competition. From studies at NAU, Eric has taken classes in statics, mechanics of materials and structural analysis which help correlate the understanding elements of bridge design into practice.

Mohammed Aadil Farried

Aadil has almost four years of administrative (Level 3) work experience from working at the Center for International Education (CIE) at NAU and one year work experience at HSBC

Colombo Sri Lanka. Aadil has taken steel design, architecture, and mechanics of materials which correlates directly to the bridge design project.

Emma Keiser

Emma has worked in CNC machining, manufacturing, and currently holds an internship position with ADOT in construction. Her experience with project and construction oversight contributes to project management of this project. In addition, her metalworking experience is relevant towards the project's fabrication and construction requirements. She has taken NAU coursework in structural analysis and materials science.

Joshua Lamphier

Josh has worked on the transportation design project for the Pacific Southwest Conference in the past. He has also taken a steel design class at NAU that directly correlates to the project being conducted. Lastly, he has completed an internship with Civiltec Engineering that lends experience for the project in general.

Tatianna Smith

Tatianna has worked for the Bureau of Reclamation as an intern in the concrete dams department, where she learned to become sufficient in AutoCAD, Civil 3D, and Revit. Through this job she has also gained project manager experience, leading a team to the construction of the addition to Guayabal Dam in Puerto Rico.

6.3 Work Plan

The project required the services of an engineer in training, engineer, senior engineer, lab tech, and an administrative assistant. The Engineer in Training was a part of the analysis and fabrication for the project. They helped do research, analysis, and design alongside the interns. The EIT attended the meetings as well as helped the team to create a final design report and website. The engineer was responsible for the overall progress of the project at all the stages of development. This includes ensuring the team stayed on schedule and on budget. It also included monitoring the analysis, fabrication, and contacting various companies to request funds, materials, and services. The senior engineer provided the final check on all the milestones of the progression of the project before it continued. This involved reviewing reports, designs, calculations, and assisting in the design analysis. The lab tech focused directly on fabrication management and the creation of shop drawings. The administrative assistant did most of the fundraising and provided research materials for the project. They frequently attended meetings, in which they took notes and created agendas. The administrative assistant also handled public relations and project imagery. The breakdown of each position's estimated work for this project is shown in Appendix D.

6.4 Summary of Staffing Plan

The following rates are based on the average rates of engineers in Arizona plus an additional 175% to account for overhead costs.

Table 10: Position Billing Rates

Position Billing Rate ($/hr)	
SENG	$210
ENG	$150
EIT	$80
LAB	$100
AA	$55

Senior Engineer

The senior engineer contributed approximately 59.5 hours to the project and was billed at $210/hr. The senior engineer is licensed and has over 10 years of working experience in structural engineering or related fields. On this project, the senior engineer was responsible for design oversight, and critical review. The senior engineer approved the final design decisions and plans. The total cost to staff a senior engineer on this project was estimated at $12,495.00.

Engineer

The engineer contributed approximately 108.25 hours to the project and was billed at $150/hr. The engineer is licensed and has over 4 years of experience. The engineer was responsible for design development and project management. The engineer also communicated with all relevant parties regarding the project development and fabrication. The total cost to staff the engineer on this project was estimated at $16,237.50.

Engineer in Training

The EIT contributed approximately 83.25 hours to the project and was billed at $80/hr. The EIT possesses a degree and EIT certification. The EIT was responsible for project communication and design development. The EIT was involved in all aspects of the project, not including drafting services. The total cost to staff an EIT on this project was estimated at $6,660.00.

Lab Technician

The lab technician contributed approximately 102.25 hours to the project and was billed at $100/hr. The lab technician has at least 5 years of experience in drafting. The lab tech was responsible for creating the design’s plan sets and managing fabrication processes. The total cost to staff a lab technician on this project was estimated at $10,225.00.

Administrative Assistant

The administrative assistant contributed approximately 41.5 hours to the project and was billed at $55/hr. The administrative assistant has at least 2 years of experience in an administrative field. The administrative assistant was responsible for project fundraising, document management, public relations, and project visuals. The total cost to staff an administrative assistant on this project was estimated at $2282.50.

6.5 Comparison to Proposal

In the proposal, the staffing plan consisted of the senior engineer contributing approximately 80 hours to the project and the total cost to staff a senior engineer on this project was estimated at $16,800.00. The engineer contributed approximately 194 hours to the project and the total cost to staff an engineer on this project was estimated at $29,100.00. The EIT contributed approximately 152 hours to the project and the total cost to staff an EIT on this project was estimated at $12,160.00. The lab technician contributed approximately 158 hours to the project and the total cost to staff a lab technician on this project was estimated at $15,800.00. The administrative assistant contributed approximately 72 hours to the project and the total cost to staff an administrative assistant on this project was estimated at $3,960.00. The values that are presented above for the current staffing cost is based on the actual hours that the team performed over the semester. The work log and final hours per position are located in tables in Appendix D.

7.0 Cost of Engineering Services

7.1 Personnel Cost

Total personnel cost for the design and fabrication of the bridge are seen in the table below and yield a total cost of $47,900.

Table 11: Personnel Cost

Position	Total Hours	Cost Per Hour	Total Cost Per Position
SENG	59.5	$210	$12,495.00
ENG	108.25	$150	$16,237.50
EIT	83.25	$80	$6,660.00
LAB	102.25	$100	$10,225.00
AA	41.5	$55	$2,282.50
		Total:	$47,900.00

7.2 Additional Cost

Materials

Materials used to complete the construction and fabrication of the bridge are seen below. Steel cost purchased from Page steel totals $579.22 and fastener costs from Copper State totals $65.81. The total cost for materials used in the project is $645.22.

Table 12: Steel Cost Estimate

Steel Cost Estimate				
Quantity	**Description**	**Length**	**PLF Pricing**	**Price**
5	1x1x0.065 hss	20	1.38	138.15
3	1x1x0.065 hss	24	1.38	99.47
3	2x1x0.065 hss	24	2.02	145.26
5	0.75x0.75x0.065 hss	20	0.92	91.80
1	48"x48"x11-gauge sheet	N/A	104.55	104.55
			Total:	579.22

Table 13: Fastener Cost Estimate

Fastener Cost Estimate					
Quantity	**Grade**	**Length**	**Type**	**Price/Unit**	**Total Price**
300	8-Zinc Coated	2"	Half Threaded	$0.22	$66.00

<u>Equipment</u>

Equipment purchased for use in fabrication of the bridge including hardware, PPE, etc. is seen in the table below and totals $123.57.

Table 14: Equipment Cost

Equipment Cost		
Supplier	**Details**	**Price**
Homco	Drill bits and cutting oil	$50.00
Home Depot	Chuck key for drill press	$2.72
Home Depot	PPE, files, WD40 and cutoff disks	$70.85
	Total:	$123.57

<u>Subcontracting</u>

Subcontracting used for fabrication includes outsourcing of connection plate plasma cutting to Mingus Welding and connection plate welding to Eddie Byron. The total cost of such services totals $820 and is seen in the tables below.

Table 15: Welding Cost

Welding Cost		
Cost per Hour	**Total Hours**	**Total Cost**
$60	7	$420.00

Table 16: Plasma Cutting Cost

Plasma Cutting Cost			
Art/CAD fee	**Cutting Cost per Hour**	**Total Hours**	**Total Cost**
$50	$100	3.5	$400

<u>Travel</u>

Travel completed during the project includes trips to Cottonwood to drop off and pick up materials needed for bridge connections. Cost of such trips are based on personal vehicle costs and totals $57.20 per trip. The total cost for three trips was $171.60

7.3 Total Cost

The total cost for the project to-date is $49,660.

7.4 Comparison to Proposal

The proposal consisted of the AISC competition costs in addition to the material, subcontracting, and equipment costs. Therefore, the proposal had a much larger overall cost estimation than the current cost estimate gathered without the competition. The cost of materials was initially predicted to be around $2000 for the members, bolts, and connections. The equipment estimation was predicted to about $500 for the various drilling and grinding attachments. Subcontracting costs were predicted to about $240 in total to weld the particular parts together. The travel portion was significantly higher because the van rental was estimated to be at $500, cost of hotel estimated at $2700 for three nights, and food services estimated at $1120 for four days. Total cost for the proposal was $132,396.

8.0 Conclusion

This project utilized the bridge designed by the 2019/2020 team and improves on aspects such as connection design to create a stronger overall design. The bridge was then analyzed under the new design and compared to the existing design analysis. Lastly the bridge was built by the team and loaded to failure in order to compare the expected values to the actual observed values and produce a performance report. In field testing, it was determined that the bridge failed in the predicted location. However, the bridge held much more weight than predicted and deflected much more before yielding. This was likely due to the conservative estimates in the model and the capacity calculations. This could have also been due to the material strength being higher than 60ksi, as material strengths are specified as the minimum required strength. This would have increased the capacity of each connection and member.

References

[1] Steel Construction Manual, 15th ed. 2017.

[2] M. C. H. Yam and J. J. R. Cheng, "Behavior and design of gusset plate connections in compression," *Journal of Constructional Steel Research*, vol. 58, no. 5-8, pp. 1143–1159, Jan. 2002.

Appendices

Appendix A: Plan Sets

Appendix B: Analysis of Existing Bridge Connections

Appendix C: Analysis of New Bridge

Appendix D: Administrative Documents

Appendix E: Field Testing Data

Appendix A - Existing Bridge Information

Contents:

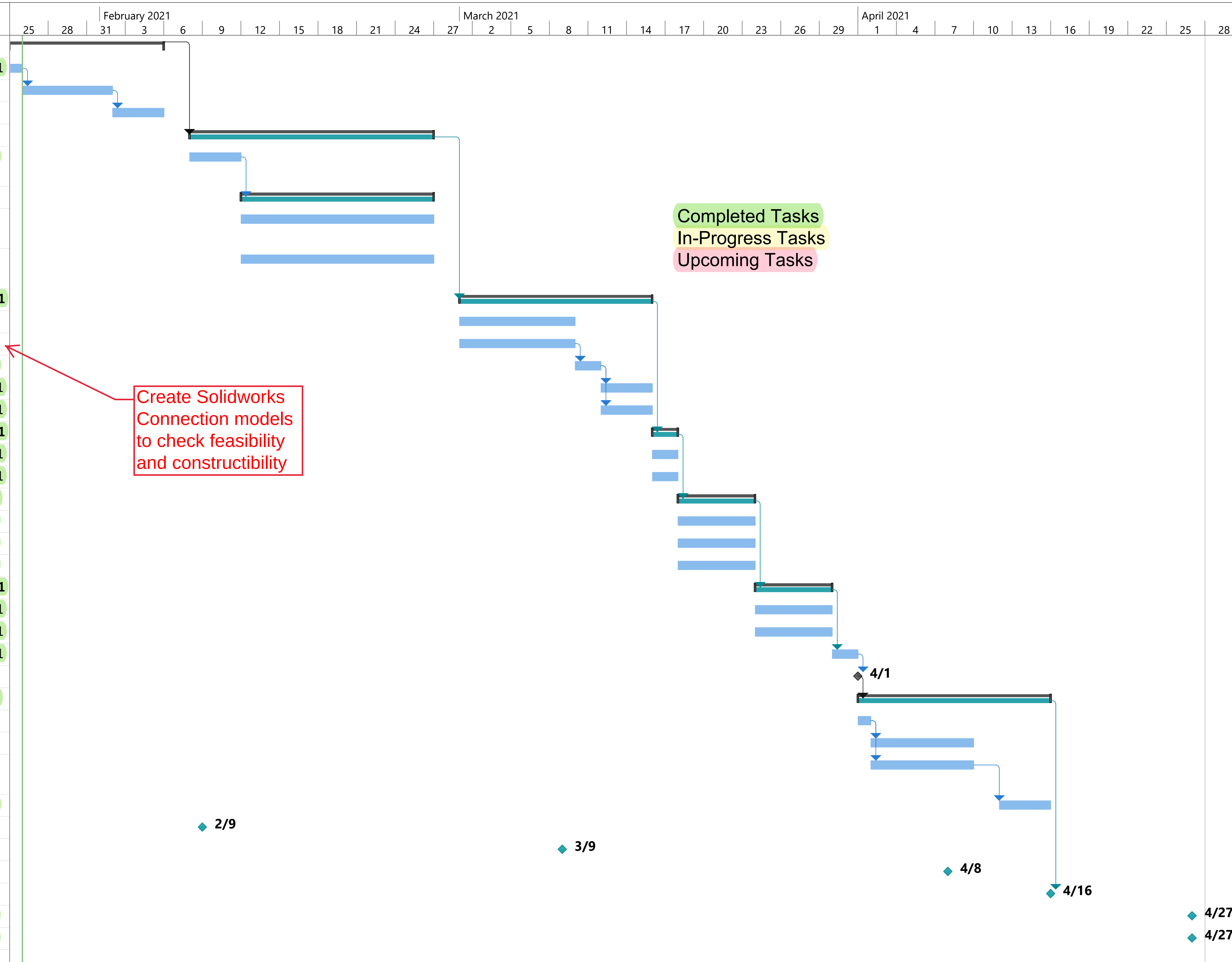
- 2019-2020 Plan Set with New Revisions
- Existing Connections Redrawn and Labeled
- Bridge Legend

PROPRIETARY AND CONFIDENTIAL			UNLESS OTHERWISE SPECIFIED:		NAME	DATE
			DIMENSIONS ARE IN INCHES TOLERANCES: FRACTIONAL ± ANGULAR: MACH ± BEND ± TWO PLACE DECIMAL ± THREE PLACE DECIMAL ±	DRAWN		
				CHECKED		
				ENG APPR.		
				MFG APPR.		
			INTERPRET GEOMETRIC TOLERANCING PER:	Q.A.		
			MATERIAL	COMMENTS:		
	NEXT ASSY	USED ON	FINISH			
	APPLICATION		DO NOT SCALE DRAWING			

TITLE:

SIZE	DWG. NO.	REV
B	Appendix A	
SCALE:	WEIGHT:	SHEET 1 OF 1

ID	Task Name	Start	Finish
1	**Analyze current bridge**	**Mon 1/25/21**	**Fri 2/5/21**
2	Select four loading points	Mon 1/25/21	Mon 1/25/21
3	Model and load bridge to failure at each point	Tue 1/26/21	Mon 2/1/21
4	Identify weaknesses in existing connection desi	Tue 2/2/21	Fri 2/5/21
5	**Overhaul existing connections**	**Mon 2/8/21**	**Fri 2/26/21**
6	Identify solutions to existing connection design flaws	Mon 2/8/21	Thu 2/11/21
7	**Engineer connections**	**Fri 2/12/21**	**Fri 2/26/21**
8	Design connections to withstand minimum loading at all hypothetical loading points	Fri 2/12/21	Fri 2/26/21
9	Design connections to outperform exisitng bridge performance	Fri 2/12/21	Fri 2/26/21
10	**Model and test new design**	**Mon 3/1/21**	**Mon 3/15/21**
11	Create RISA 3D model for bridge	Mon 3/1/21	Tue 3/9/21
12	Create ~~RISA 3D~~ model for connections	Mon 3/1/21	Tue 3/9/21
13	Load model at all four loading points	Wed 3/10/21	Thu 3/11/21
14	Predict max load capacity	Fri 3/12/21	Mon 3/15/21
15	Predict failure points	Fri 3/12/21	Mon 3/15/21
16	**Create plan sets**	**Tue 3/16/21**	**Wed 3/17/21**
17	Create new plan set for overall design	Tue 3/16/21	Wed 3/17/21
18	Create new plan set for connections	Tue 3/16/21	Wed 3/17/21
19	**Acquire materials for construction**	**Thu 3/18/21**	**Tue 3/23/21**
20	Collect steel	Thu 3/18/21	Tue 3/23/21
21	Collect hardware	Thu 3/18/21	Tue 3/23/21
22	Collect all other miscellaneous materials	Thu 3/18/21	Tue 3/23/21
23	**Fabricate parts**	**Wed 3/24/21**	**Mon 3/29/21**
24	In-house fabrication	Wed 3/24/21	Mon 3/29/21
25	Outsourced fabrication	Wed 3/24/21	Mon 3/29/21
26	**Assemble bridge**	**Tue 3/30/21**	**Wed 3/31/21**
27	**Load bridge to failure**	Thu 4/1/21	Thu 4/1/21
28	**Create performance report**	**Thu 4/1/21**	**Thu 4/15/21**
29	Gather data from loading and failure	Thu 4/1/21	Thu 4/1/21
30	Compare predicted versus actual results	Fri 4/2/21	Fri 4/9/21
31	Compare the performance of the updated design versus the original design	Fri 4/2/21	Fri 4/9/21
32	Compile report based on findings	Mon 4/12/21	Thu 4/15/21
33	**30% Presentation and Report**	Tue 2/9/21	Tue 2/9/21
34	**60% Presentation and Report**	**Tue 3/9/21**	**Tue 3/9/21**
35	**90% Presentation and Report**	Thu 4/8/21	Thu 4/8/21
36	**UGRADS Presentation**	Fri 4/16/21	Fri 4/16/21
37	**Website**	Tue 4/27/21	Tue 4/27/21
38	**Final Report**	Tue 4/27/21	Tue 4/27/21
39			



Project: Updated Project Sched
Date: Tue 1/26/21

Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Deadline | Progress | Manual Progress

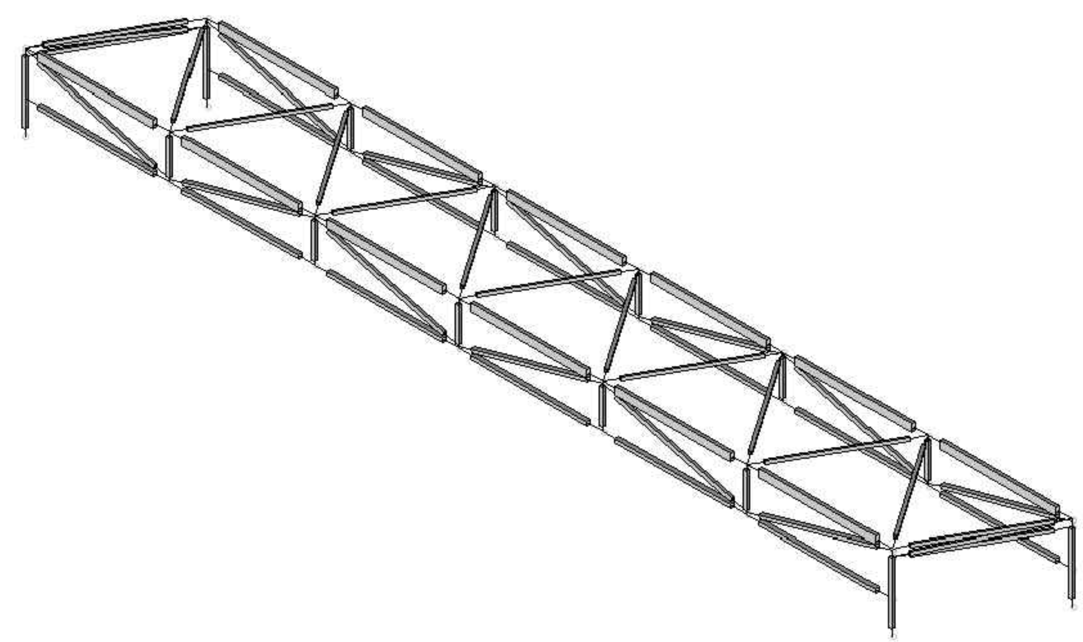
2020 NAU AISC STEEL BRIDGE

CENE 486: Senior Design

Prepared for:
Mark Lamer, P.E
Northern Arizona University

Prepared by:
Emalee Sena
Hailley Ndubizu
Samantha Cole
Steven Bloomfield

Sponsored by:
K-Zell Metals
Page Steel
Copper State Nut & Bolt Co.
Mingus Highschool Welding Team



Notes:

- All bolt holes are $\frac{7}{16}$" in diameter.
- All bolt hole dimensions identified are outer edge of steel to center of hole.
- All bolt holes go through both walls of steel.

Abbreviations:

- TYP. = Typical
- ø = Diameter
- R = Radius

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NO.	DATE:
1	2/1/2020
2	2/13/2020
3	3/8/2020

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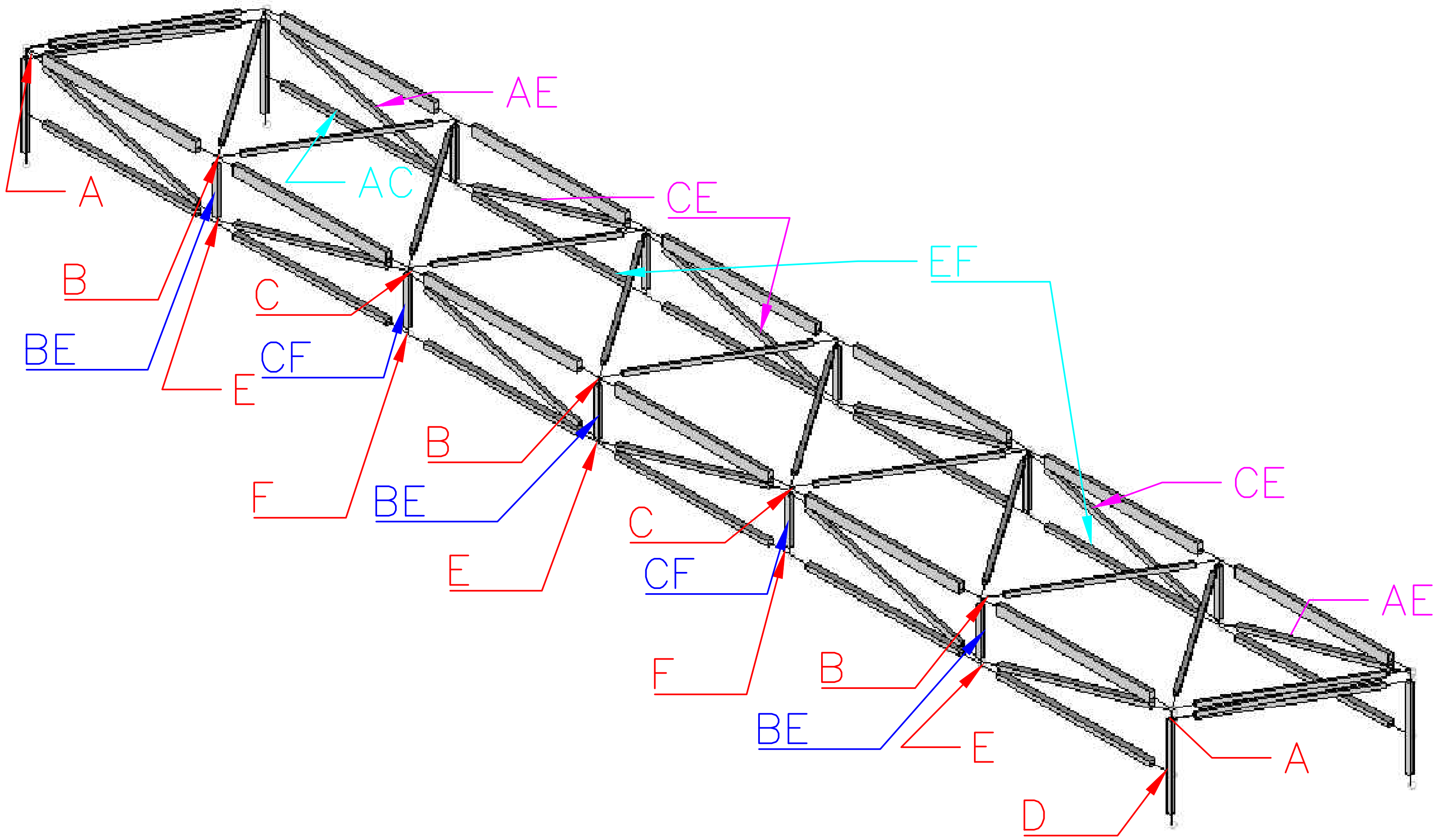
DATE: 3/10/2020
SCALE: NTS

2020 STEEL BRIDGE TEAM
60% PLAN SET



Truss Member Schedule

Member	Material	Length	Quantity
AA	1"x1"x.065" HSS Tube	1'–8"	6
AB	1"x2"x.083" HSS Tube	3'–4 1/2"	6
AC	1"x1"x.065" HSS Tube	3'–3 1/2"	6
AE	1"x1"x.065" HSS Tube	3'–3"	6
BC	1"x2"x.083" HSS Tube	3'–4"	10
BE	1"x1"x.065" HSS Tube	0'–10 1/2"	8
CE	1"x1"x.065" HSS Tube	3'–3"	10
CF	1"x1"x.065" HSS Tube	0'–10 1/2"	6
EF	1"x1"x.065" HSS Tube	3'–4"	10

Lateral Bracing Member Schedule

Member	Material	Length	Quantity
BR	.75X.75X.065 HSS TUBE	3'–3"	20

Connection Schedule

Member	Area (squ. inch)	Quantity
A	1'–5 7/8"	8
B	1'–1 1/8"	12
C	2'–2 11/32"	8
D	0'–7 1/16"	8
E	1'–1 1/16"	12
F	0'–7 1/8"	8
Tab	0'–2 13/16"	20

Note: All Connections are ASTM 1011 Steel with a yield strength of 50 ksi.

~SHEET~ 2 OF 10

Northern Arizona University
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1	2/1/2020
2	2/13/2020
3	3/8/2020

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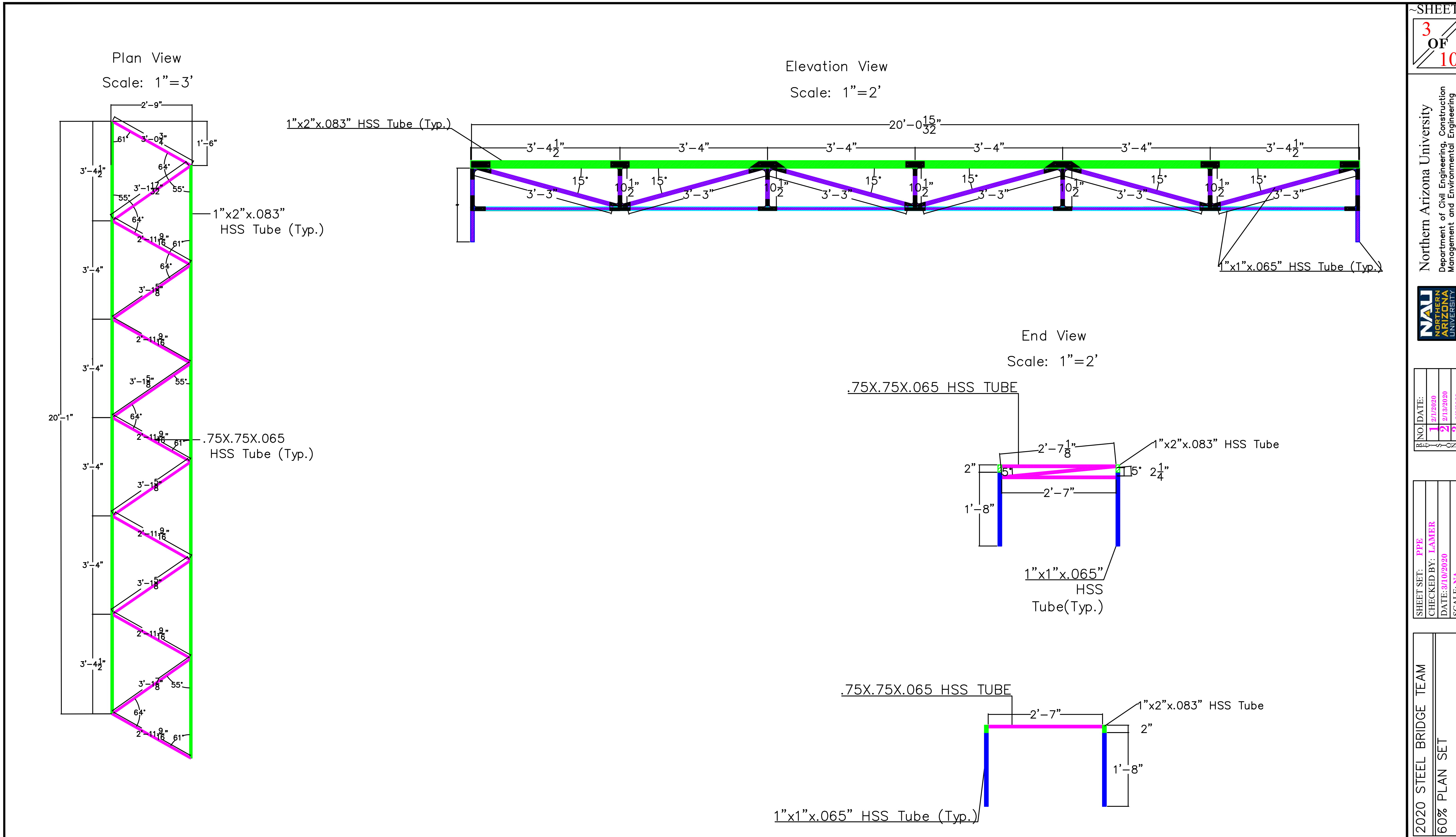
DATE: 3/10/2020
SCALE: NTS

2020 STEEL BRIDGE TEAM
60% PLAN SET


Plan View
Scale: 1"=3'
1"x2"x.083" HSS Tube (Typ.)
.75X.75X.065 HSS Tube (Typ.)
20'-1"
Elevation View
Scale: 1"=2'
1"x2"x.083" HSS Tube (Typ.)
1"x1"x.065" HSS Tube (Typ.)
End View
Scale: 1"=2'
.75X.75X.065 HSS TUBE
1"x2"x.083" HSS Tube
1"x1"x.065" HSS Tube(Typ.)
.75X.75X.065 HSS TUBE
1"x2"x.083" HSS Tube
1"x1"x.065" HSS Tube (Typ.)
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3 OF 10
Northern Arizona University
Department of Civil Engineering, Construction Management and Environmental Engineering
NO. DATE:
1 2/1/2020
2 2/13/2020
3 3/8/2020
SHEET SET: PPE
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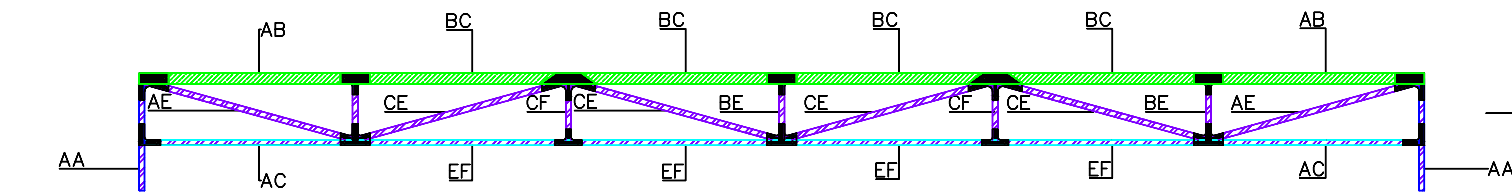
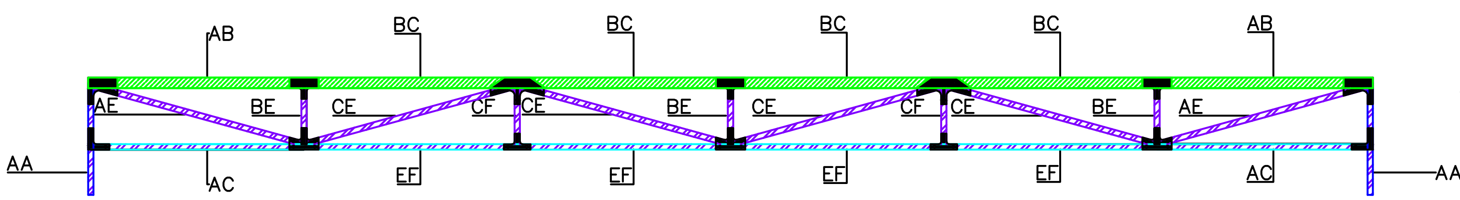
DATE: 3/10/2020
SCALE: NA
2020 STEEL BRIDGE TEAM
60% PLAN SET

Elevation View
Member Key
Scale: 1"=3'

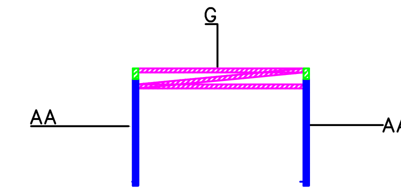


Left Side of Bridge

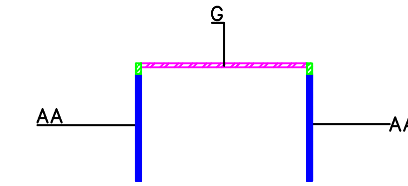


Right Side of Bridge

End View
Member Key
Scale: 1"=3'

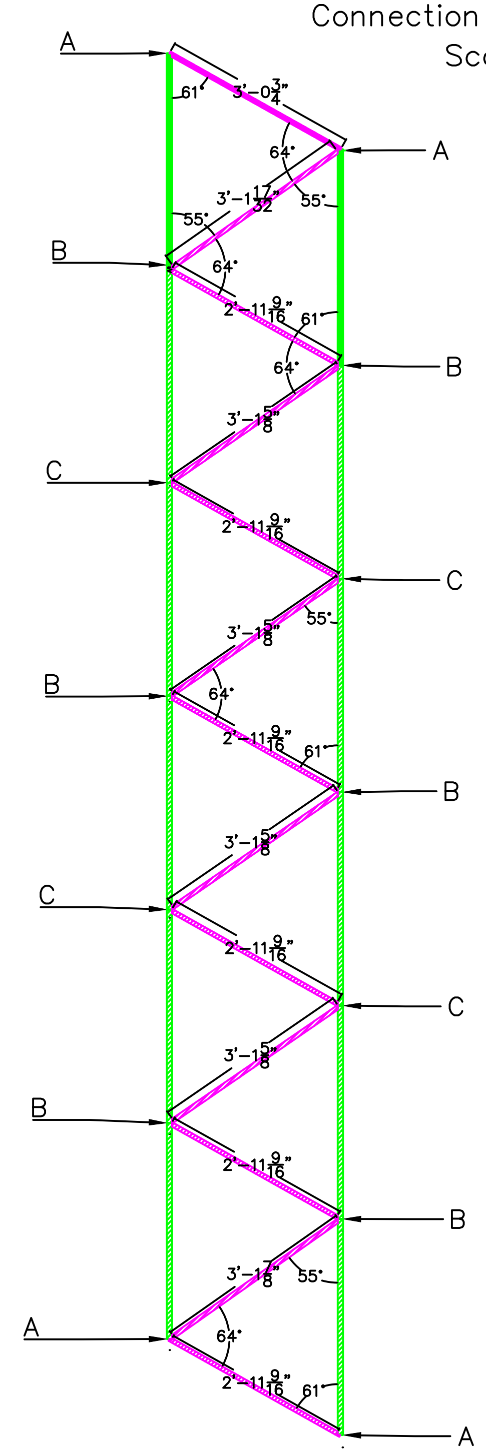


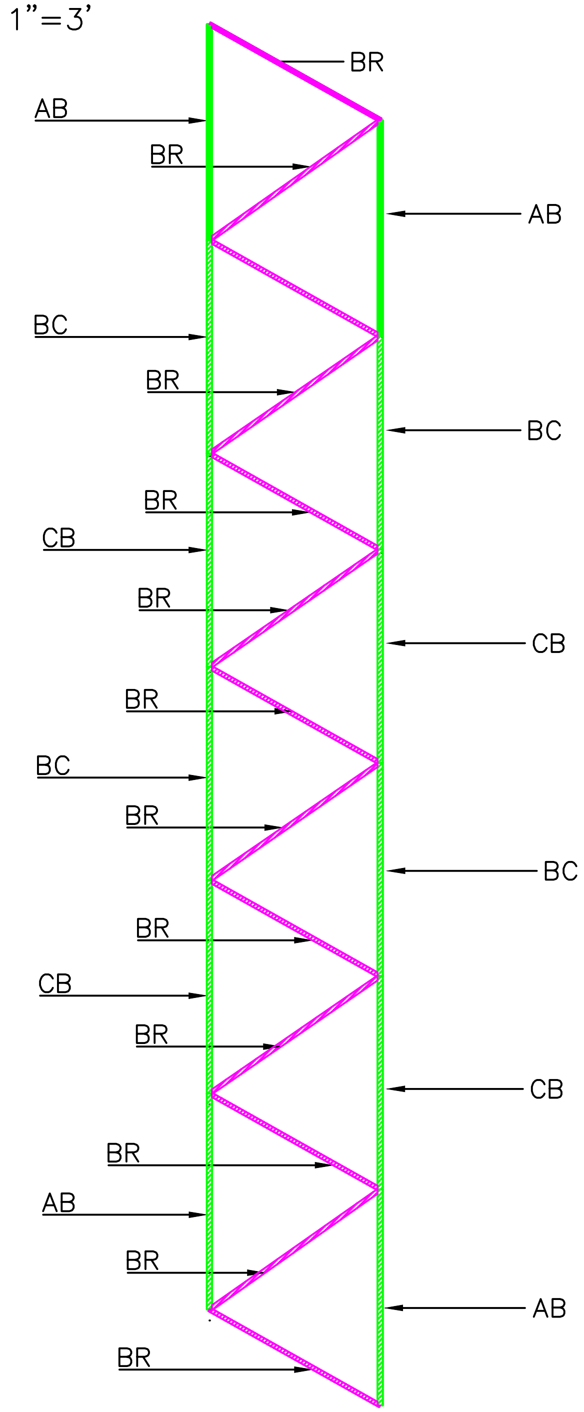
Outside View



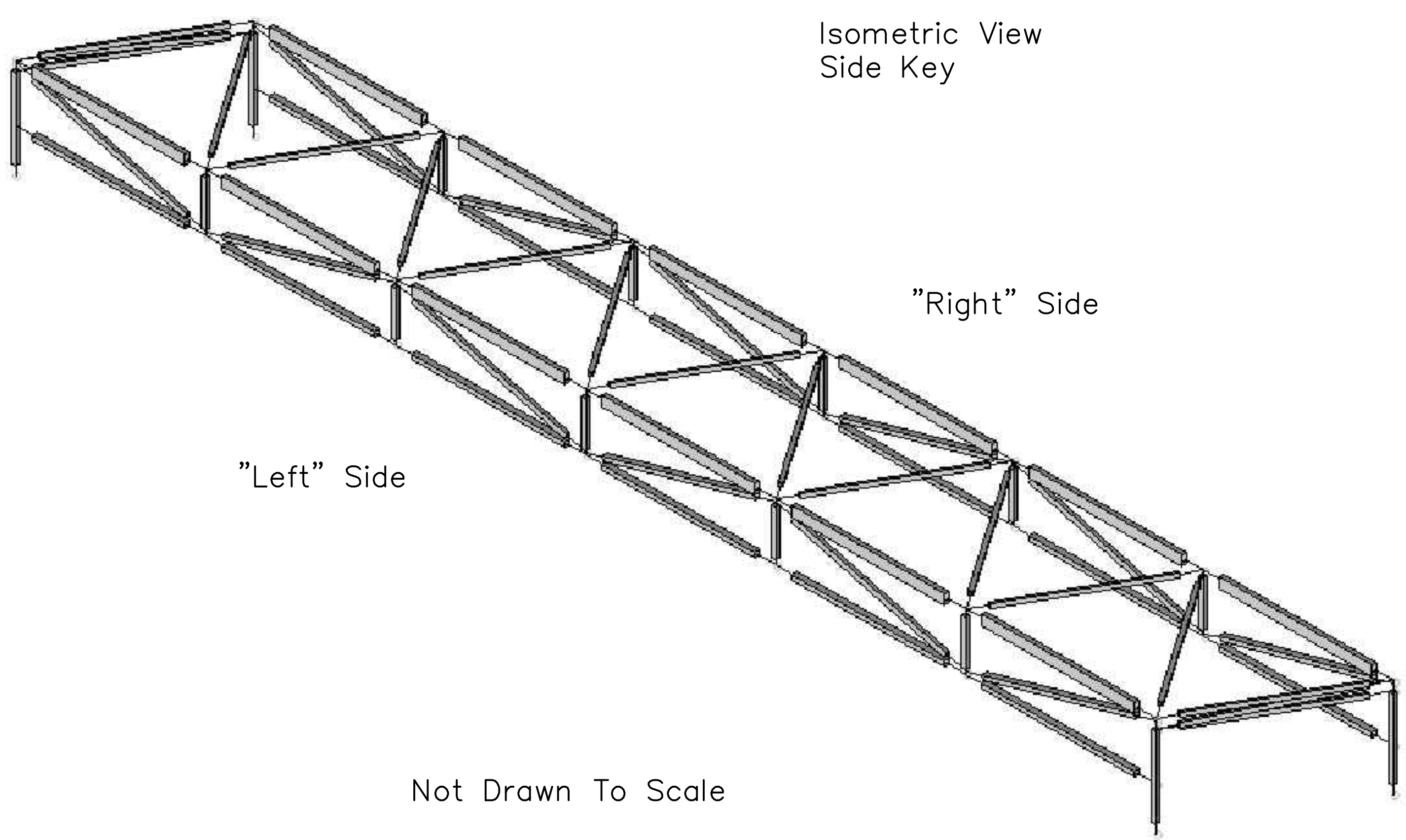
Inside View

Plan View
Connection and Member Key
Scale: 1"=3'





Isometric View
Side Key



Not Drawn To Scale

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4 OF 10

Northern Arizona University
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1	2/1/2020
2	2/13/2020
3	3/8/2020

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CHECKED BY: LAMER
DATE: 3/10/2020
SCALE: NA

2020 STEEL BRIDGE TEAM
60% PLAN SET

Member AA
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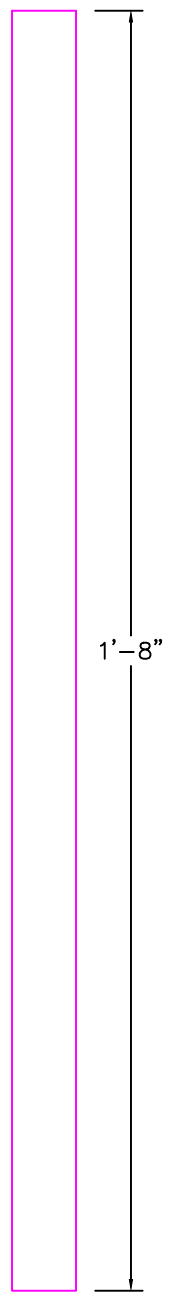
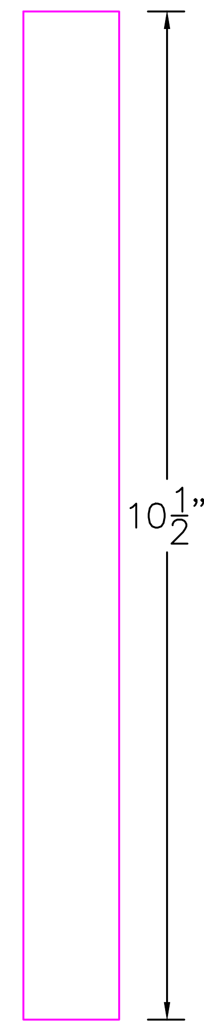
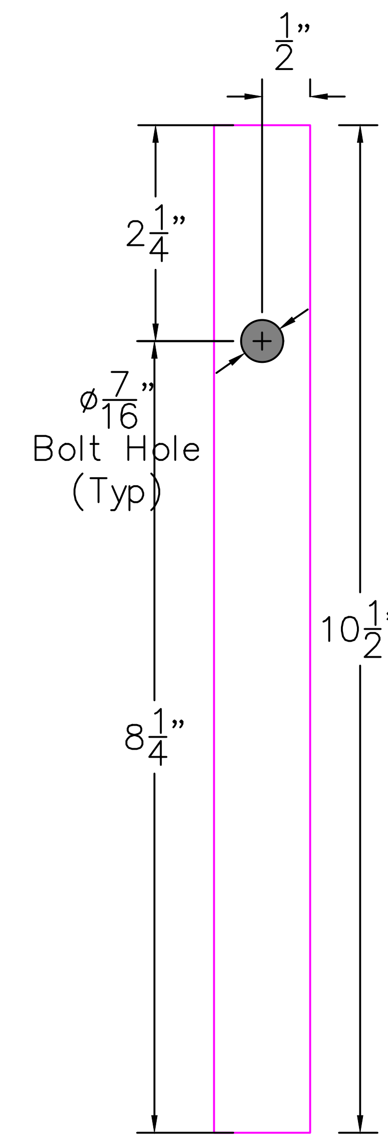
HSS Tube
Scale: 1"=3"



Member BE
Quantity: 8
1"x1"x0.065"
HSS Tube
Scale: 1"=2"





Member CF
Quantity: 6
1"x1"x0.065"
HSS Tube
Scale: 1"=2"

Note: All bolt holes are going through both walls of the tube.

~SHEET~
5 OF 10

Northern Arizona University
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REVISIONS NO.	DATE:
1	2/1/2020
2	2/13/2020
3	3/8/2020

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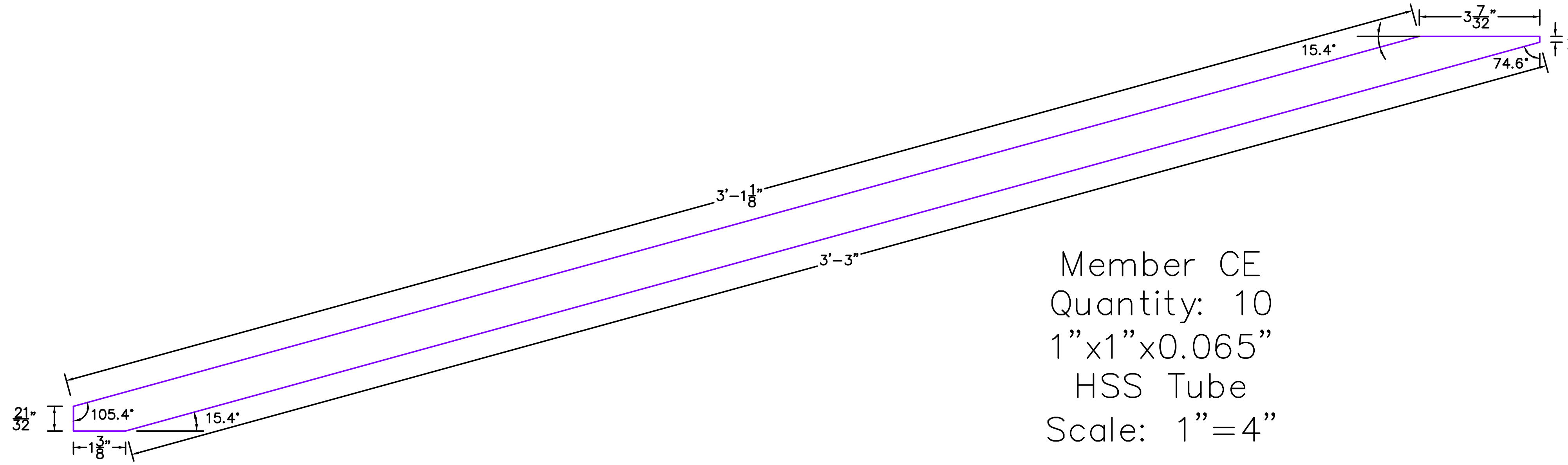
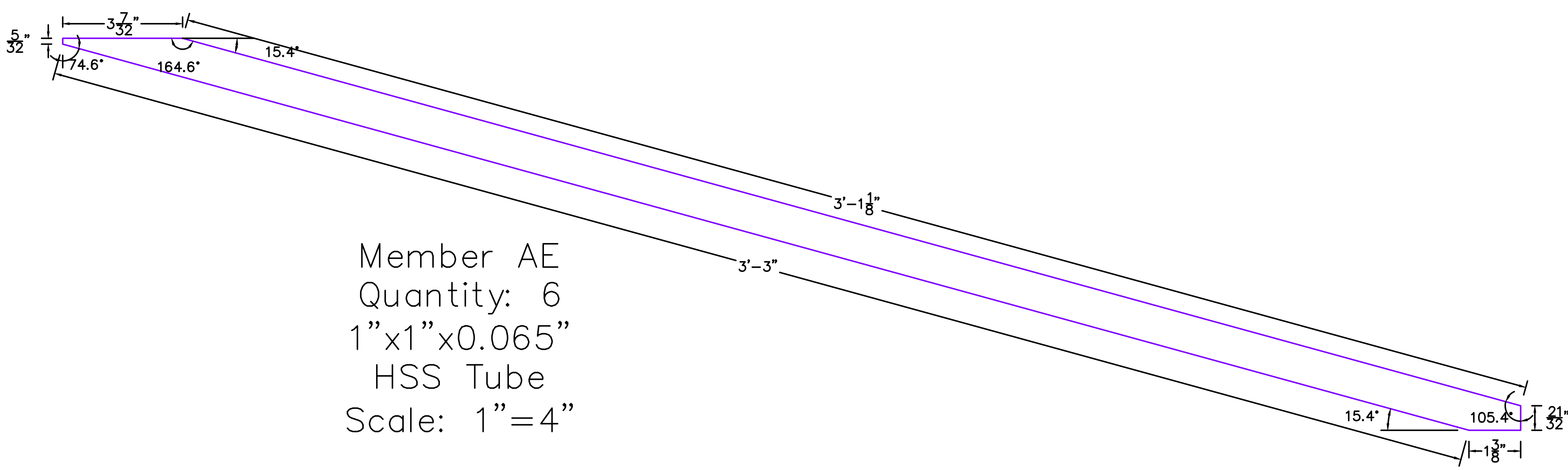
CHECKED BY: LAMER
DATE: 3/10/2020
SCALE: NA

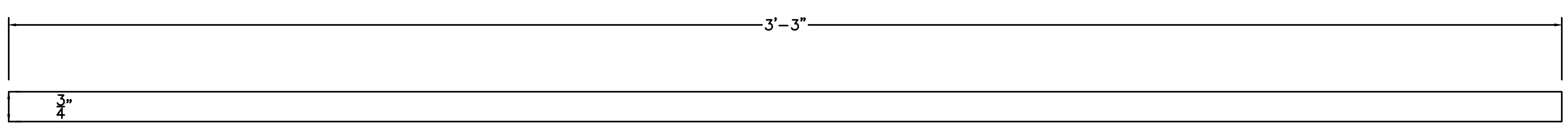
2020 STEEL BRIDGE TEAM
60% PLAN SET



Member CE
Quantity: 10
1"x1"x0.065"
HSS Tube
Scale: 1"=4"



Member AE
Quantity: 6
1"x1"x0.065"
HSS Tube
Scale: 1"=4"



Bracing
Quantity: 20
0.75"x0.75"x0.065" HSS Tube
Scale: 1"=4"

~SHEET~ 6 OF 10

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REVISIONS

NO.	DATE:
1	2/1/2020
2	2/13/2020
3	3/8/2020

SHEET SET: M2
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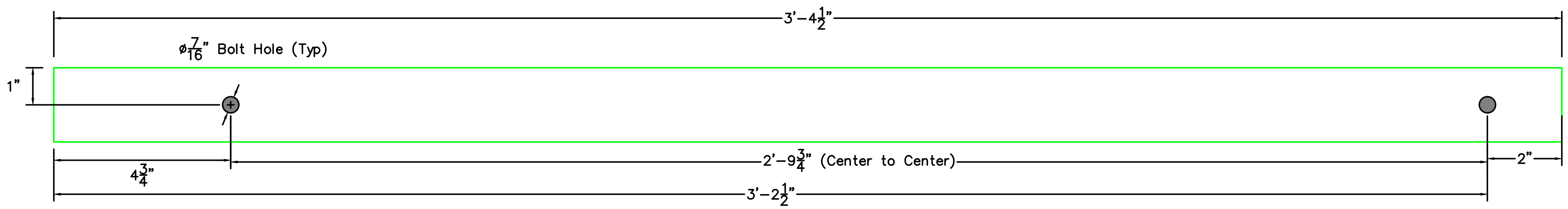
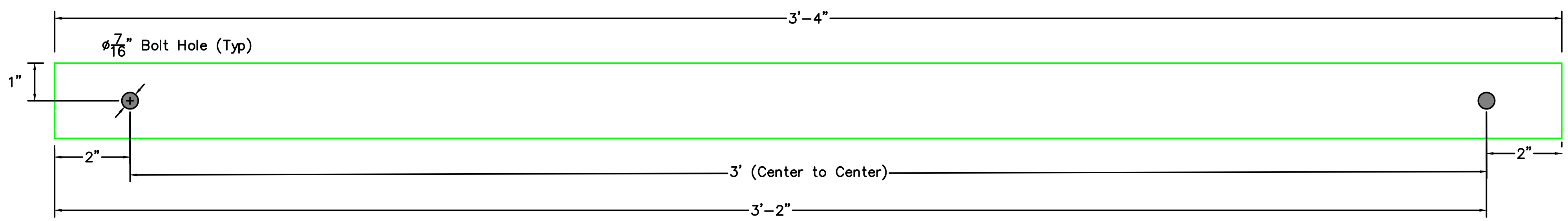
DATE: 3/10/2020
SCALE: NA

2020 STEEL BRIDGE TEAM
60% PLAN SET

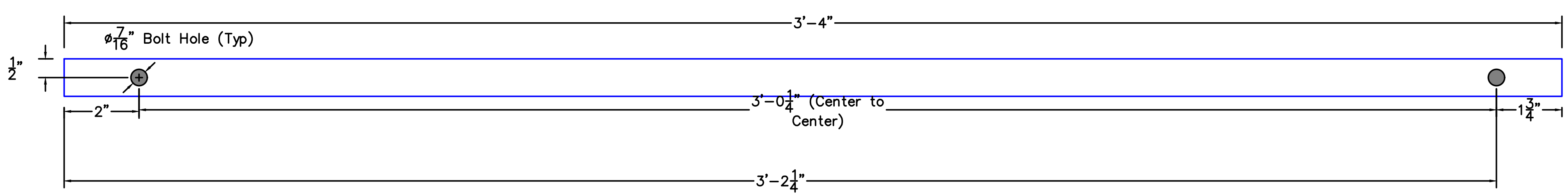
~SHEET~ 7 OF 10



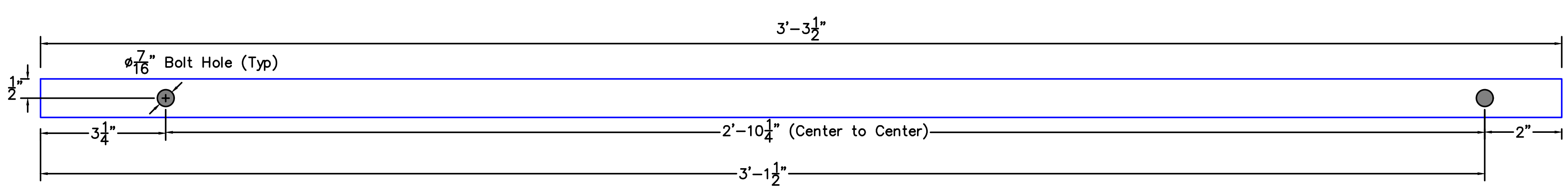
Member AB
Quantity: 6
1"x2"x0.083"
HSS Tube
Scale: 1"=4"



Member BC
Quantity: 10
1"x2"x0.083"
HSS Tube
Scale: 1"=4"



Member EF
Quantity: 10
1"x1"x0.065"
HSS Tube
Scale: 1"=4"



Member AC
Quantity: 6
1"x1"x0.065"
HSS Tube
Scale: 1"=4"

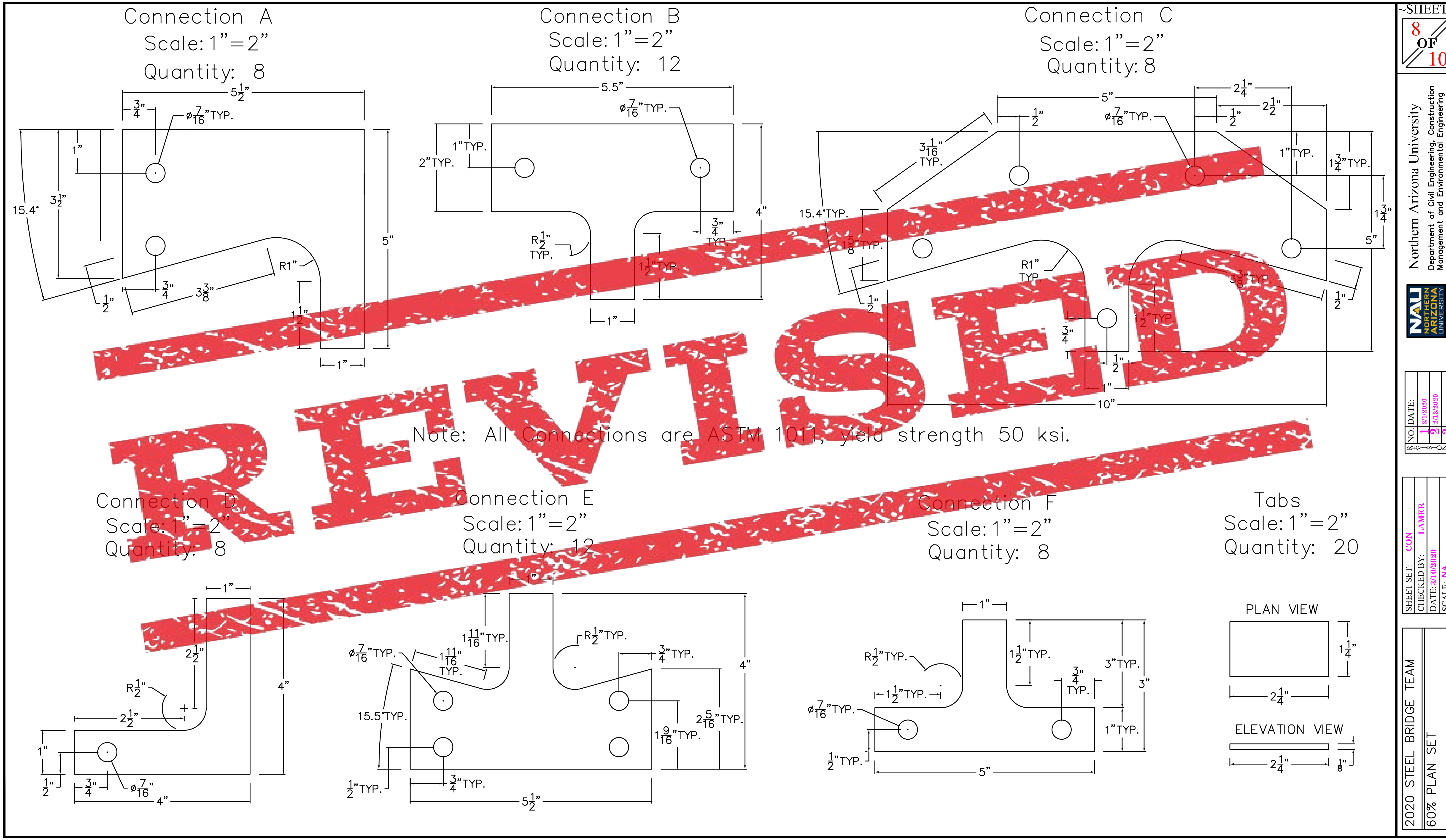
Note: All bolt holes are going through both walls of the tube.

Northern Arizona University
Department of Civil Engineering, Construction Management and Environmental Engineering

NO.	DATE:
1	2/1/2020
2	2/13/2020
3	3/8/2020

SHEET SET: M3
CHECKED BY: LAMER
DATE: 3/10/2020
SCALE: NA

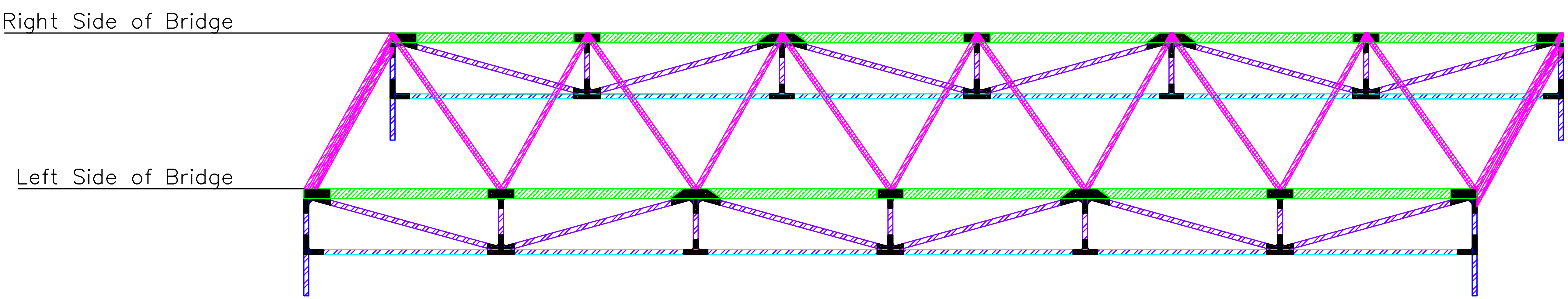
2020 STEEL BRIDGE TEAM
60% PLAN SET



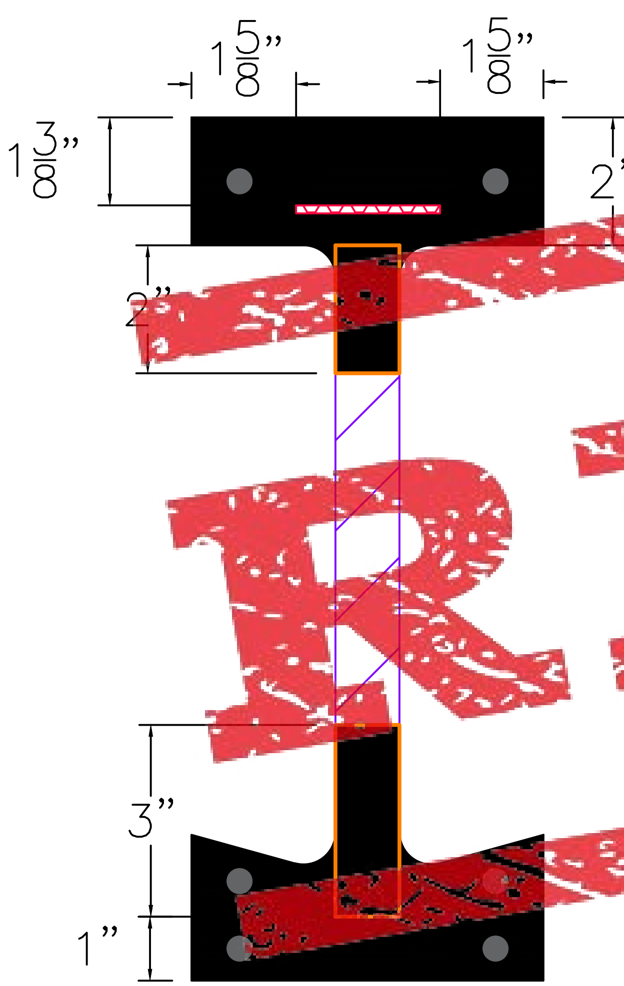
Connection A
Scale: 1"=2"
Quantity: 8
Connection B
Scale: 1"=2"
Quantity: 12
Connection C
Scale: 1"=2"
Quantity: 8
REVISED
Note: All Connections are ASTM 1011, yield strength 50 ksi.
Connection D
Scale: 1"=2"
Quantity: 8
Connection E
Scale: 1"=2"
Quantity: 12
Connection F
Scale: 1"=2"
Quantity: 8
Tabs
Scale: 1"=2"
Quantity: 20
PLAN VIEW
ELEVATION VIEW
~SHEET~
8 OF 10
Northern Arizona University
Department of Civil Engineering, Construction Management and Environmental Engineering
NAU NORTHERN ARIZONA UNIVERSITY
REVISIONS
NO. DATE:
1 2/1/2020
2 2/13/2020
3 3/8/2020
SHEET SET: CON
CHECKED BY: LAMER
DATE: 3/10/2020
SCALE: NA
2020 STEEL BRIDGE TEAM
60% PLAN SET

Isometric View

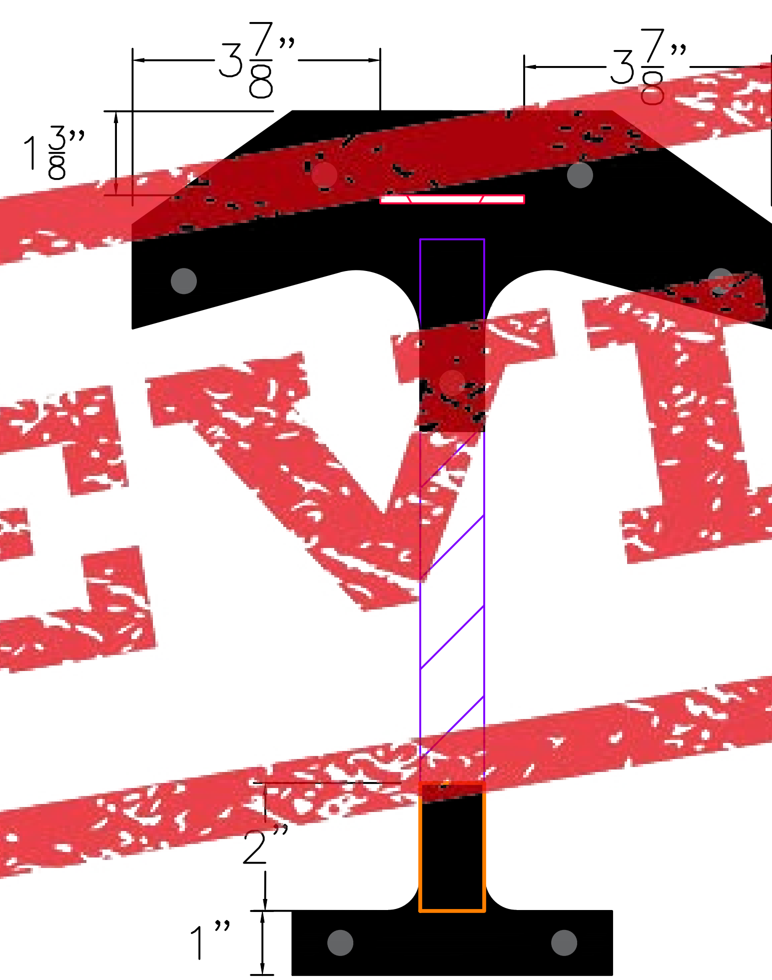
Scale: 1”=2’



Welded Piece #1
Quantity: 6
Scale: 1”=3”



Welded Piece #2
Quantity: 4
Scale: 1”=3”



– Welded Piece #1

Connection E will be welded to the bottom 3” of Member BE. Connection B will be welded to the top 2” of Member BE. Each connection will be welded to both sides of the member.

– Welded Piece #2

Connection F will be welded to the bottom 2” of Member CF. Connection C will NOT be welded to Member CF, but is just shown here for reference. Connection F will be welded to both sides of the member, just as before.

– Welded Piece #3

Connection D will be welded 5” from the bottom of Member AA. Connection A will be welded to the top 3” of Member AA. This will be done on both sides of the member, as before.

REVISED

~SHEET~ 9 OF 10

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NO.	DATE:
1	2/1/2020
2	2/13/2020
3	3/8/2020

SHEET SET: W1
CHECKED BY: LAMER
DATE: 3/10/2020
SCALE: NA

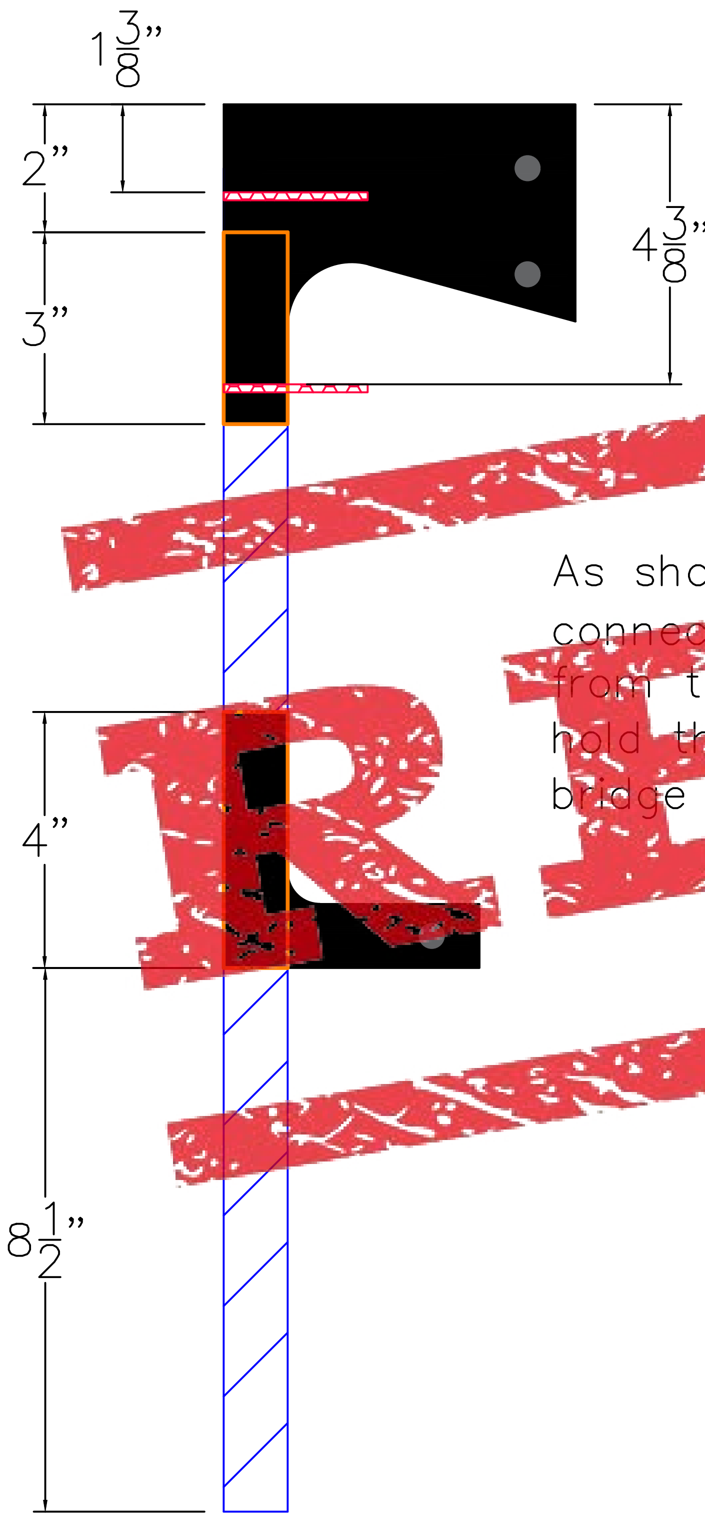
2020 STEEL BRIDGE TEAM
60% PLAN SET

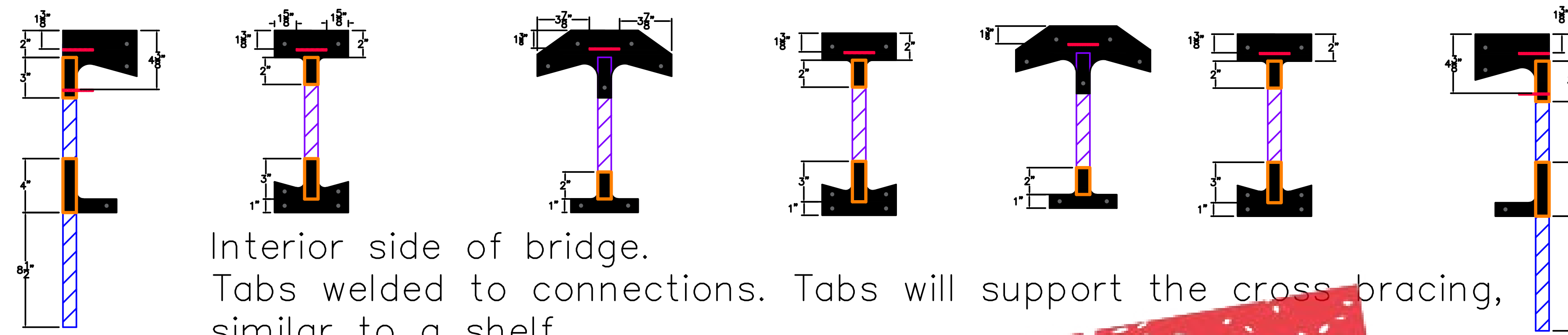
~SHEET~ 10 OF 10

Welded Piece #3

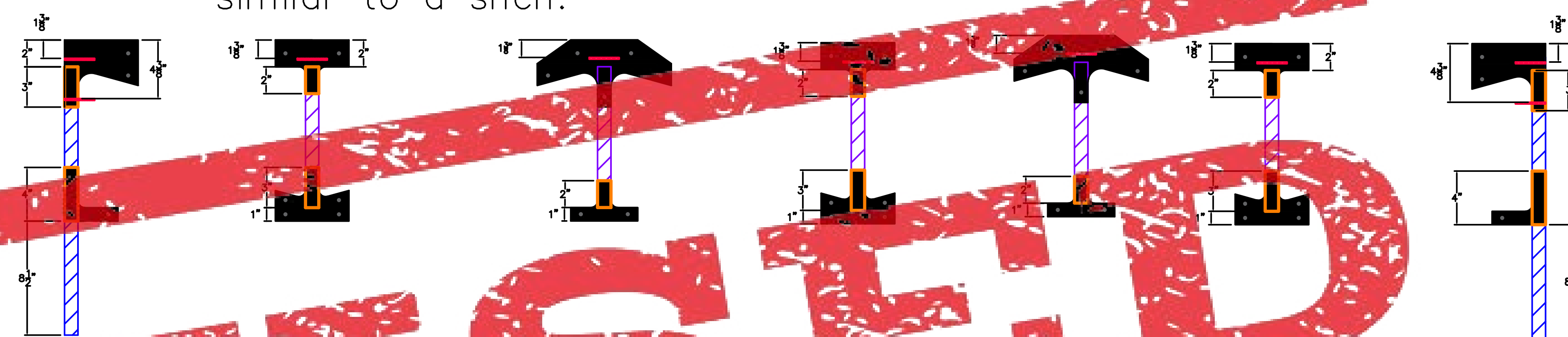
Quantity: 4

Scale: 1"=3"



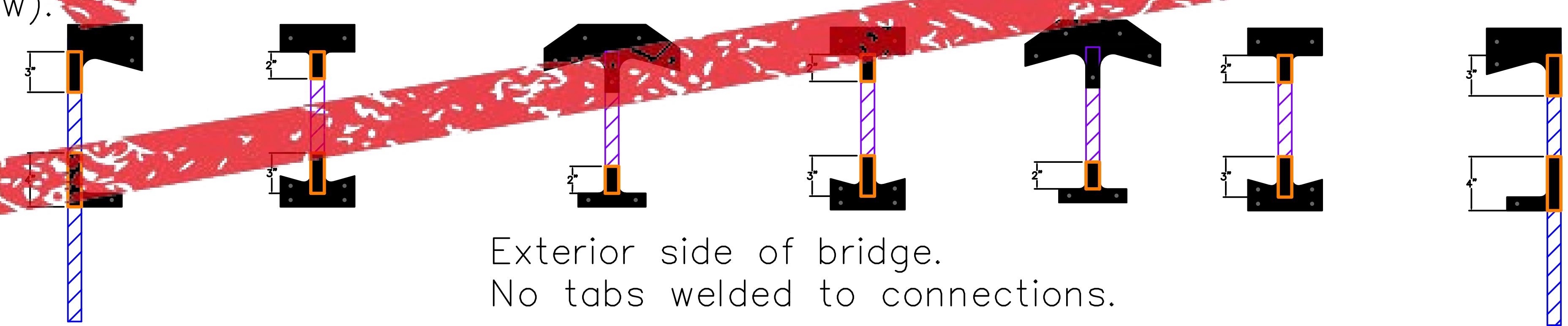


Interior side of bridge.
Tabs welded to connections. Tabs will support the cross bracing, similar to a shelf.

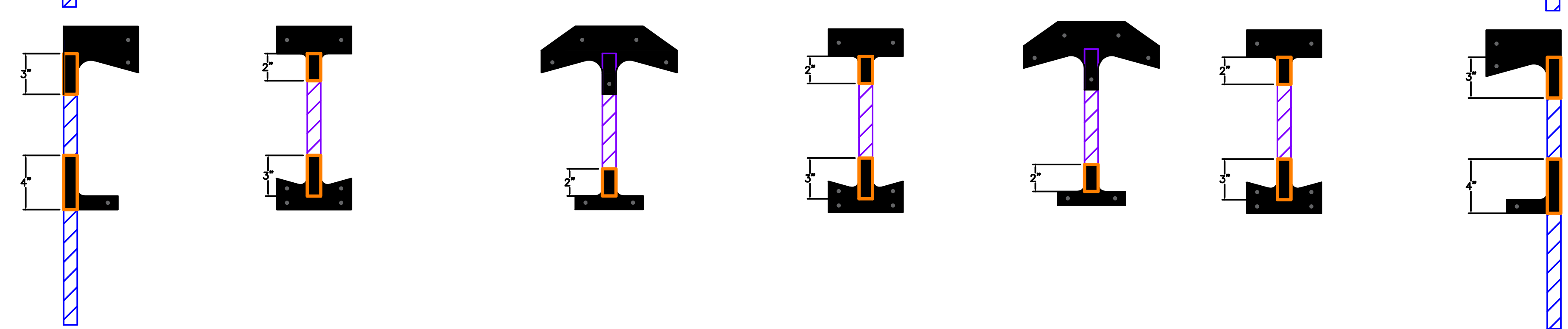


REVISED

As shown above, the tabs (denoted by the red rectangle) will be welded 1 3/8" from the top of the connections placed along the top chord. Only on Connection A will the tabs also be welded 4 3/8" from the top. This will be done on all connections facing the interior of the bridge (as the tabs hold the lateral bracing members) and will NOT be done on any tabs facing the exterior of the bridge (shown below).



Exterior side of bridge.
No tabs welded to connections.



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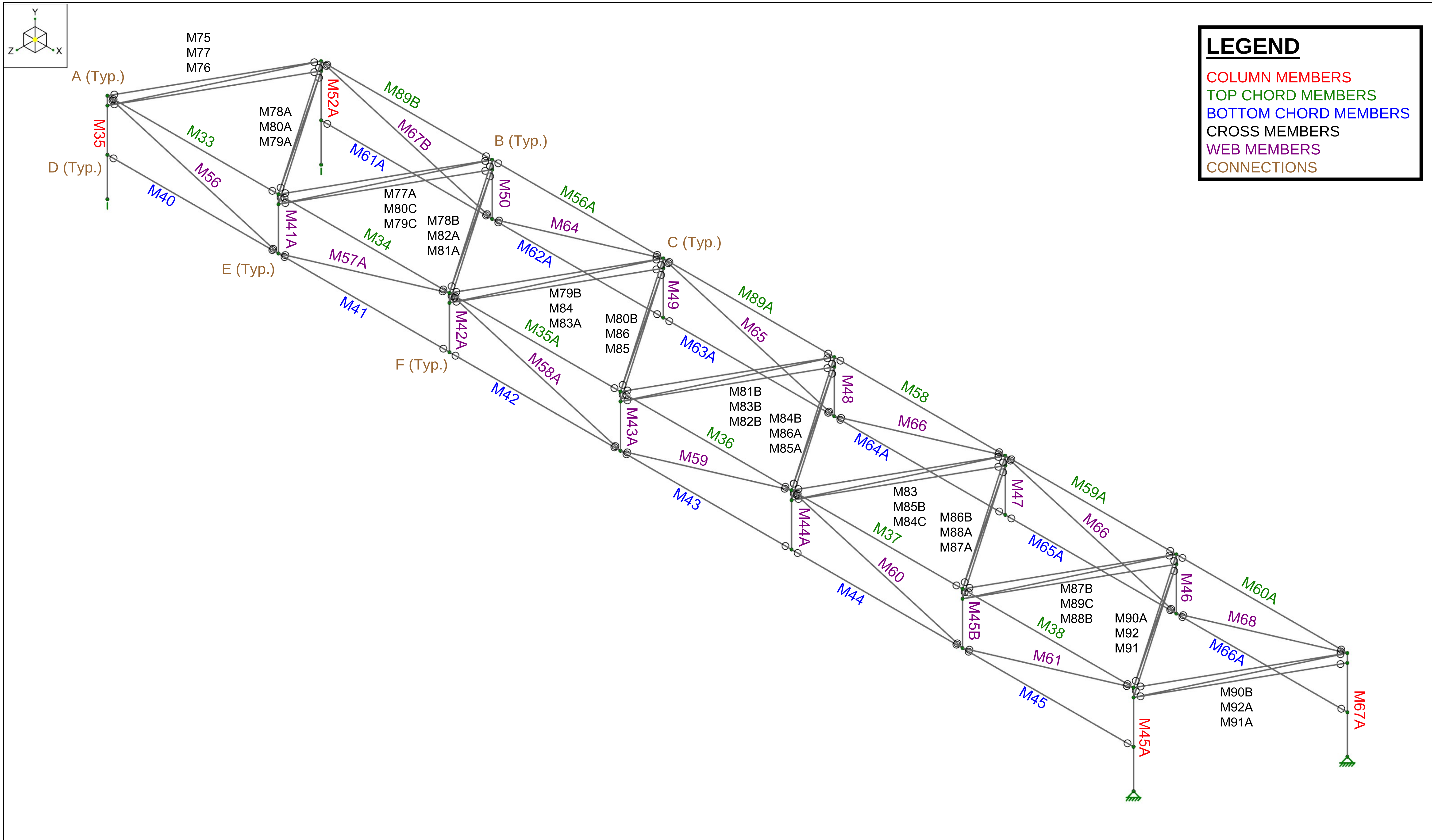
NO.	DATE:
1	2/1/2020
2	2/13/2020
3	3/8/2020

SHEET SET: W2
CHECKED BY: LAMER
DATE: 3/10/2020
SCALE: NA

2020 STEEL BRIDGE TEAM
60% PLAN SET



Horizontal cross member naming listed top down (typical).	Bridge Legend - RISA Labels	Drawn By: Eric Barton
		Mar 04, 2021
		Existing bridge loaded to failure.r3d

4 3 2 1

B

Revisions and Add-Ons[New Connection Designs]

Contents:

- Bridge Assembly Drawing
- Connection A Drawing
- Connection B Drawing
- Connection 0.5B Drawing
- Connection C Drawing
- Connection D Drawing

B

A

A

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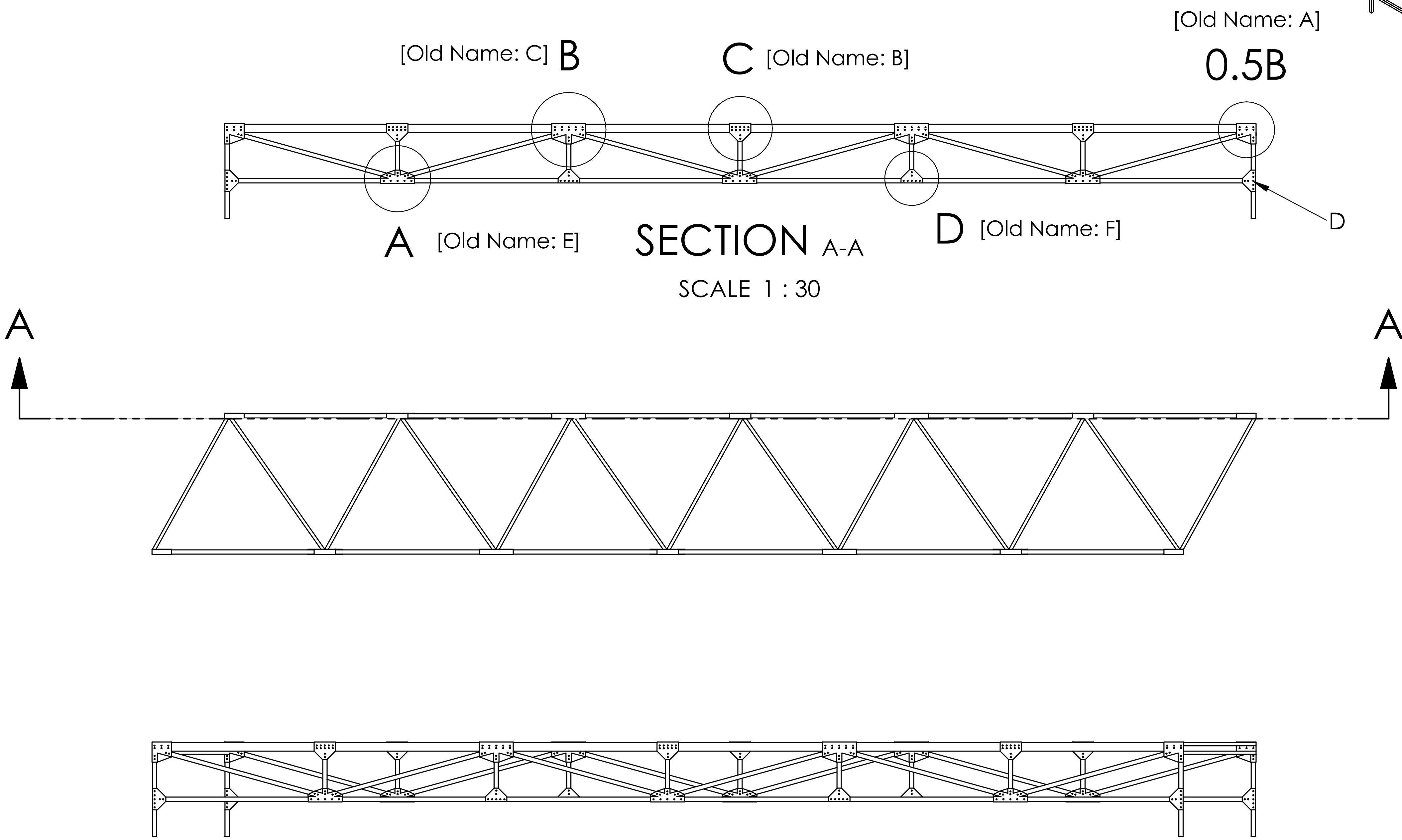
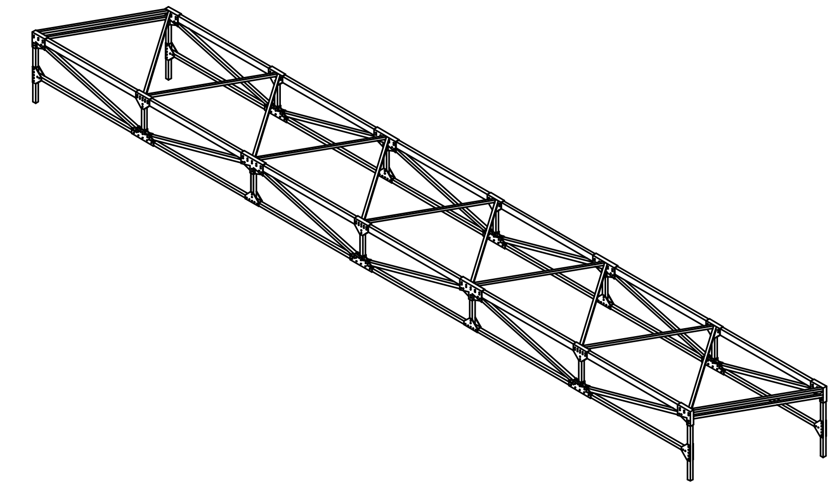
		UNLESS OTHERWISE SPECIFIED:		NAME	DATE	
		DIMENSIONS ARE IN INCHES TOLERANCES: FRACTIONAL± ANGULAR: MACH± BEND ± TWO PLACE DECIMAL ± THREE PLACE DECIMAL ±	DRAWN			TITLE:
			CHECKED			Additional Plans
			ENG APPR.			
			MFG APPR.			
		INTERPRET GEOMETRIC TOLERANCING PER:	Q.A.			
		MATERIAL	COMMENTS:			SIZE B DWG. NO. New Connections REV
NEXT ASSY	USED ON	FINISH				
APPLICATION		DO NOT SCALE DRAWING				SCALE: WEIGHT: SHEET 1 OF 1

4 3 2 1

New Naming Scheme for Connection Locations





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DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±

INTERPRET GEOMETRIC TOLERANCING PER:

MATERIAL

FINISH

DO NOT SCALE DRAWING

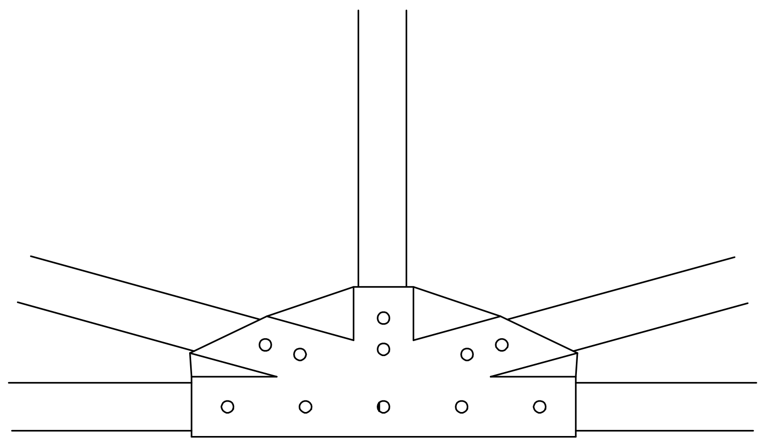
	NAME	DATE
DRAWN	E. KEISER	
CHECKED		
ENG APPR.		
MFG APPR.		
Q.A.		

COMMENTS: Shows new connection detail locations

TITLE: Overall Bridge Drawing

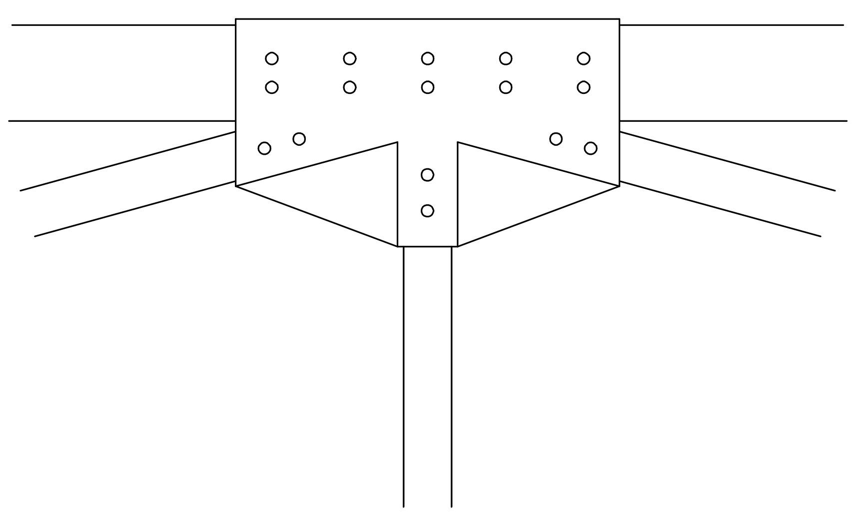
SIZE	DWG. NO.	REV
B	Bridge Assembly	1

SCALE: 1:30 WEIGHT: SHEET 1 OF 2



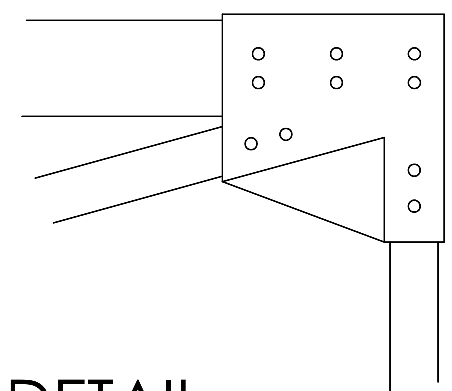
DETAIL A

SCALE 1 : 4



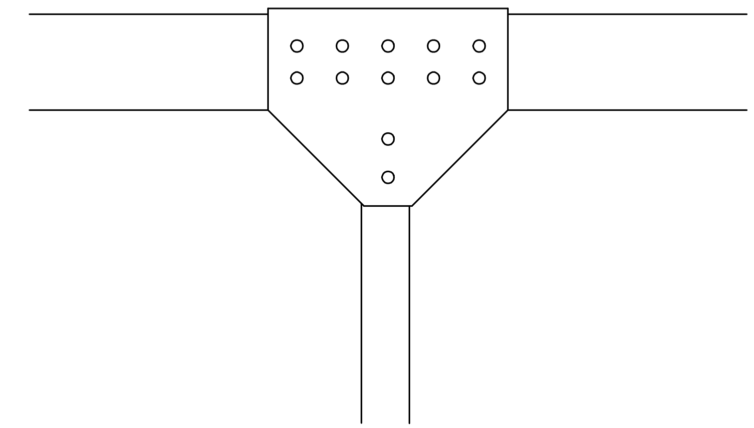
DETAIL B

SCALE 1 : 4



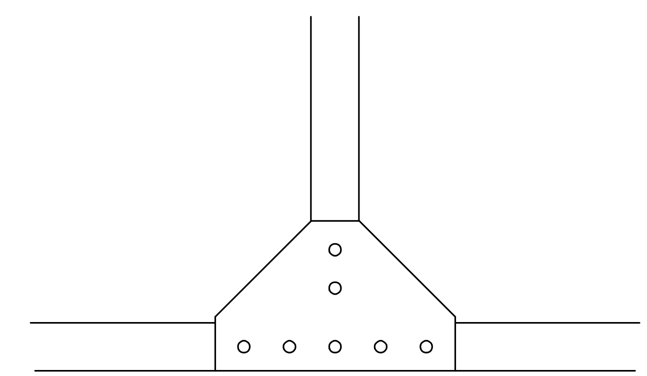
DETAIL 0.5B

SCALE 1 : 4



DETAIL C

SCALE 1 : 4



DETAIL D

SCALE 1 : 4

NEXT ASSY	USED ON
APPLICATION	

UNLESS OTHERWISE SPECIFIED:

DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±

INTERPRET GEOMETRIC TOLERANCING PER:

MATERIAL

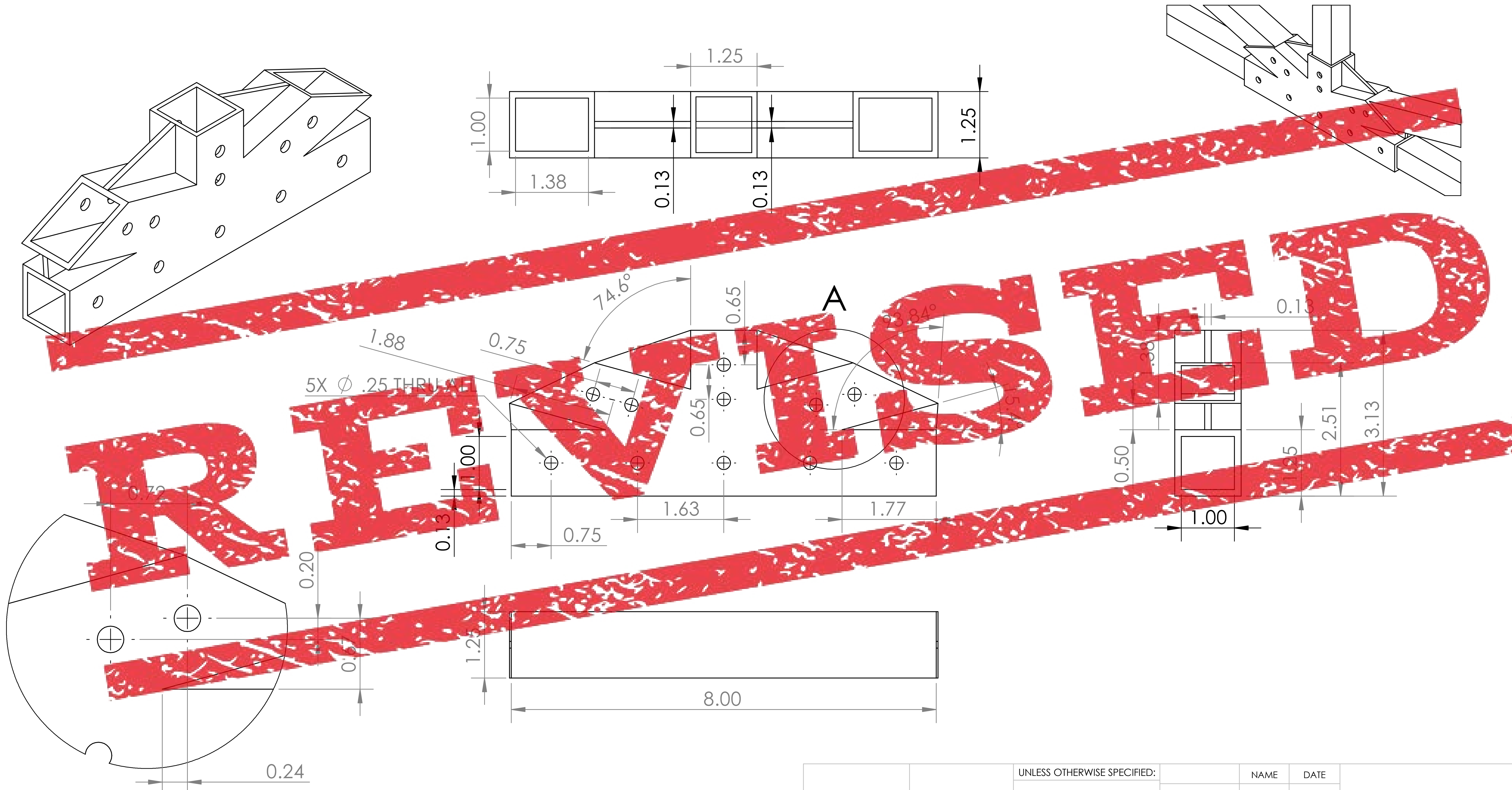
FINISH

DO NOT SCALE DRAWING

	NAME	DATE
DRAWN	E. KEISER	
CHECKED		
ENG APPR.		
MFG APPR.		
Q.A.		
COMMENTS:		

TITLE: Details for bridge assembly connections

SIZE	DWG. NO.	REV
B	Bridge Assembly	
SCALE: 1:4	WEIGHT:	SHEET 2 OF 2



DETAIL A

SCALE 1 : 1

NEXT ASSY	USED ON
APPLICATION	

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DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±

INTERPRET GEOMETRIC TOLERANCING PER:

MATERIAL

FINISH

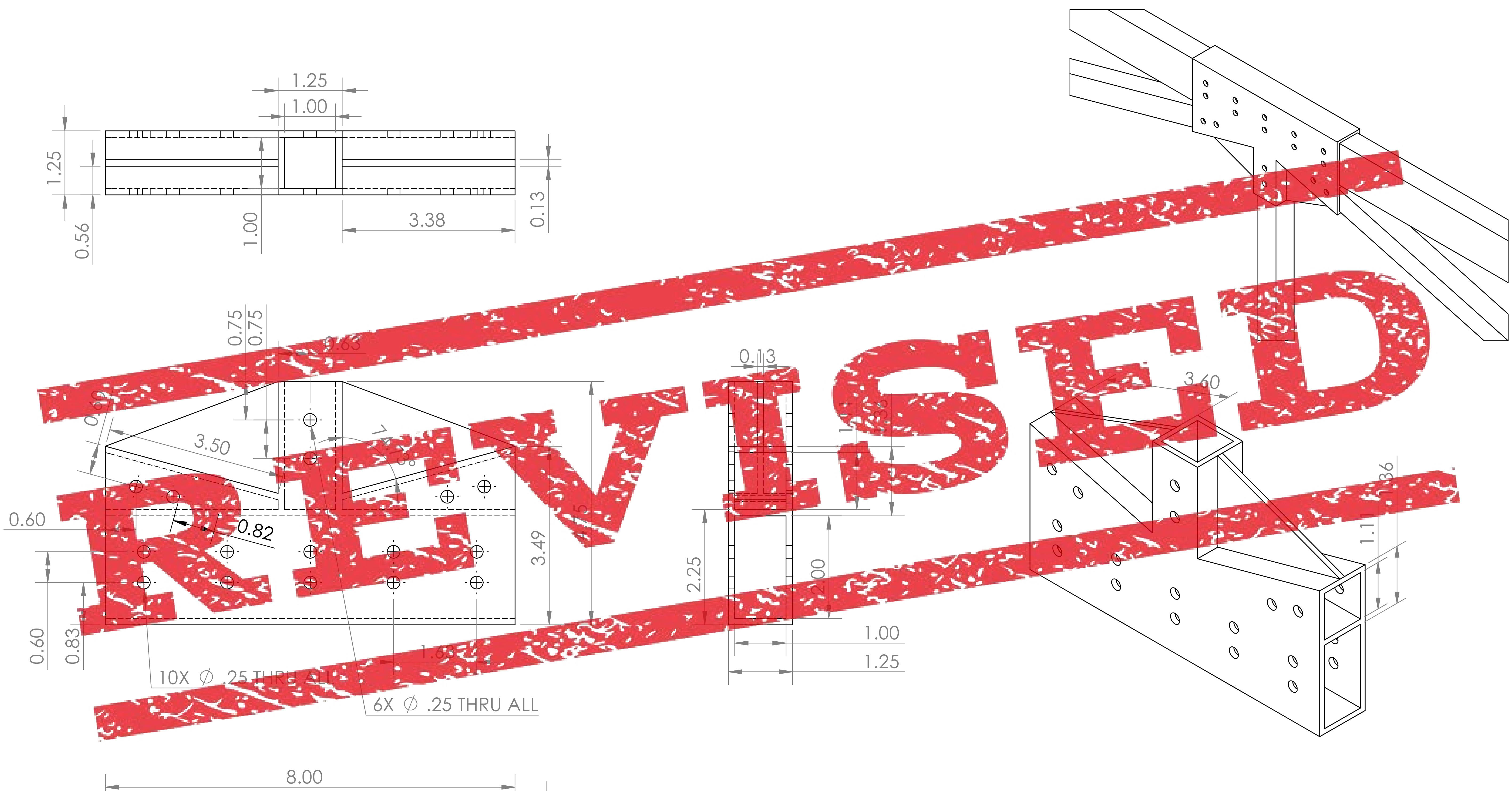
DO NOT SCALE DRAWING

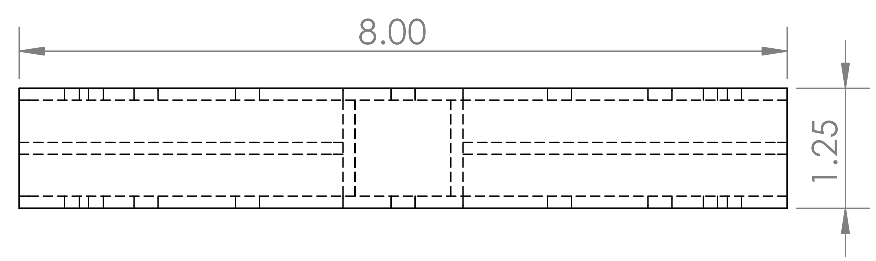
	NAME	DATE
DRAWN		
CHECKED		
ENG APPR.		
MFG APPR.		
Q.A.		
COMMENTS:		

TITLE:

SIZE	DWG. NO.	REV
B	Detail A	

1:2	WEIGHT:	SHEET 1 OF 1





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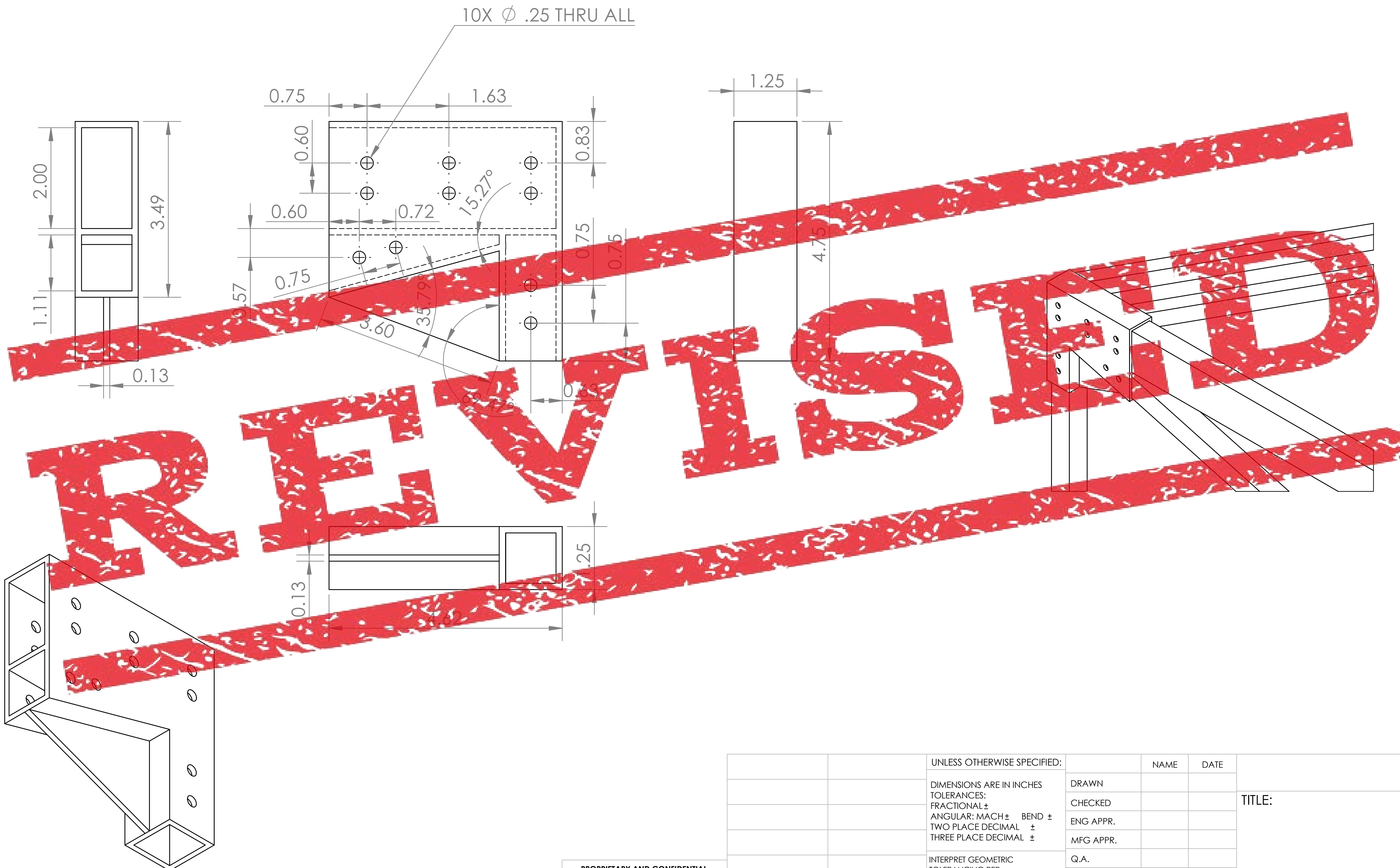
		DIMENSIONS ARE IN INCHES TOLERANCES: FRACTIONAL± ANGULAR: MACH± BEND ± TWO PLACE DECIMAL ± THREE PLACE DECIMAL ±	DRAWN			TITLE:
			CHECKED			
			ENG APPR.			
			MFG APPR.			
		INTERPRET GEOMETRIC TOLERANCING PER:	Q.A.			
		MATERIAL	COMMENTS:			SIZE B — DWG. NO. Detail B — REV
NEXT ASSY	USED ON	FINISH				
APPLICATION		DO NOT SCALE DRAWING				SCALE: 1:2 — WEIGHT: — SHEET 1 OF 1



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APPLICATION	

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DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±

INTERPRET GEOMETRIC TOLERANCING PER:

MATERIAL

FINISH

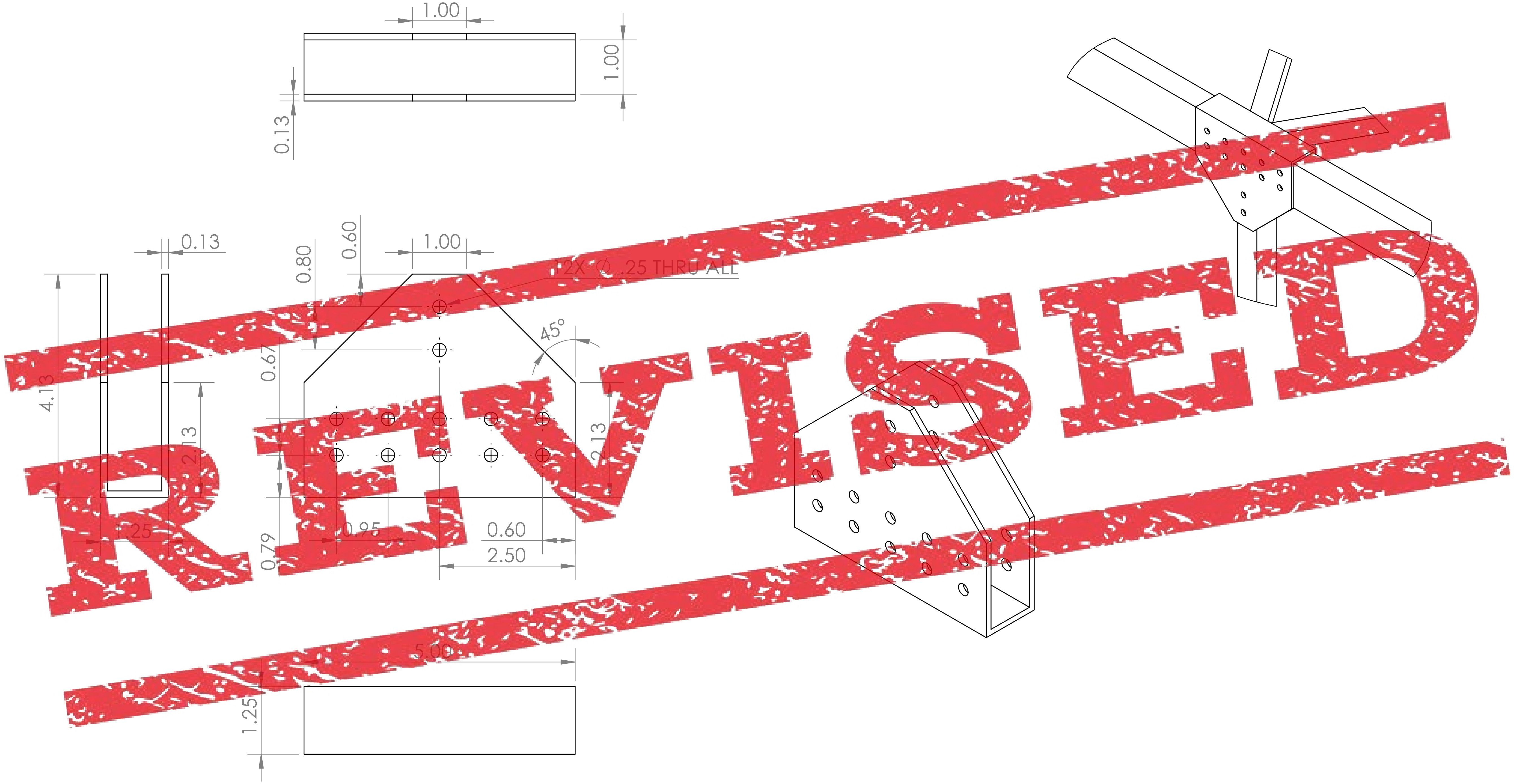
DO NOT SCALE DRAWING

	NAME	DATE
DRAWN		
CHECKED		
ENG APPR.		
MFG APPR.		
Q.A.		

COMMENTS:
END OF BRIDGE CONNECTION.
EFFECTIVELYHALF OF CONNECTION B

TITLE:

SIZE	DWG. NO.	REV
B	Detail 0.5B	
SCALE: 1:2	WEIGHT:	SHEET 1 OF 1



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NEXT ASSY	USED ON
APPLICATION	

UNLESS OTHERWISE SPECIFIED:

DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±

INTERPRET GEOMETRIC TOLERANCING PER:

MATERIAL

FINISH

DO NOT SCALE DRAWING

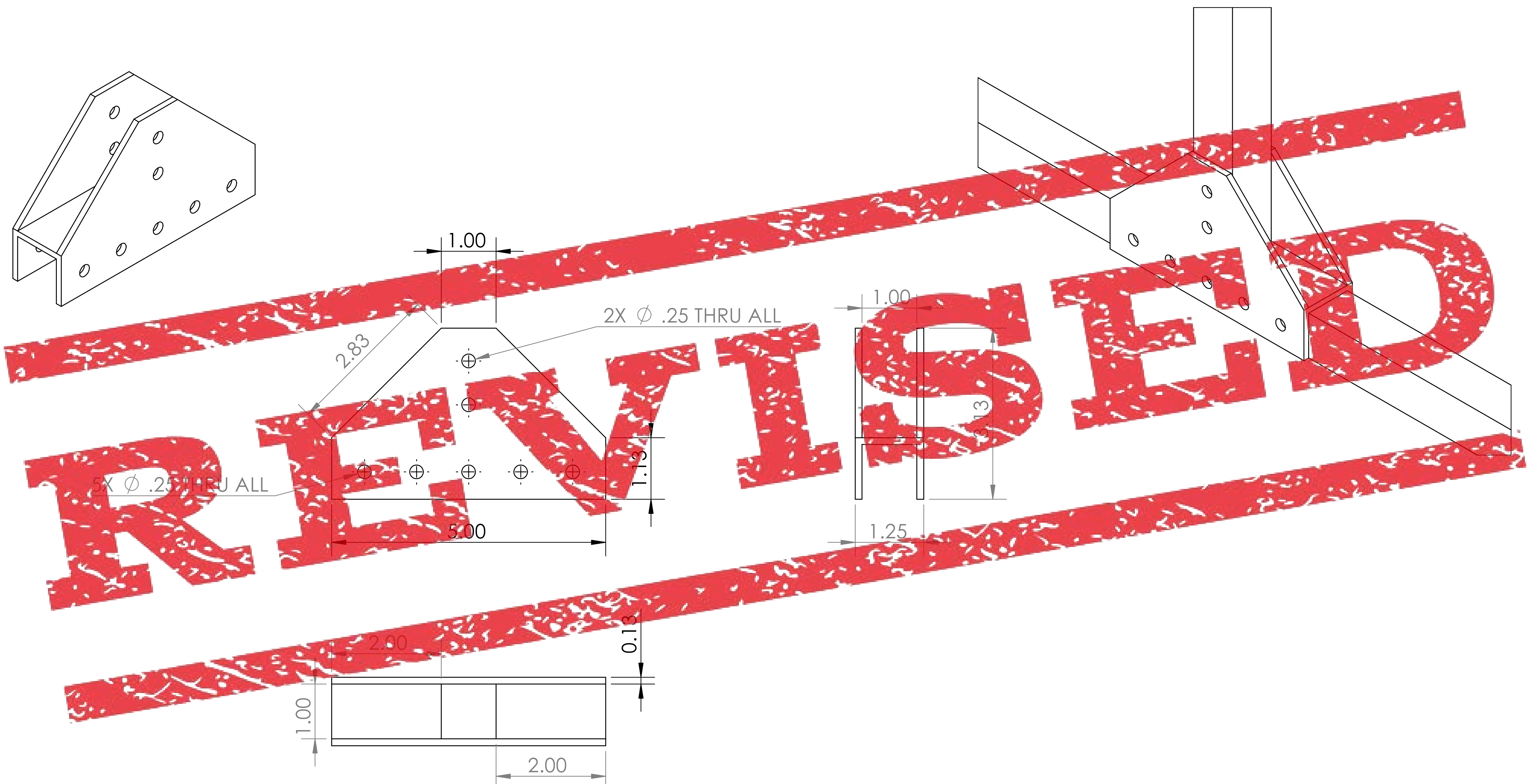
	NAME	DATE
DRAWN		
CHECKED		
ENG APPR.		
MFG APPR.		
Q.A.		

COMMENTS: FOR 3-MEMBER NODE AT TOP CHORD

TITLE:

SIZE **B** DWG. NO. Detail C REV

SCALE: 1:2 WEIGHT: SHEET 1 OF 1



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TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±

INTERPRET GEOMETRIC TOLERANCING PER:

MATERIAL

FINISH

DO NOT SCALE DRAWING

	NAME	DATE
DRAWN		
CHECKED		
ENG APPR.		
MFG APPR.		
Q.A.		

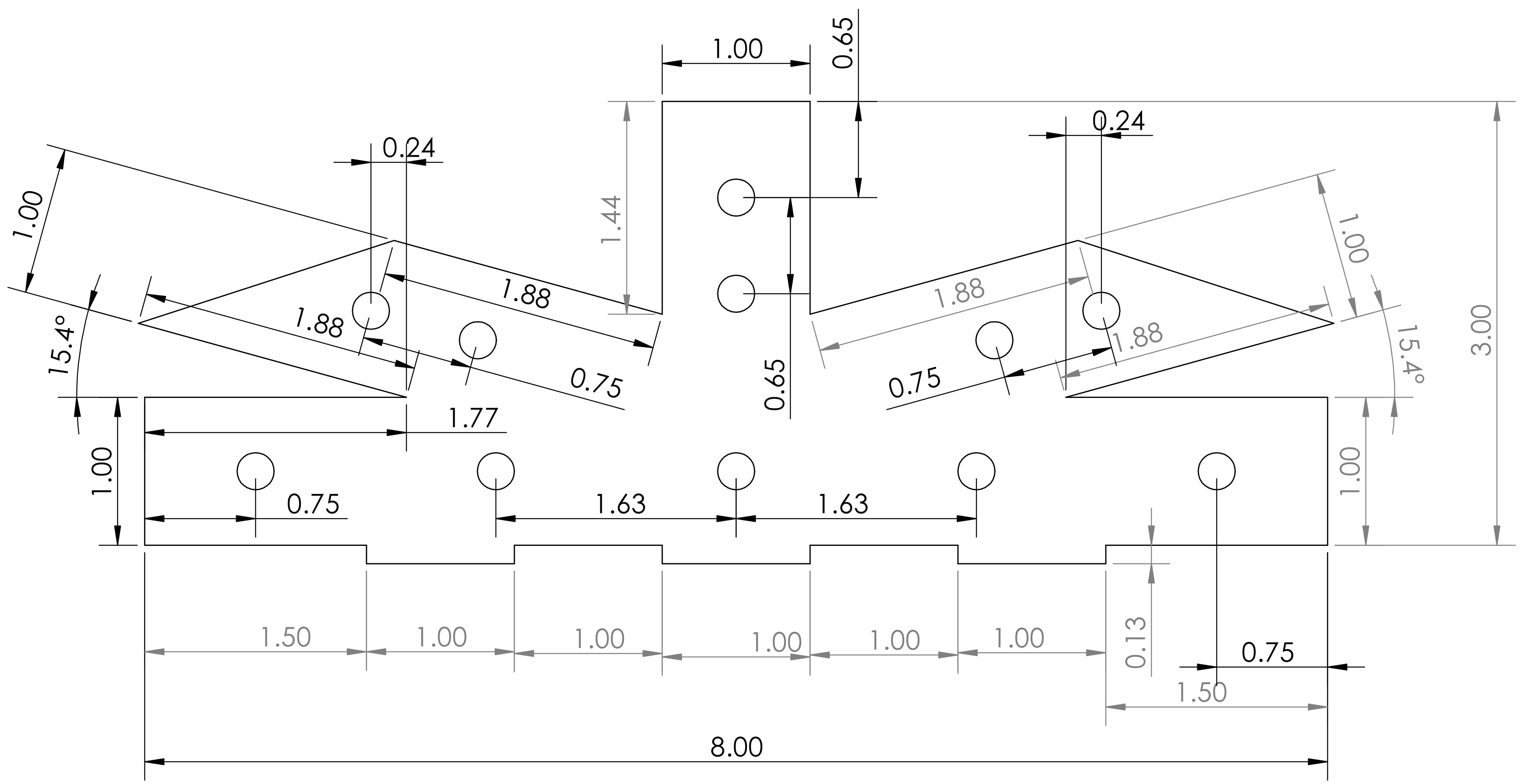
COMMENTS:

TITLE:

SIZE **B** | DWG. NO. Detail D | REV

SCALE: 1:2 | WEIGHT: | SHEET 1 OF 1

QTY REQUIRED: 12
MATERIAL: GRADE 50 ASTM 1011
THICKNESS: 11 GA



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NEXT ASSY	USED ON
A-2	Conn. A

APPLICATION

UNLESS OTHERWISE SPECIFIED:
DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±

INTERPRET GEOMETRIC TOLERANCING PER:

MATERIAL
ASTM 1011 - Grade 50 - 11 ga.

FINISH
Unfinished

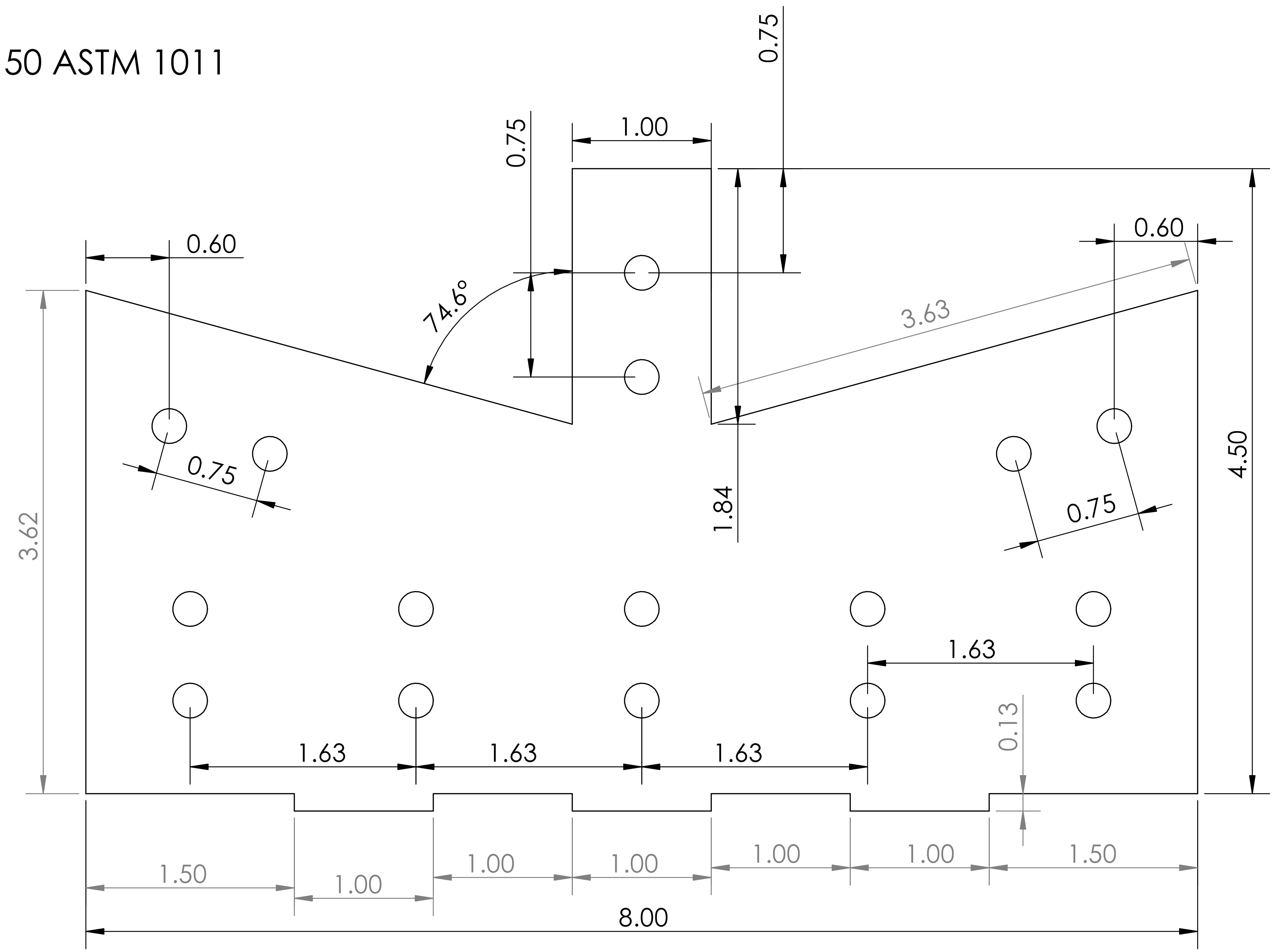
DO NOT SCALE DRAWING

	NAME	DATE
DRAWN	ELK	3.29.21
CHECKED	MEB	3.30.21
ENG APPR.		
MFG APPR.		
Q.A.		
COMMENTS:		

TITLE:
Side plates for connection A

SIZE: B
DWG. NO.: A-1
REV: 1
SCALE: 1:1
WEIGHT:
SHEET 1 OF 1

QTY REQUIRED: 8
MATERIAL: GRADE 50 ASTM 1011
THICKNESS: 11 GA



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A-2	CONN. B
NEXT ASSY	USED ON
APPLICATION	

UNLESS OTHERWISE SPECIFIED:
DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±
INTERPRET GEOMETRIC TOLERANCING PER:
MATERIAL
GRADE 50 ASTM 1011
FINISH
UNFINISHED
DO NOT SCALE DRAWING

	NAME	DATE
DRAWN	ELK	3.29.21
CHECKED	MEB	3.30.21
ENG APPR.		
MFG APPR.		
Q.A.		
COMMENTS:		

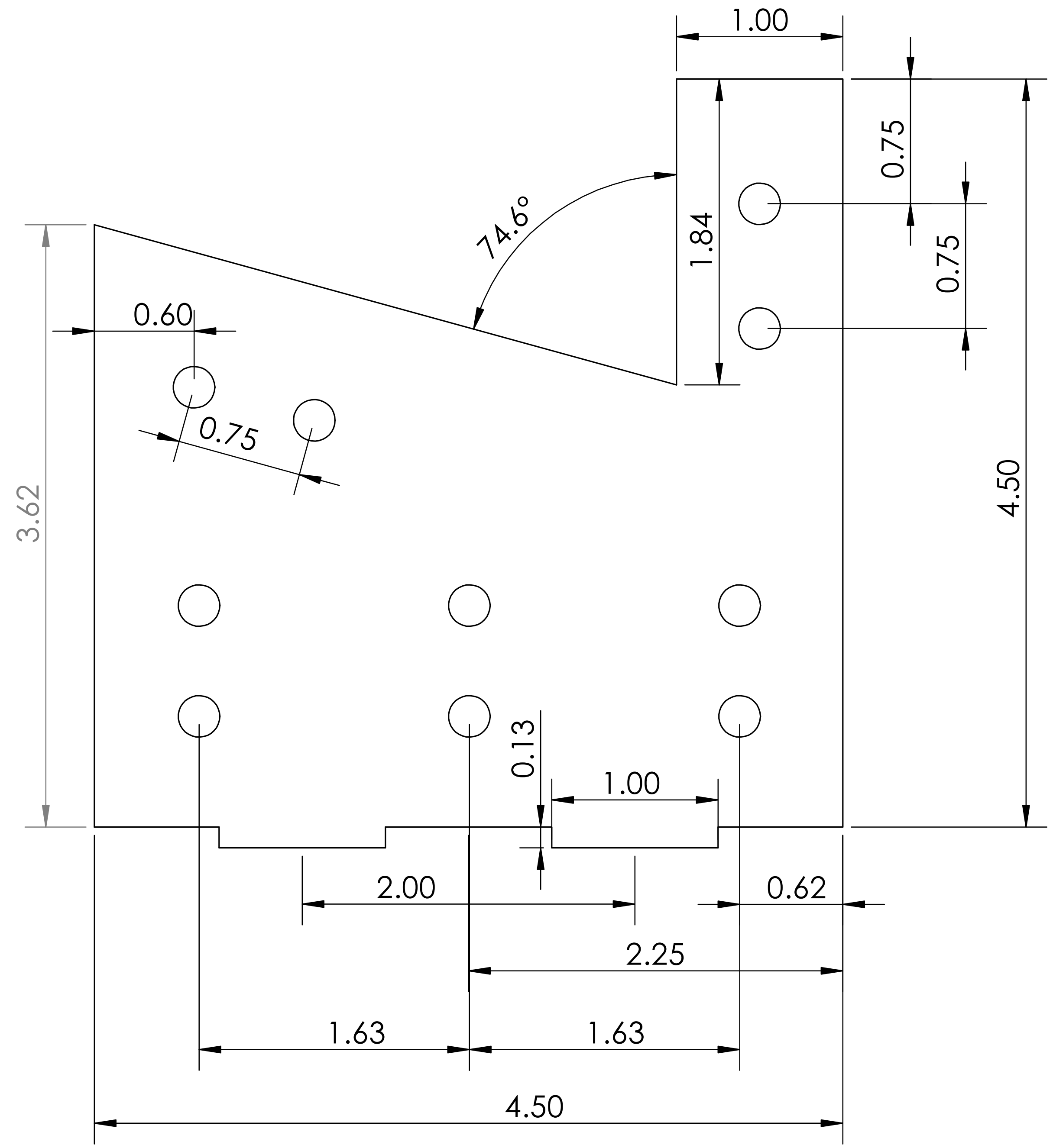
TITLE:
SIDE PLATE FOR CONN. B

SIZE **B** DWG. NO. B-1 REV 1

SCALE: 1:1 WEIGHT: SHEET 1 OF 1

SOLIDWORKS Educational Product. For Instructional Use Only.

QTY REQUIRED: 8
MATERIAL: GRADE 50 ASTM 1011
THICKNESS: 11 GA



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C-2	CONN. 0.5B
NEXT ASSY	USED ON
APPLICATION	

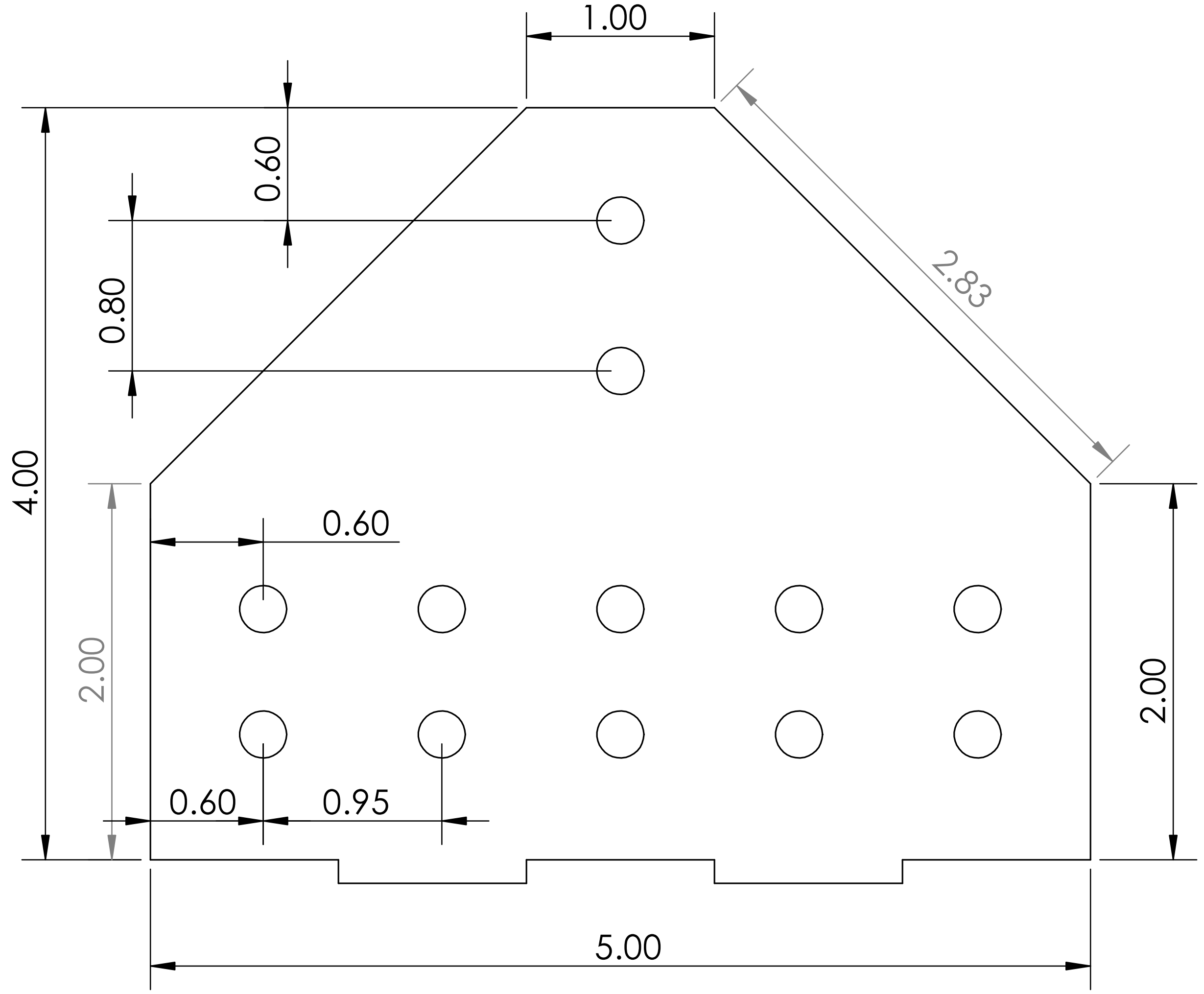
UNLESS OTHERWISE SPECIFIED:
DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±
INTERPRET GEOMETRIC TOLERANCING PER:
MATERIAL GRADE 50 ASTM 1011
FINISH UNFINISHED
DO NOT SCALE DRAWING

	NAME	DATE
DRAWN	ELK	3.29.21
CHECKED	MEB	3.30.21
ENG APPR.		
MFG APPR.		
Q.A.		
COMMENTS:		

TITLE:
SIDE PLATES FOR CONN 0.5B

SIZE B | DWG. NO. 0.5B-1 | REV 1
SCALE: 1:1 | WEIGHT: | SHEET 1 OF 1

QTY REQUIRED: 12
MATERIAL: GRADE 50 ASTM 1011
THICKNESS: 11 GA



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B-2	CONN. C	
NEXT ASSY	USED ON	
APPLICATION		

UNLESS OTHERWISE SPECIFIED:
DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±

INTERPRET GEOMETRIC TOLERANCING PER:

MATERIAL
GRADE 50 ASTM 1011

FINISH
UNFINISHED

DO NOT SCALE DRAWING

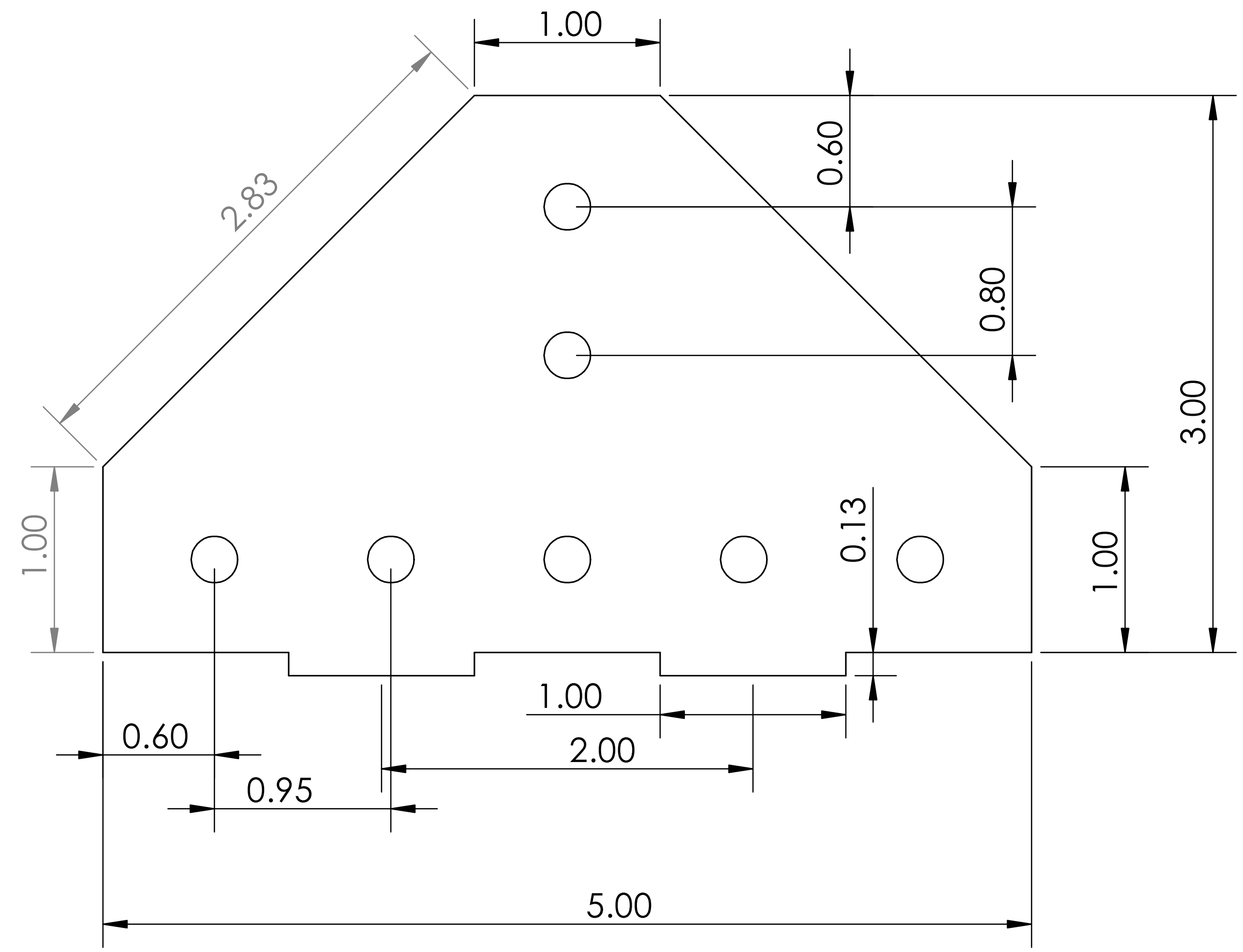
	NAME	DATE
DRAWN	ELK	3.29.21
CHECKED	MEB	3.30.21
ENG APPR.		
MFG APPR.		
Q.A.		
COMMENTS:		

TITLE:
SIDE PLATE FOR CONN. C

SIZE	DWG. NO.	REV
B	C-1	1

SCALE: 1:1 WEIGHT: SHEET 1 OF 1

QTY REQUIRED: 16
MATERIAL: GRADE 50 ASTM 1011
THICKNESS: 11 GA



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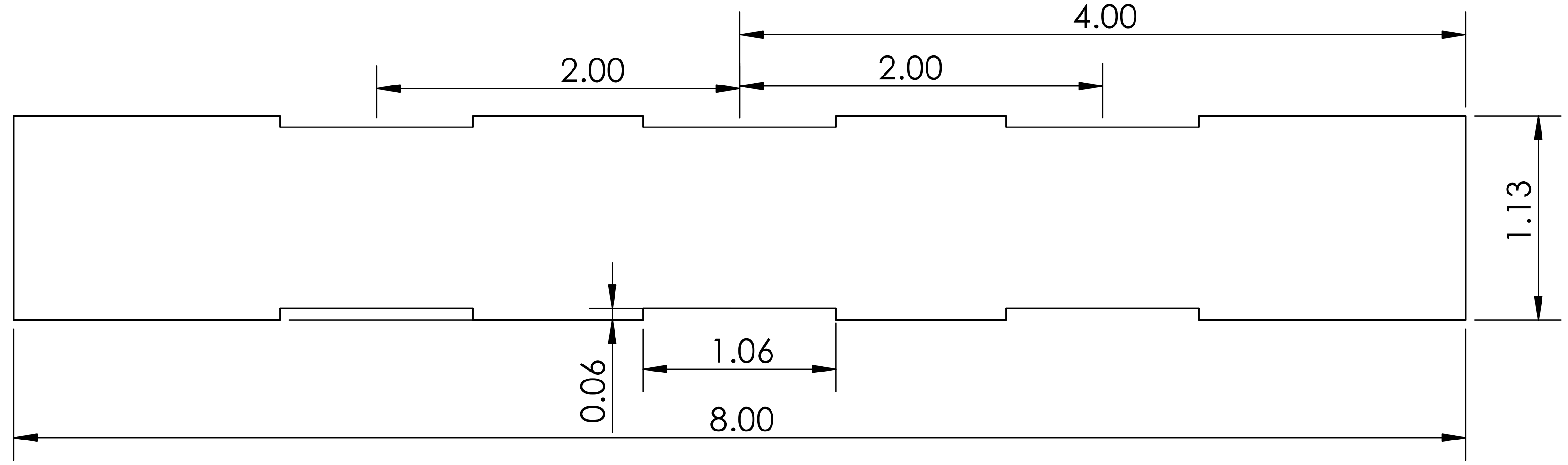
B-2	CONN. D	UNLESS OTHERWISE SPECIFIED: DIMENSIONS ARE IN INCHES TOLERANCES: FRACTIONAL± ANGULAR: MACH± BEND ± TWO PLACE DECIMAL ± THREE PLACE DECIMAL ±		NAME	DATE
NEXT ASSY	USED ON	INTERPRET GEOMETRIC TOLERANCING PER:	DRAWN	ELK	3.29.21
APPLICATION		MATERIAL GRADE 50 ASTM 1011	CHECKED	MEB	3.30.21
		FINISH UNFINISHED	ENG APPR.		
		DO NOT SCALE DRAWING	MFG APPR.		
			Q.A.		
			COMMENTS:		

TITLE: SIDE PLATE FOR CONN. D

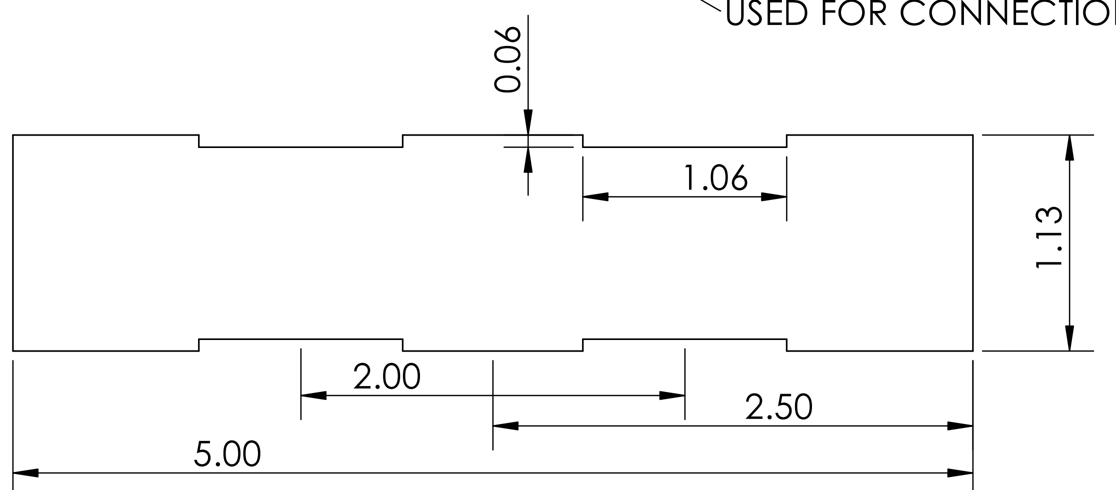
SIZE B | DWG. NO. D-1 | REV 1

SCALE: 1:1 | WEIGHT: | SHEET 1 OF 1



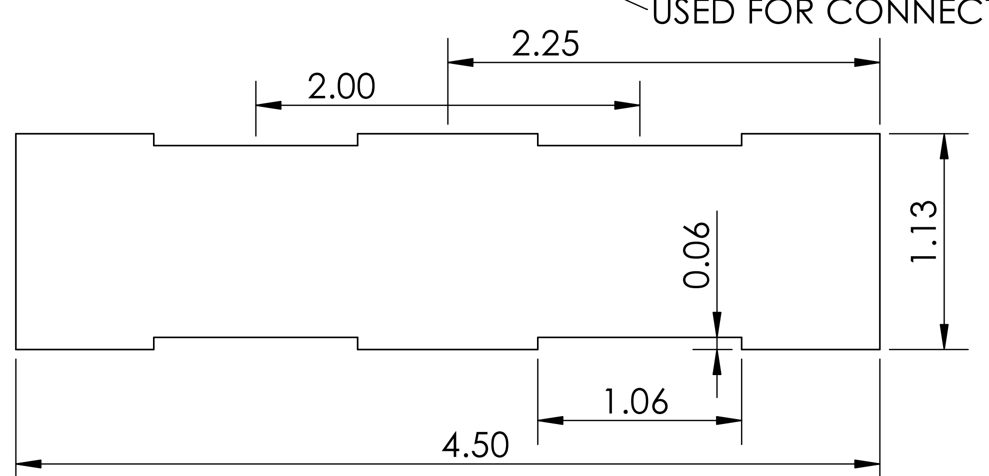
PART NO. A-2

USED FOR CONNECTION A AND B



PART NO. B-2

USED FOR CONNECTION C AND D



PART NO. C-2

USED FOR CONNECTION 0.5B

PART. NO	QTY REQUIRED
A-2	10
B-2	14
C-2	4

MATERIAL: GRADE 50 ASTM 1011
THICKNESS: 11 GA

NEXT ASSY | USED ON
APPLICATION

UNLESS OTHERWISE SPECIFIED:
DIMENSIONS ARE IN INCHES
TOLERANCES:
FRACTIONAL±
ANGULAR: MACH± BEND ±
TWO PLACE DECIMAL ±
THREE PLACE DECIMAL ±
INTERPRET GEOMETRIC TOLERANCING PER:
MATERIAL GRADE 50 ASTM 1011
FINISH UNFINISHED
DO NOT SCALE DRAWING

	NAME	DATE
DRAWN	ELK	3.29.21
CHECKED	MEB	3.30.21
ENG APPR.		
MFG APPR.		
Q.A.		
COMMENTS:		

TITLE:
BOTTOM PLATES FOR ALL CONNS.

SIZE B | DWG. NO. BP-1 | REV 1
SCALE: 1:1 | WEIGHT: | SHEET 1 OF 1

Appendix B - Analysis of Existing Bridge

Contents:

- Calculations for Existing Connection Capacities A - F
- Excel Table Legend
- For Load Case 1 - 6:
  - Excel Determination of Failure Areas [REFER TO LEGEND]
  - RISA Loading Graphics for Predicted Max Loading
  - RISA Deflection Graphics

		UNLESS OTHERWISE SPECIFIED:		NAME	DATE	
		DIMENSIONS ARE IN INCHES TOLERANCES: FRACTIONAL ± ANGULAR: MACH ± BEND ± TWO PLACE DECIMAL ± THREE PLACE DECIMAL ±	DRAWN			TITLE:
			CHECKED			
			ENG APPR.			
			MFG APPR.			
PROPRIETARY AND CONFIDENTIAL		INTERPRET GEOMETRIC TOLERANCING PER:	Q.A.			
		MATERIAL	COMMENTS:			SIZE B DWG. NO. Appendix B REV
NEXT ASSY	USED ON	FINISH				
APPLICATION		DO NOT SCALE DRAWING				SCALE: WEIGHT: SHEET 1 OF 1

Calculated Capacities for Existing Connection A

Calculations

Plate Properties (Table 1)		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength	50	ksi
Number of Plates	2	

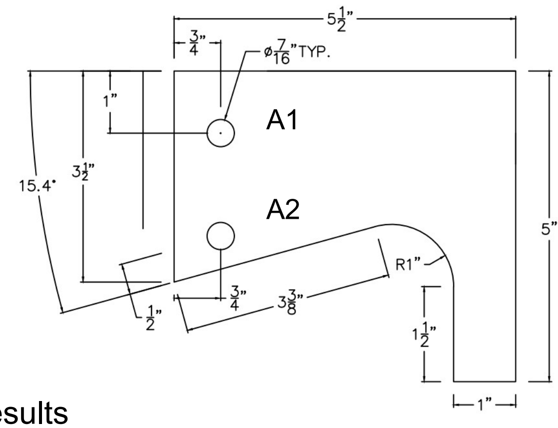
Tensile Stength (YLS) (Table 2)		
Phi_t*Pn	39.375	kip
Ag	0.4375	in^2
Fy	50	ksi
Reduction Factor	0.9	
Number of Plates	2	
Tensile Stength (FLS)		
Phi_t*Pn	33.75	kip
An = Ae	0.375	in^2
Fu	60	ksi
Reduction Factor	0.75	
Number of Plates	2	

Bearing and Tearout Strength at Bolt Hole 1 (Table 3)		
Bearing		
Phi*Rn	12.66	kip
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	8.96	kip
l_c	0.53125	
Reduction Factor	0.75	
Fu	60	ksi

Bearing and Tearout Strength at Bolt Hole 2 (Table 4)		
Bearing		
Phi*Rn	12.66	kip
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	8.96	kip
l_c	0.53125	
Reduction Factor	0.75	
Fu	60	ksi

Continuation of Table 2 (Table 7)	
Ae/Ag	0.857
1.2*(Fy/Fu)	1
1.2*(Fy/Fu) > Ae/Ag, hence tensile rupture controls (FLS	

Connection A
Scale: 1"=6'



Results

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Bearing and Tearout Strength	J3.10	16.1-135

Overall Strengths of Connection B (kip)	
Tensile Strength (YLS)	39.38
Tensile Strength (FLS)	33.75
Strength at A1	8.96
Strength at A2	8.96

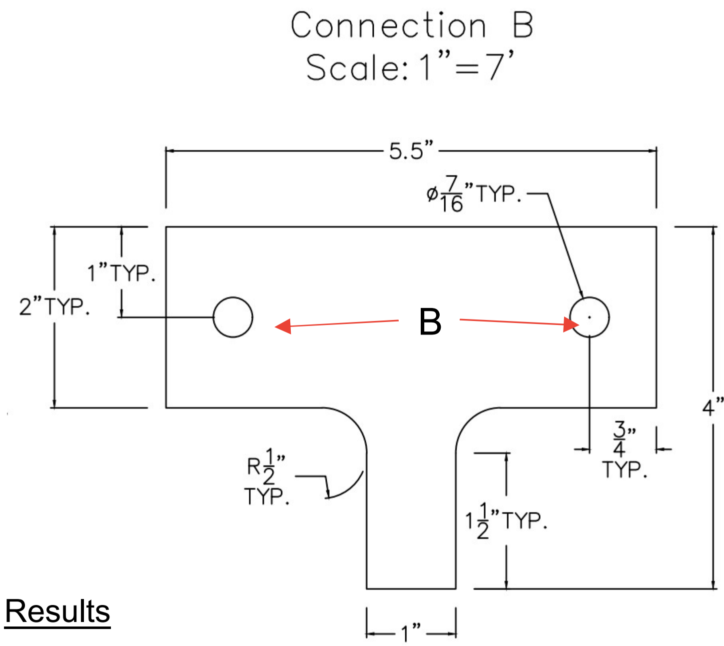
Calculated Capacities for Existing Connection B

Calculations

Plate Properties		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength	50	ksi
Area		
Gross Cross Sectional Area		

Tensile Strength (YLS)		
Phi_t*Pn	22.5	kip
Ag	0.25	in^2
Fy	50	ksi
Reduction Factor	0.9	
Number of Plates	2	
Tensile Stength (FLS)		
Phi_t*Pn	16.875	kip
An	0.1875	in^2
Fu	60	ksi
Reduction Factor	0.75	
Number of Plates	2	

Bearing and Tearout Strength at Bolt Hole		
Bearing		
Phi*Rn	12.66	kip
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	8.96	kip
l_c	0.53125	in
Reduction Factor	0.75	
Fu	60	ksi



Results

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Bearing and Tearout Strength	J3.10	16.1-135

Overall Strengths of Connection B (kip)	
Tensile Strength (YLS)	22.50
Tensile Strength (FLS)	16.88
Strength at B	8.96

Calculated Capacities for Existing Connection C

Calculations

Plate Properties		
Steel Type	ASTM 1011	
Thickness	0.125	n
Yield Strength	50	ksi
Area		
Gross Cross Sectional Area		

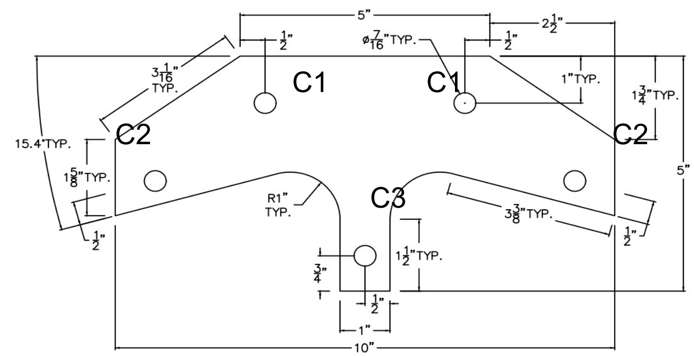
Tensile Stength (YLS)		
Phi_t*Pn	39.375	kip
Ag	0.4375	n^2
Fy	50	ksi
Reduction Factor	0.9	
Number of Plates	2	
Tensile Stength (FLS)		
Phi_t*Pn	33.03	kip
An	0.367	n^2
Fu	60	ksi
Reduction Factor	0.75	
Number of Plates	2	

Bearing and Tearout Strength at Bolt Hole 1		
Bearing		
Phi*Rn	12.66	kip
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	21.09	kip
l_c	1.25	in
Reduction Factor	0.75	
Fu	60	ksi

Bearing and Tearout Strength at Bolt Hole 2		
Bearing		
Phi*Rn	12.66	kip
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	8.44	kip
l_c	0.5	in
Reduction Factor	0.75	
Fu	60	ksi

Bearing and Tearout Strength at Bolt Hole 3		
Bearing		
Phi*Rn	12.66	kip
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	8.96	kip
l_c	0.53125	in
Reduction Factor	0.75	
Fu	60	ksi

Connection C
Scale: 1"=6'



Results

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Bearing and Tearout Strength	J3.10	16.1-135

Overall Strengths of Connection B (kip)	
Tensile Strength (YLS)	39.38
Tensile Strength (FLS)	33.03
Strength at C1	21.09
Strength at C2	8.44
Strength at C3	8.96

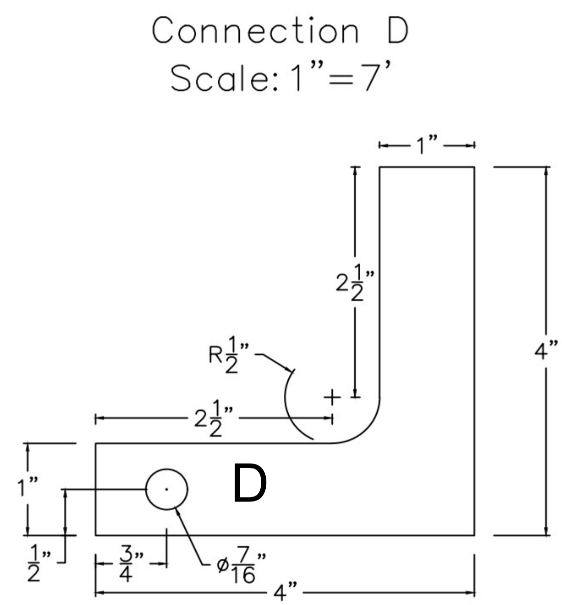
Calculated Capacities for Existing Connection

Calculations

Plate Properties		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength	50	ksi
Area		
Gross Cross Sectional Area		

Tensile Stength (YLS)		
Phi_t*Pn	11.25	kip
Ag	0.125	in^2
Fy	50	ksi
Reduction Factor	0.9	
Number of Plates	2	
Tensile Stength (FLS)		
Phi_t*Pn	5.625	kip
An	0.0625	in^2
Fu	60	ksi
Reduction Factor	0.75	
Number of Plates	2	

Bearing and Tearout Strength at Bolt Hole		
Bearing		
Phi*Rn	12.66	
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	8.96	kip
l_c	0.53125	
Reduction Factor	0.75	
Fu	60	ksi



Results

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Bearing and Tearout Strength	J3.10	16.1-135

Overall Strengths of Connection B (kip)	
Tensile Strength (YLS)	11.25
Tensile Strength (FLS)	5.63
Strength at C1	8.96

Calculated Capacities for Existing Connection E

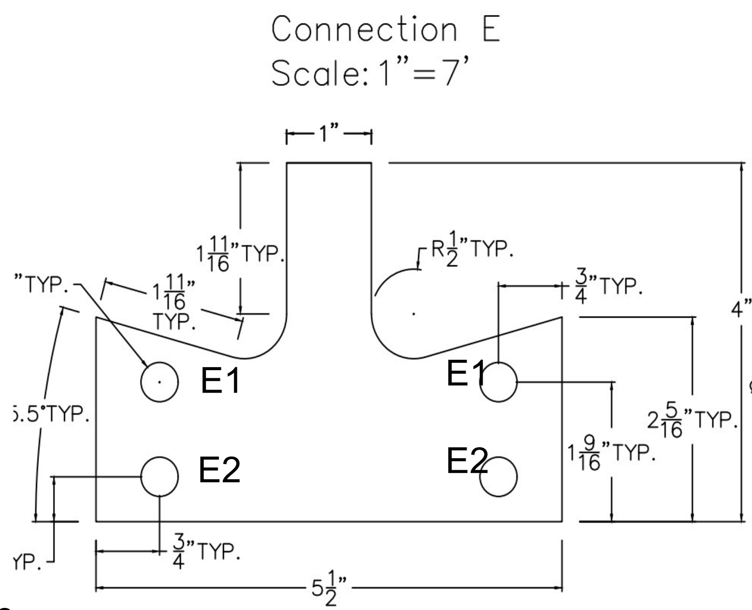
Calculations

Plate Properties		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength	50	ksi
Area		
Gross Cross Sectional Area		

Tensile Stength (YLS)		
Phi_t*Pn	26.019	kip
Ag	0.2891	in^2
Fy	50	ksi
Reduction Factor	0.9	
Number of Plates	2	
Tensile Stength (FLS)		
Phi_t*Pn	20.394	kip
An	0.2266	in^2
Fu	60	ksi
Reduction Factor	0.75	
Number of Plates	2	

Bearing and Tearout Strength at Bolt Hole 1		
Bearing		
Phi*Rn	12.66	
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	10.02	kip
l_c	0.59375	in
Reduction Factor	0.75	
Fu	60	ksi

Bearing and Tearout Strength at Bolt Hole 2		
Bearing		
Phi*Rn	12.66	
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	8.96	kip
l_c	0.53125	in
Reduction Factor	0.75	
Fu	60	ksi



Results

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Bearing and Tearout Strength	J3.10	16.1-135

Overall Strengths of Connection B (kip)	
Tensile Strength (YLS)	26.02
Tensile Strength (FLS)	20.39
Strength at C1	10.02
Strength at C2	8.96

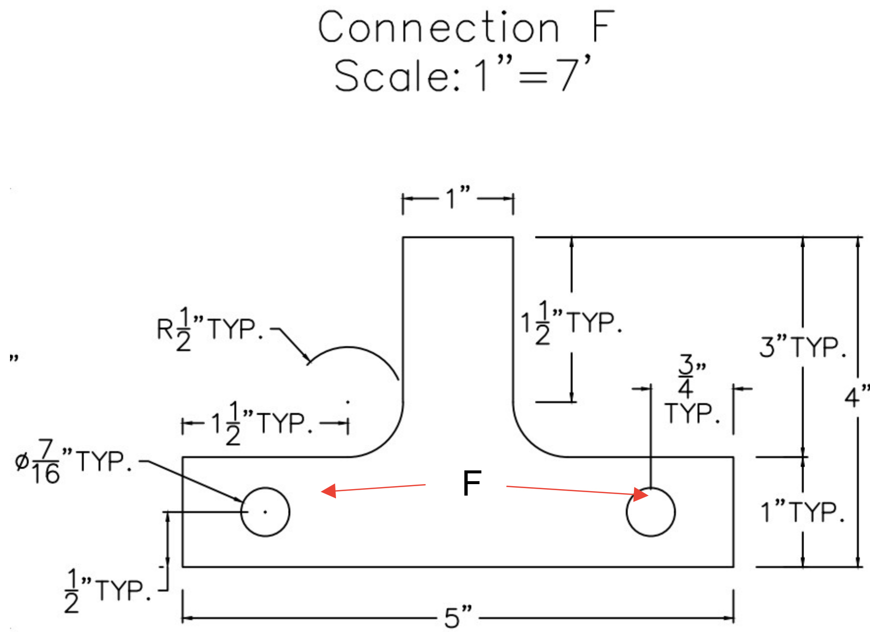
Calculated Capacities for Existing Connection F

Calculations

Plate Properties		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength	50	ksi
Area		
Gross Cross Sectional Area		

Tensile Stength (YLS)		
t Pn	11.25	kip
Ag	0.125	in^2
Fy	50	ksi
Reduction Factor	0.9	
Number of Plates	2	
Tensile Stength (FLS)		
Phi_t*Pn	5.625	kip
An	0.0625	in^2
Fu	60	ksi
Reduction Factor	0.75	
Number of Plates	2	

Bearing and Tearout Strength at Bolt Hole		
Bearing		
Phi*Rn	12.66	
Reduction Factor	0.75	
Fu	60	ksi
Tearout		
Phi*Rn	8.96	kip
l_c	0.53125	
Reduction Factor	0.75	
Fu	60	ksi



Results

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Bearing and Tearout Strength	J3.10	16.1-135

Overall Strengths of Connection B (kip)	
Tensile Strength (YLS)	11.25
Tensile Strength (FLS)	5.63
Strength at C1	8.96

Determination of Failure Locations for the Existing Bridge Design

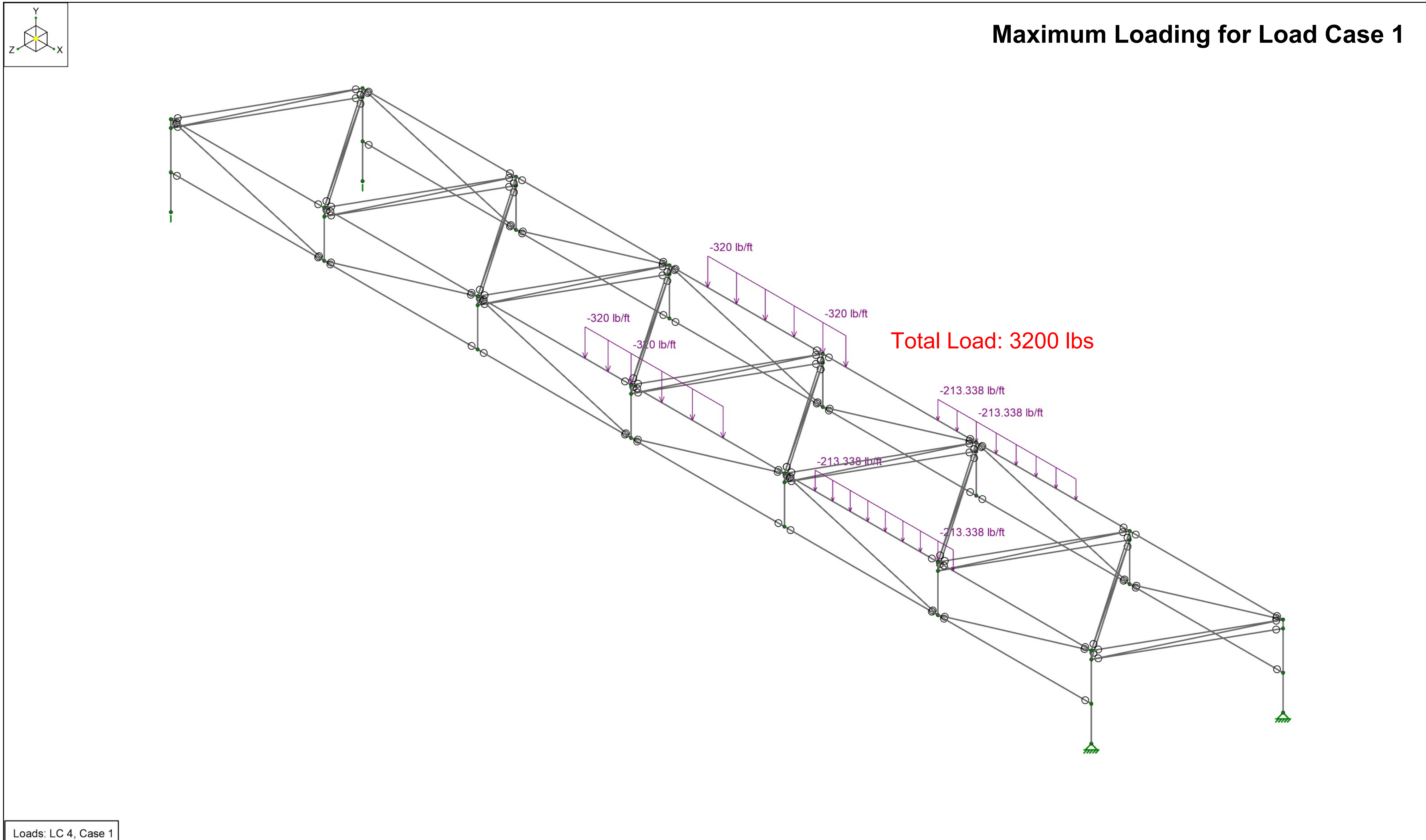
Max Internal Axial Forces in Web Members
Versus
Capacity of Corresponding Connections

Legend	
Axial [lb] Column Color Scheme [+] : Compression *(this column only)* [-] : Tension *(this column only)*	Bottom 50% of Force Distribution Values
	Median Values [negligible axial force]
	Top 50% of Force Distribution Values
Demand vs Capacity Column Color Scheme Demonstrates % Loaded for each connection in terms of capacity	Top 50% of Values [Farthest from Failure]
	Median Values [Not Predicted to Fail]
	Bottom 50% of Values [Closest to Failure]
Plan Set ID Color Scheme *See Existing Plan Set in Appendix A for reference*	Top Chord Members
	Bottom Chord Members
	Vertical Columns at Bridge Ends
	Web Members

Load Case 1: Analysis of Connection Capacities versus Axial Loading

RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	Connection 1	Connection 2	Connection 1 Capacity, kips	Connection 2 Capacity, kips	Controlling Capacity	% Loaded
M56A	BC	2588.774	-2.59	B	C1	8.96	12.66	8.96	-28.89%
M58	BC	6453.102	-6.45	B	C1	8.96	12.66	8.96	-72.02%
M59A	BC	3053.647	-3.05	B	C1	8.96	12.66	8.96	-34.08%
M60A	AB	3040.53	-3.04	A1	B	8.96	8.96	8.96	-33.93%
M62A	EF	-5072.81	5.07	E2	F	8.96	5.625	5.625	90.18%
M63A	EF	-5055.19	5.06	E2	F	8.96	5.625	5.625	89.87%
M64A	EF	-5598.41	5.60	E2	F	8.96	5.625	5.625	99.53%
M65A	EF	-5563.79	5.56	E2	F	8.96	5.625	5.625	98.91%
M66A	AC	-22.743	0.02	D	E2	5.625	8.96	5.625	0.40%
M67A	AA	929.108	-0.93	A2	A2	8.96	8.96	8.96	-10.37%
M35	AA	645.508	-0.65	A2	D	8.96	5.625	5.625	-11.48%
M33	AB	2120.753	-2.12	A1	B	8.96	8.96	8.96	-23.67%
M34	BC	2142.072	-2.14	B	C1	8.96	12.66	8.96	-23.91%
M35A	BC	6144.011	-6.14	B	C1	8.96	12.66	8.96	-68.57%
M36	BC	6179.237	-6.18	B	C1	8.96	12.66	8.96	-68.96%
M37	BC	3558.323	-3.56	B	C1	8.96	12.66	8.96	-39.71%
M38	AB	3547.491	-3.55	A1	B	8.96	8.96	8.96	-39.59%
M40	AC	-12.396	0.01	D	E2	5.625	8.96	5.625	0.22%
M41	EF	-4233.02	4.23	E2	F	8.96	5.625	5.625	75.25%
M42	EF	-4272.65	4.27	E2	F	8.96	5.625	5.625	75.96%
M43	EF	-5576.45	5.58	E2	F	8.96	5.625	5.625	99.14%
M44	EF	-5598.28	5.60	E2	F	8.96	5.625	5.625	99.53%
M45	AC	-34.201	0.03	D	E2	5.625	8.96	5.625	0.61%
M45A	AA	1079.495	-1.08	A2	D	8.96	5.625	5.625	-19.19%
M56	AE	-2208.94	2.21	A2	E1	8.96	10.02	8.96	24.65%
M57A	EC	2170.691	-2.17	C2	E1	8.44	10.02	8.44	-25.72%
M58A	CE	-1957.13	1.96	C2	E1	8.44	10.02	8.44	23.19%
M59	EC	-620.418	0.62	C2	E1	8.44	10.02	8.44	7.35%
M60	CE	2141.988	-2.14	C2	E1	8.44	10.02	8.44	-25.38%
M61	AE	-3669.96	3.67	A2	E1	8.96	10.02	8.96	40.96%
M64	EC	2591.568	-2.59	C2	E1	8.44	10.02	8.44	-30.71%
M65	CE	-1495.87	1.50	C2	E1	8.44	10.02	8.44	17.72%
M66	EC	-906.513	0.91	C2	E1	8.44	10.02	8.44	10.74%
M67	CE	2596.802	-2.60	C2	E1	8.44	10.02	8.44	-30.77%
M68	AE	-3165.41	3.17	A2	E1	8.96	10.02	8.96	35.33%
M61A	AC	-14.49	0.01	D	E2	5.625	8.96	5.625	0.26%
M67B	AE	-2700.02	2.70	A2	E1	8.96	10.02	8.96	30.13%
M41A	BE	6.712	-0.01	B	E2	8.96	8.96	8.96	-0.07%
M42A	CF	18.438	-0.02	C3	F	8.96	5.625	5.625	-0.33%
M43A	BE	767.453	-0.77	B	E2	8.96	8.96	8.96	-8.57%
M44A	CF	39.208	-0.04	C3	F	8.96	5.625	5.625	-0.70%
M45B	BE	444.708	-0.44	B	E2	8.96	8.96	8.96	-4.96%
M46	BE	172.619	-0.17	B	E2	8.96	8.96	8.96	-1.93%
M47	CF	52.64	-0.05	C3	F	8.96	5.625	5.625	-0.94%
M48	BE	719.425	-0.72	B	E2	8.96	8.96	8.96	-8.03%
M49	CF	30.72	-0.03	C3	F	8.96	5.625	5.625	-0.55%
M50	BE	28.473	-0.03	B	E2	8.96	8.96	8.96	-0.32%
M52A	AA	802.527	-0.80	A2	D	8.96	5.625	5.625	-14.27%
M89A	BC	6482.374	-6.48	B	C1	8.96	12.66	8.96	-72.35%
M89B	AB	2596.608	-2.60	A1	B	8.96	8.96	8.96	-28.98%

Maximum Loading for Load Case 1



		SK-4
		Mar 04, 2021
		Existing bridge loaded to failure.r3d

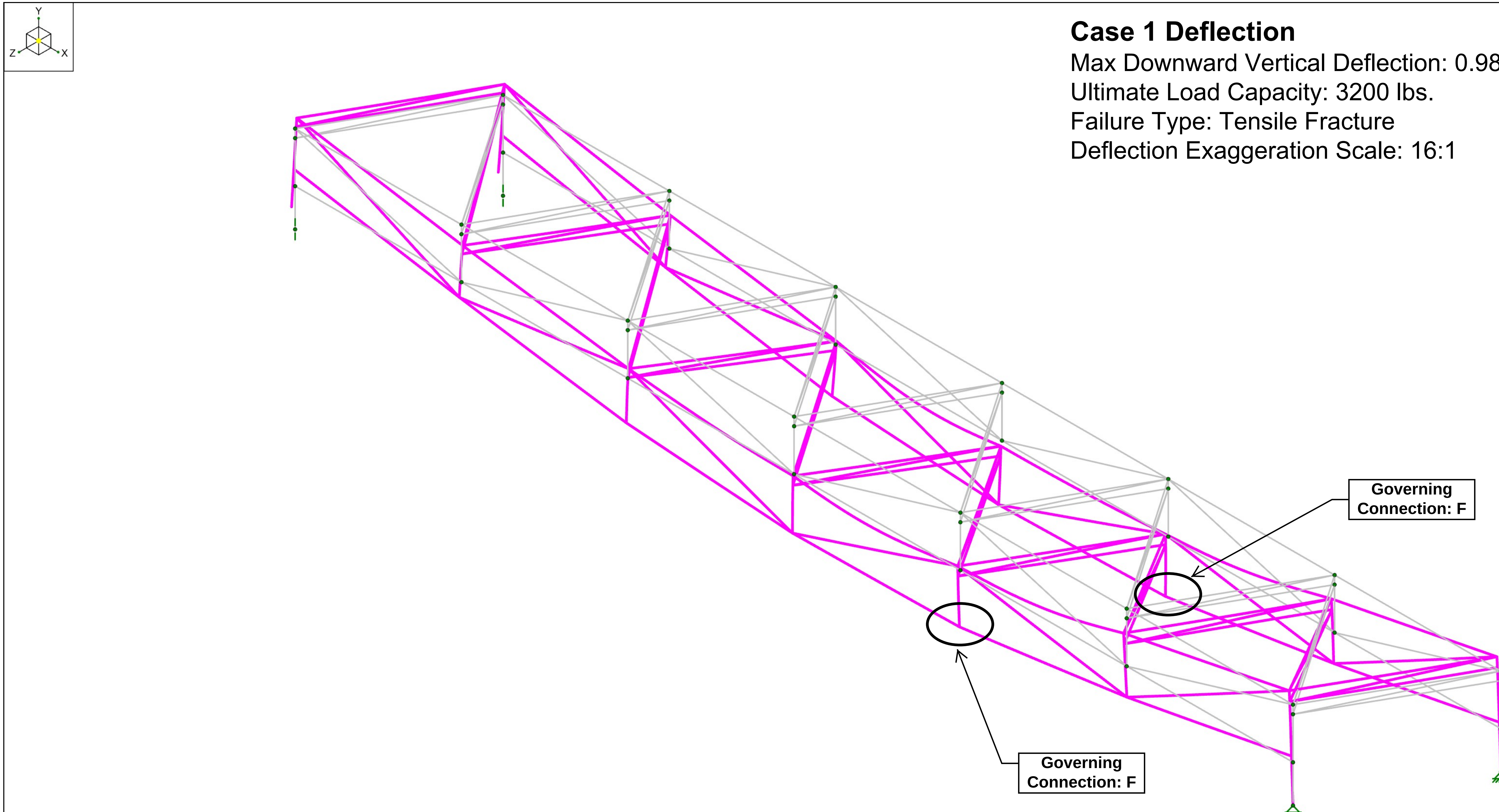
Case 1 Deflection

Max Downward Vertical Deflection: 0.987 in

Ultimate Load Capacity: 3200 lbs.

Failure Type: Tensile Fracture

Deflection Exaggeration Scale: 16:1



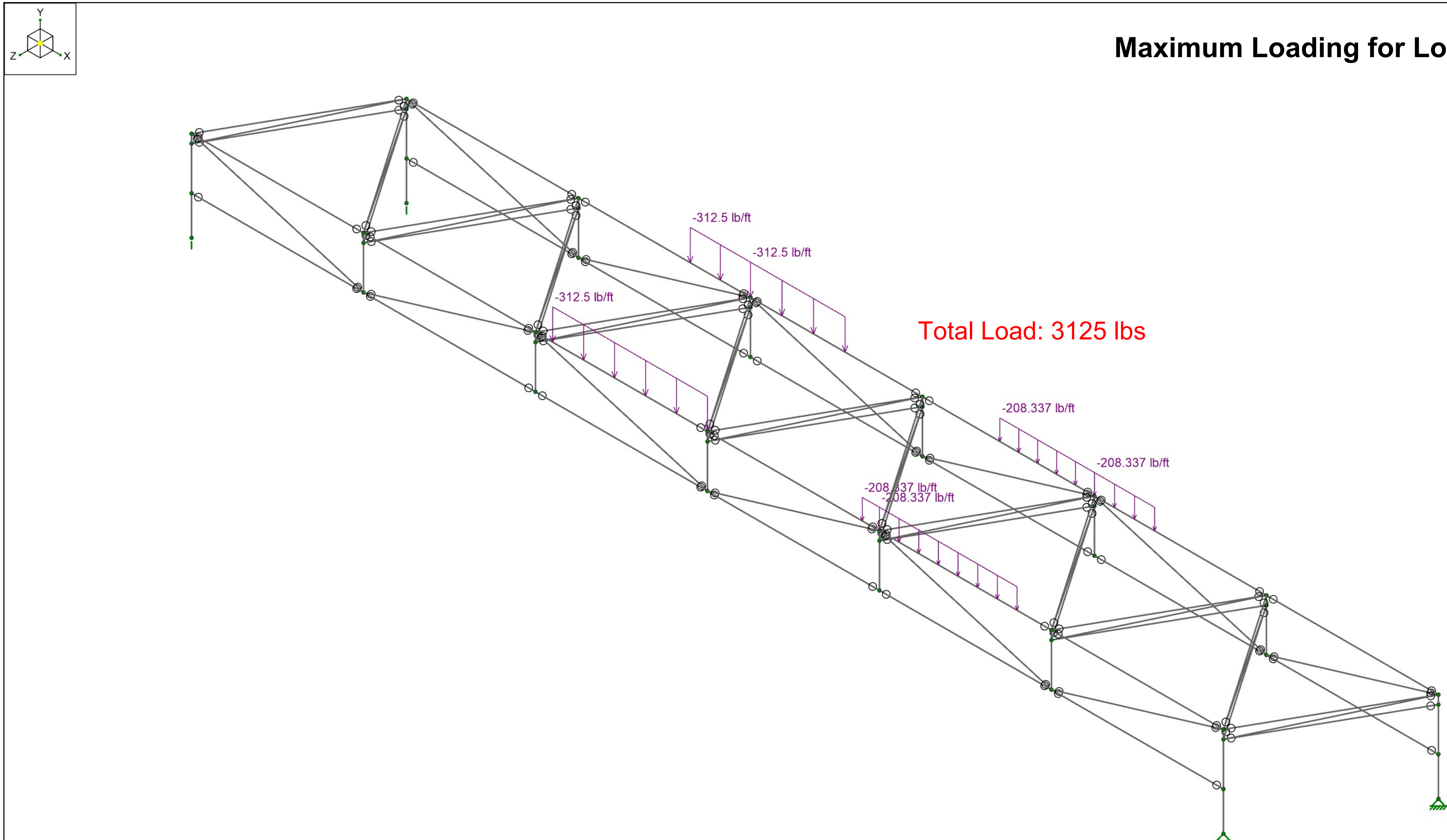
Results for LC 4, Case 1

		SK-3
		Mar 04, 2021
		Existing bridge loaded to failure.r3d

Load Case 2: Analysis of Connection Capacities versus Axial Loading

RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	Connection 1	Connection 2	Connection 1 Capacity, kips	Connection 2 Capacity, kips	Controlling Capacity	% Loaded
M56A	BC	2966.438	-2.97	B	C1	8.96	12.66	8.96	-33.11%
M58	BC	5744.046	-5.74	B	C1	8.96	12.66	8.96	-64.11%
M59A	BC	2576.562	-2.58	B	C1	8.96	12.66	8.96	-28.76%
M60A	AB	2560.569	-2.56	A1	B	8.96	8.96	8.96	-28.58%
M62A	EF	-5592.69	5.59	E2	F	8.96	5.625	5.625	99.43%
M63A	EF	-5570.68	5.57	E2	F	8.96	5.625	5.625	99.03%
M64A	EF	-5000.45	5.00	E2	F	8.96	5.625	5.625	88.90%
M65A	EF	-4964.29	4.96	E2	F	8.96	5.625	5.625	88.25%
M66A	AC	-17.492	0.02	D	E2	5.625	8.96	5.625	0.31%
M67A	AA	781.72	-0.78	A2	A2	8.96	8.96	8.96	-8.72%
M35	AA	757.715	-0.76	A2	D	8.96	5.625	5.625	-13.47%
M33	AB	2485.46	-2.49	A1	B	8.96	8.96	8.96	-27.74%
M34	BC	2502.877	-2.50	B	C1	8.96	12.66	8.96	-27.93%
M35A	BC	5994.154	-5.99	B	C1	8.96	12.66	8.96	-66.90%
M36	BC	6020.457	-6.02	B	C1	8.96	12.66	8.96	-67.19%
M37	BC	3055.767	-3.06	B	C1	8.96	12.66	8.96	-34.10%
M38	AB	3051.988	-3.05	A1	B	8.96	8.96	8.96	-34.06%
M40	AC	-16.162	0.02	D	E2	5.625	8.96	5.625	0.29%
M41	EF	-4959.58	4.96	E2	F	8.96	5.625	5.625	88.17%
M42	EF	-4999.51	5.00	E2	F	8.96	5.625	5.625	88.88%
M43	EF	-5214.34	5.21	E2	F	8.96	5.625	5.625	92.70%
M44	EF	-5236.3	5.24	E2	F	8.96	5.625	5.625	93.09%
M45	AC	-24.504	0.02	D	E2	5.625	8.96	5.625	0.44%
M45A	AA	931.286	-0.93	A2	D	8.96	5.625	5.625	-16.56%
M56	AE	-2588.2	2.59	A2	E1	8.96	10.02	8.96	28.89%
M57A	EC	2547.911	-2.55	C2	E1	8.44	10.02	8.44	-30.19%
M58A	CE	-1044.61	1.04	C2	E1	8.44	10.02	8.44	12.38%
M59	EC	-838.687	0.84	C2	E1	8.44	10.02	8.44	9.94%
M60	CE	2282.768	-2.28	C2	E1	8.44	10.02	8.44	-27.05%
M61	AE	-3163.57	3.16	A2	E1	8.96	10.02	8.96	35.31%
M64	EC	2743.773	-2.74	C2	E1	8.44	10.02	8.44	-32.51%
M65	CE	-210.912	0.21	C2	E1	8.44	10.02	8.44	2.50%
M66	EC	-784.709	0.78	C2	E1	8.44	10.02	8.44	9.30%
M67	CE	2474.325	-2.47	C2	E1	8.44	10.02	8.44	-29.32%
M68	AE	-2666.16	2.67	A2	E1	8.96	10.02	8.96	29.76%
M61A	AC	-21.973	0.02	D	E2	5.625	8.96	5.625	0.39%
M67B	AE	-3078.98	3.08	A2	E1	8.96	10.02	8.96	34.36%
M41A	BE	8.168	-0.01	B	E2	8.96	8.96	8.96	-0.09%
M42A	CF	30.607	-0.03	C3	F	8.96	5.625	5.625	-0.54%
M43A	BE	564.042	-0.56	B	E2	8.96	8.96	8.96	-6.30%
M44A	CF	33.978	-0.03	C3	F	8.96	5.625	5.625	-0.60%
M45B	BE	253.619	-0.25	B	E2	8.96	8.96	8.96	-2.83%
M46	BE	61.354	-0.06	B	E2	8.96	8.96	8.96	-0.68%
M47	CF	44.005	-0.04	C3	F	8.96	5.625	5.625	-0.78%
M48	BE	303.65	-0.30	B	E2	8.96	8.96	8.96	-3.39%
M49	CF	42.769	-0.04	C3	F	8.96	5.625	5.625	-0.76%
M50	BE	95.175	-0.10	B	E2	8.96	8.96	8.96	-1.06%
M52A	AA	908.551	-0.91	A2	D	8.96	5.625	5.625	-16.15%
M89A	BC	5773.141	-5.77	B	C1	8.96	12.66	8.96	-64.43%
M89B	AB	2967.545	-2.97	A1	B	8.96	8.96	8.96	-33.12%

Maximum Loading for Load Case 2

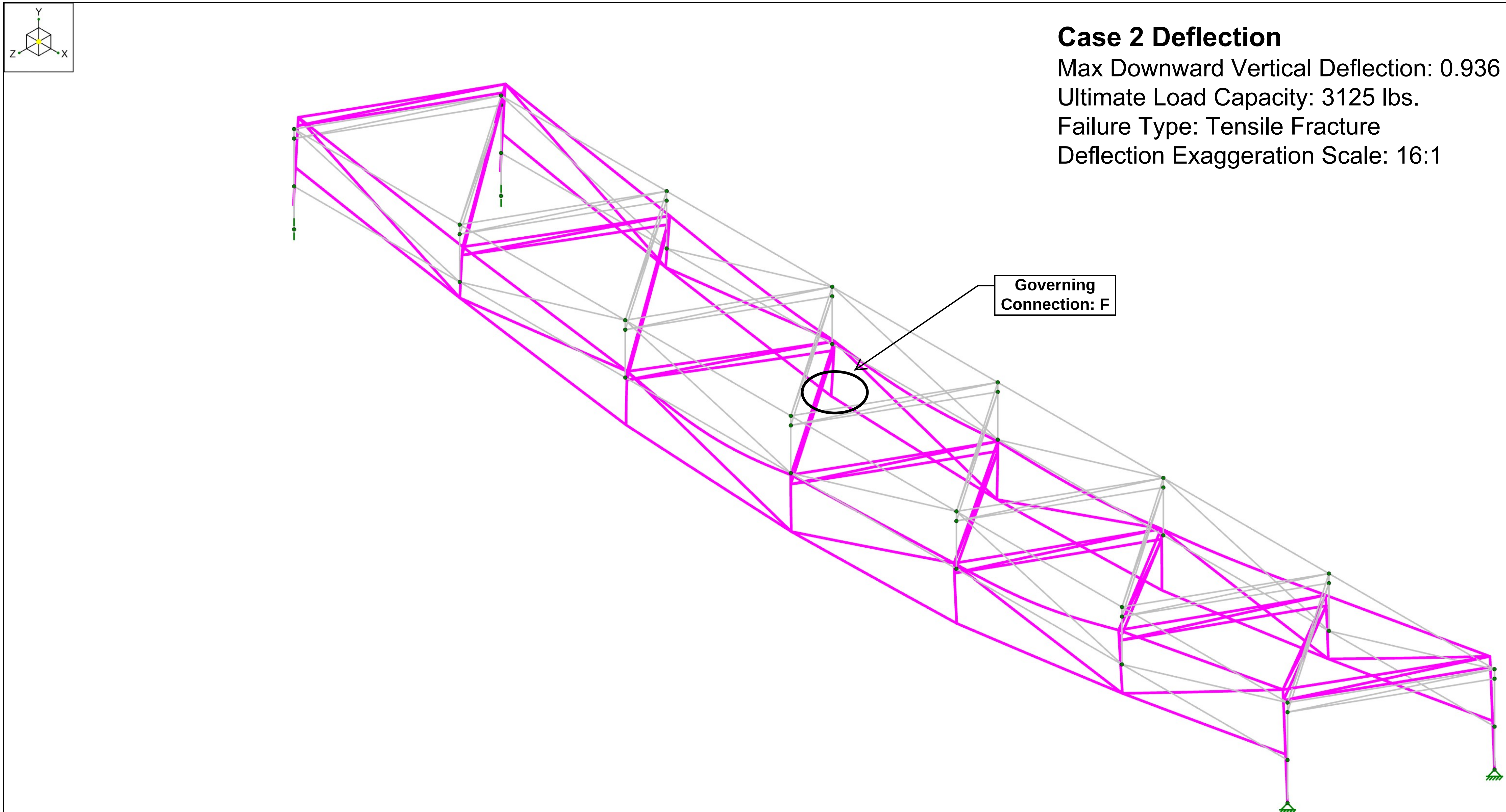


Loads: LC 5, Case 2

		SK-5
		Mar 04, 2021
		Existing bridge loaded to failure.r3d

Case 2 Deflection

Max Downward Vertical Deflection: 0.936 in
Ultimate Load Capacity: 3125 lbs.
Failure Type: Tensile Fracture
Deflection Exaggeration Scale: 16:1



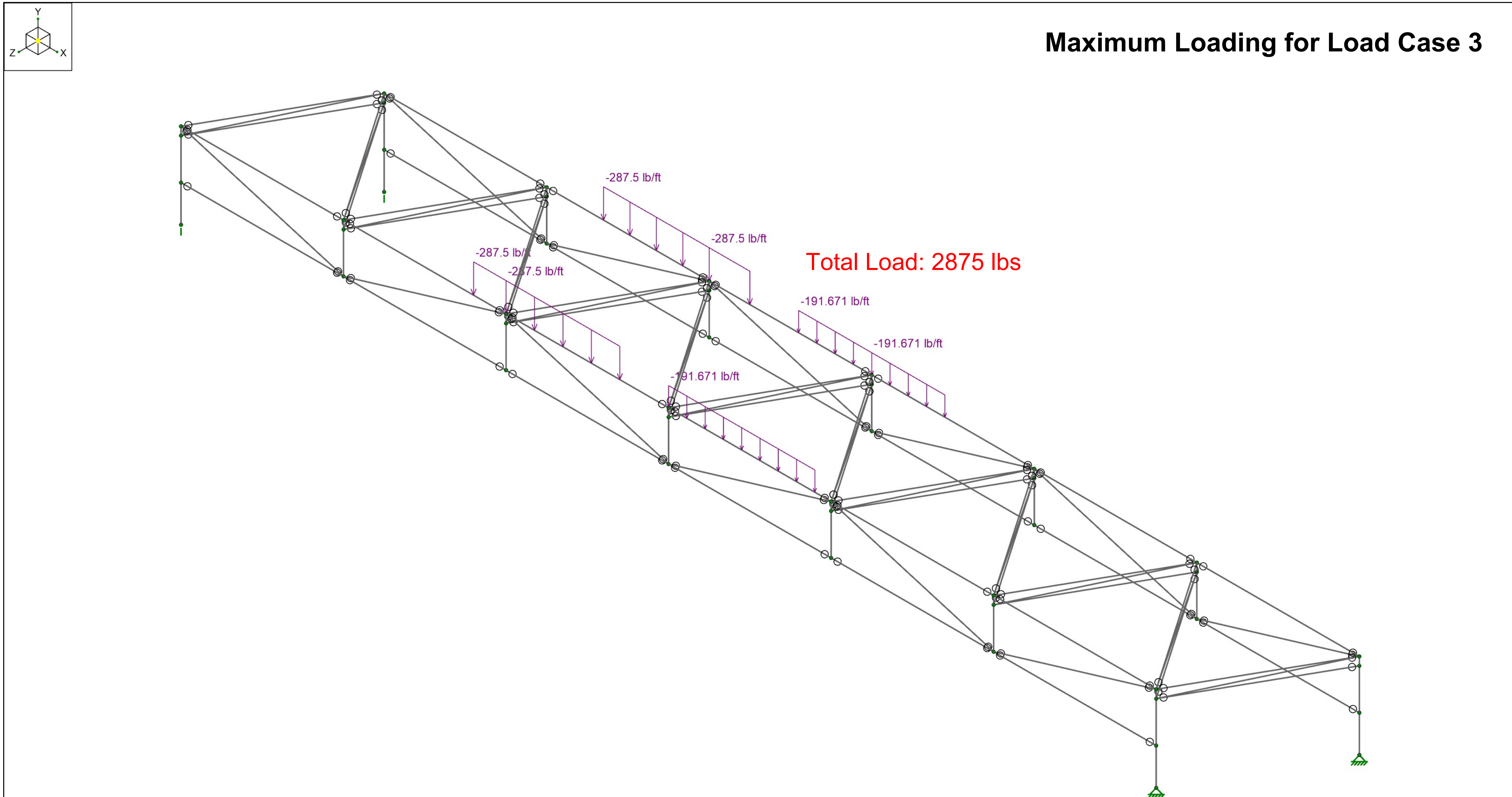
Results for LC 5, Case 2

		SK-4
		Mar 04, 2021
		Existing bridge loaded to failure.r3d

Load Case 3: Analysis of Connection Capacities versus Axial Loading

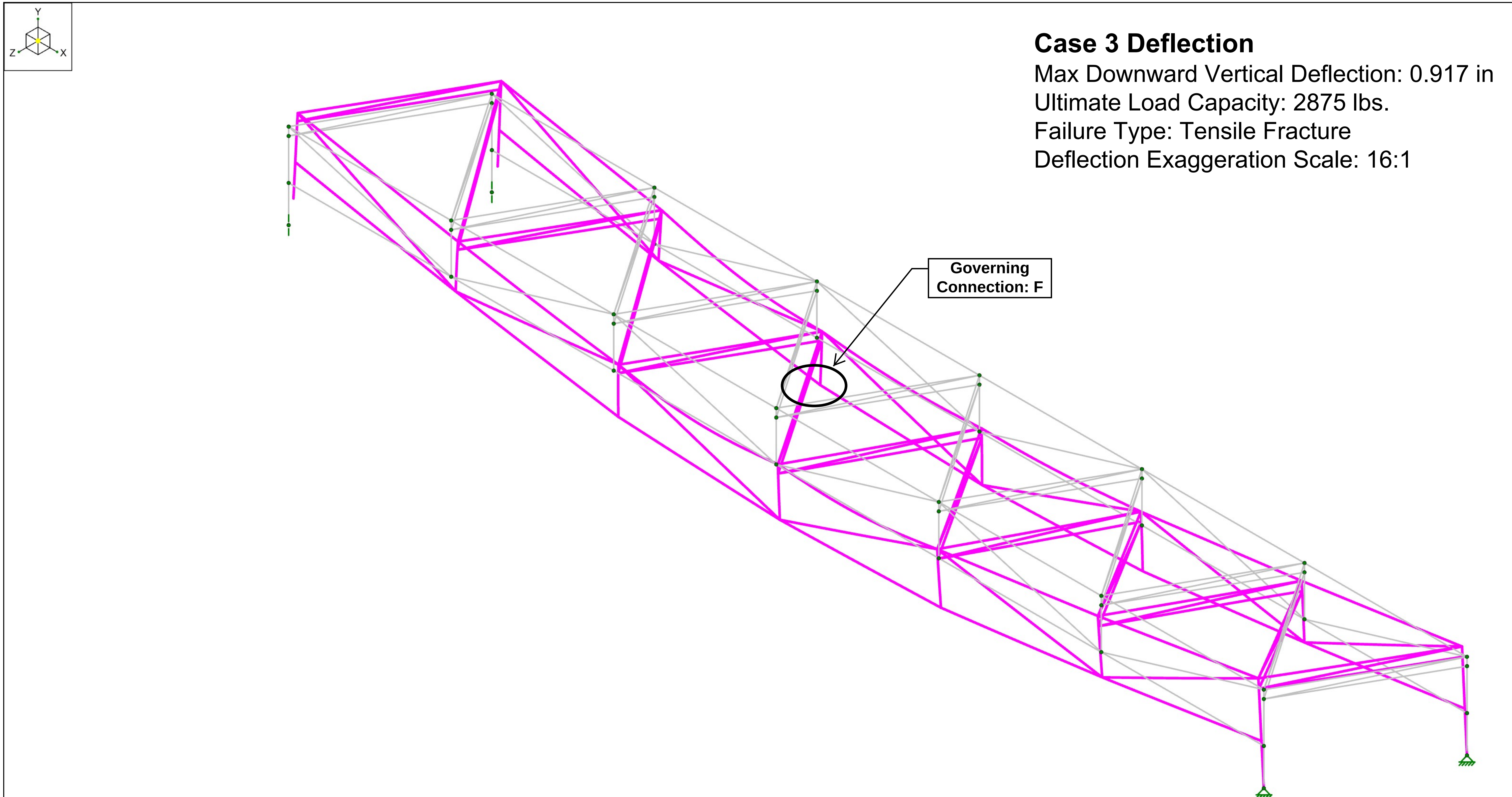
RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	Connection 1	Connection 2	Connection 1 Capacity, kips	Connection 2 Capacity, kips	Controlling Capacity	% Loaded
M56A	BC	3198.32	-3.20	B	C1	8.96	12.66	8.96	-35.70%
M58	BC	5545.363	-5.55	B	C1	8.96	12.66	8.96	-61.89%
M59A	BC	1964.224	-1.96	B	C1	8.96	12.66	8.96	-21.92%
M60A	AB	1943.933	-1.94	A1	B	8.96	8.96	8.96	-21.70%
M62A	EF	-5572.309	5.57	E2	F	8.96	5.625	5.625	99.06%
M63A	EF	-5549.607	5.55	E2	F	8.96	5.625	5.625	98.66%
M64A	EF	-3910.766	3.91	E2	F	8.96	5.625	5.625	69.52%
M65A	EF	-3874.532	3.87	E2	F	8.96	5.625	5.625	68.88%
M66A	AC	-11.284	0.01	D	E2	5.625	8.96	5.625	0.20%
M67A	AA	592.477	-0.59	A2	A2	8.96	8.96	8.96	-6.61%
M35	AA	832.402	-0.83	A2	D	8.96	5.625	5.625	-14.80%
M33	AB	2723.136	-2.72	A1	B	8.96	8.96	8.96	-30.39%
M34	BC	2734.242	-2.73	B	C1	8.96	12.66	8.96	-30.52%
M35A	BC	5943.868	-5.94	B	C1	8.96	12.66	8.96	-66.34%
M36	BC	5968.042	-5.97	B	C1	8.96	12.66	8.96	-66.61%
M37	BC	2367.578	-2.37	B	C1	8.96	12.66	8.96	-26.42%
M38	AB	2374.113	-2.37	A1	B	8.96	8.96	8.96	-26.50%
M40	AC	-18.776	0.02	D	E2	5.625	8.96	5.625	0.33%
M41	EF	-5361.892	5.36	E2	F	8.96	5.625	5.625	95.32%
M42	EF	-5397.491	5.40	E2	F	8.96	5.625	5.625	95.96%
M43	EF	-4617.703	4.62	E2	F	8.96	5.625	5.625	82.09%
M44	EF	-4633.961	4.63	E2	F	8.96	5.625	5.625	82.38%
M45	AC	-11.385	0.01	D	E2	5.625	8.96	5.625	0.20%
M45A	AA	735.709	-0.74	A2	D	8.96	5.625	5.625	-13.08%
M56	AE	-2835.301	2.84	A2	E1	8.96	10.02	8.96	31.64%
M57A	EC	2722.54	-2.72	C2	E1	8.44	10.02	8.44	-32.26%
M58A	CE	-581.636	0.58	C2	E1	8.44	10.02	8.44	6.89%
M59	EC	-1413.393	1.41	C2	E1	8.44	10.02	8.44	16.75%
M60	CE	2365.676	-2.37	C2	E1	8.44	10.02	8.44	-28.03%
M61	AE	-2469.902	2.47	A2	E1	8.96	10.02	8.96	27.57%
M64	EC	2486.622	-2.49	C2	E1	8.44	10.02	8.44	-29.46%
M65	CE	-20.356	0.02	C2	E1	8.44	10.02	8.44	0.24%
M66	EC	-1709.815	1.71	C2	E1	8.44	10.02	8.44	20.26%
M67	CE	1983.359	-1.98	C2	E1	8.44	10.02	8.44	-23.50%
M68	AE	-2024.575	2.02	A2	E1	8.96	10.02	8.96	22.60%
M61A	AC	-28.454	0.03	D	E2	5.625	8.96	5.625	0.51%
M67B	AE	-3303.061	3.30	A2	E1	8.96	10.02	8.96	36.86%
M41A	BE	29.807	-0.03	B	E2	8.96	8.96	8.96	-0.33%
M42A	CF	38.134	-0.04	C3	F	8.96	5.625	5.625	-0.68%
M43A	BE	596.149	-0.60	B	E2	8.96	8.96	8.96	-6.65%
M44A	CF	26.437	-0.03	C3	F	8.96	5.625	5.625	-0.47%
M45B	BE	26.659	-0.03	B	E2	8.96	8.96	8.96	-0.30%
M46	BE	15.261	-0.02	B	E2	8.96	8.96	8.96	-0.17%
M47	CF	24.978	-0.02	C3	F	8.96	5.625	5.625	-0.44%
M48	BE	515.694	-0.52	B	E2	8.96	8.96	8.96	-5.76%
M49	CF	42.213	-0.04	C3	F	8.96	5.625	5.625	-0.75%
M50	BE	235.283	-0.24	B	E2	8.96	8.96	8.96	-2.63%
M52A	AA	968.683	-0.97	A2	D	8.96	5.625	5.625	-17.22%
M89A	BC	5575.19	-5.58	B	C1	8.96	12.66	8.96	-62.22%
M89B	AB	3190.122	-3.19	A1	B	8.96	8.96	8.96	-35.60%

Maximum Loading for Load Case 3



Loads: LC 6, Case 3

		SK-6
		Mar 04, 2021
		Existing bridge loaded to failure.r3d



Case 3 Deflection

Max Downward Vertical Deflection: 0.917 in
Ultimate Load Capacity: 2875 lbs.
Failure Type: Tensile Fracture
Deflection Exaggeration Scale: 16:1

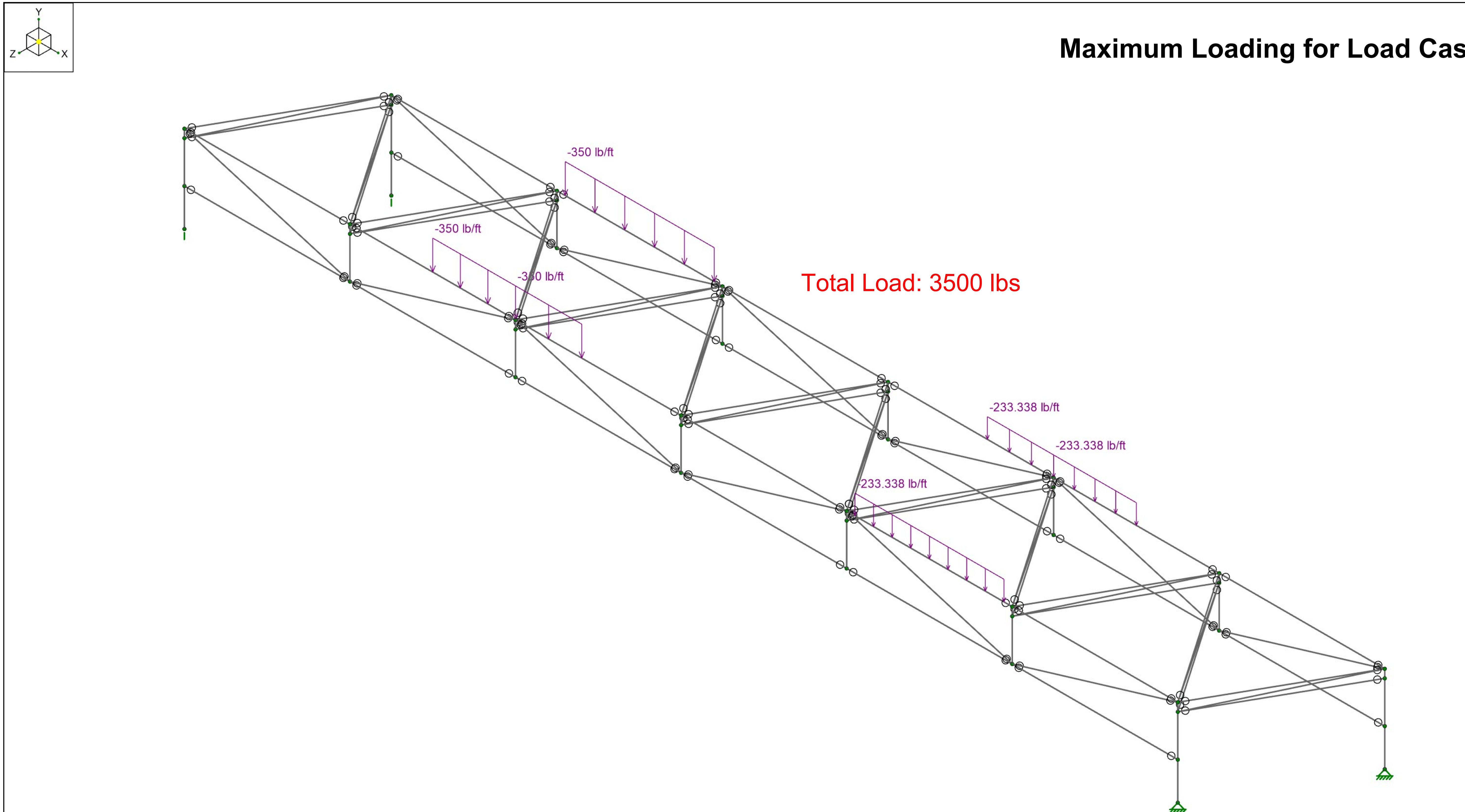
Results for LC 6, Case 3

SK-5
Mar 04, 2021
Existing bridge loaded to failure.r3d

Load Case 4: Analysis of Connection Capacities versus Axial Loading

RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	Connection 1	Connection 2	Connection 1 Capacity, kips	Connection 2 Capacity, kips	Controlling Capacity	% Loaded
M56A	BC	3617.882	-3.62	B	C1	8.96	12.66	8.96	-40.38%
M58	BC	5195.052	-5.20	B	C1	8.96	12.66	8.96	-57.98%
M59A	BC	2585.065	-2.59	B	C1	8.96	12.66	8.96	-28.85%
M60A	AB	2564.889	-2.56	A1	B	8.96	8.96	8.96	-28.63%
M62A	EF	-5332.15	5.33	E2	F	8.96	5.625	5.625	94.79%
M63A	EF	-5303	5.30	E2	F	8.96	5.625	5.625	94.28%
M64A	EF	-4831.93	4.83	E2	F	8.96	5.625	5.625	85.90%
M65A	EF	-4796.03	4.80	E2	F	8.96	5.625	5.625	85.26%
M66A	AC	-16.784	0.02	D	E2	5.625	8.96	5.625	0.30%
M67A	AA	781.634	-0.78	A2	A2	8.96	8.96	8.96	-8.72%
M35	AA	932.682	-0.93	A2	D	8.96	5.625	5.625	-16.58%
M33	AB	3054.507	-3.05	A1	B	8.96	8.96	8.96	-34.09%
M34	BC	3067.852	-3.07	B	C1	8.96	12.66	8.96	-34.24%
M35A	BC	5432.711	-5.43	B	C1	8.96	12.66	8.96	-60.63%
M36	BC	5461.339	-5.46	B	C1	8.96	12.66	8.96	-60.95%
M37	BC	3095.466	-3.10	B	C1	8.96	12.66	8.96	-34.55%
M38	AB	3095.224	-3.10	A1	B	8.96	8.96	8.96	-34.54%
M40	AC	-22.029	0.02	D	E2	5.625	8.96	5.625	0.39%
M41	EF	-5606.83	5.61	E2	F	8.96	5.625	5.625	99.68%
M42	EF	-5640.39	5.64	E2	F	8.96	5.625	5.625	100.27%
M43	EF	-4874.97	4.87	E2	F	8.96	5.625	5.625	86.67%
M44	EF	-4900.65	4.90	E2	F	8.96	5.625	5.625	87.12%
M45	AC	-22.164	0.02	D	E2	5.625	8.96	5.625	0.39%
M45A	AA	947.279	-0.95	A2	D	8.96	5.625	5.625	-16.84%
M56	AE	-3180.1	3.18	A2	E1	8.96	10.02	8.96	35.49%
M57A	EC	2628.171	-2.63	C2	E1	8.44	10.02	8.44	-31.14%
M58A	CE	206.232	-0.21	C2	E1	8.44	10.02	8.44	-2.44%
M59	EC	-613.18	0.61	C2	E1	8.44	10.02	8.44	7.27%
M60	CE	1889.63	-1.89	C2	E1	8.44	10.02	8.44	-22.39%
M61	AE	-3212.03	3.21	A2	E1	8.96	10.02	8.96	35.85%
M64	EC	1801.084	-1.80	C2	E1	8.44	10.02	8.44	-21.34%
M65	CE	84.785	-0.08	C2	E1	8.44	10.02	8.44	-1.00%
M66	EC	-383.727	0.38	C2	E1	8.44	10.02	8.44	4.55%
M67	CE	2292.037	-2.29	C2	E1	8.44	10.02	8.44	-27.16%
M68	AE	-2670.95	2.67	A2	E1	8.96	10.02	8.96	29.81%
M61A	AC	-33.088	0.03	D	E2	5.625	8.96	5.625	0.59%
M67B	AE	-3735.14	3.74	A2	E1	8.96	10.02	8.96	41.69%
M41A	BE	157.976	-0.16	B	E2	8.96	8.96	8.96	-1.76%
M42A	CF	44.173	-0.04	C3	F	8.96	5.625	5.625	-0.79%
M43A	BE	130.134	-0.13	B	E2	8.96	8.96	8.96	-1.45%
M44A	CF	29.123	-0.03	C3	F	8.96	5.625	5.625	-0.52%
M45B	BE	381.475	-0.38	B	E2	8.96	8.96	8.96	-4.26%
M46	BE	116.351	-0.12	B	E2	8.96	8.96	8.96	-1.30%
M47	CF	41.366	-0.04	C3	F	8.96	5.625	5.625	-0.74%
M48	BE	97.106	-0.10	B	E2	8.96	8.96	8.96	-1.08%
M49	CF	37.225	-0.04	C3	F	8.96	5.625	5.625	-0.66%
M50	BE	560.998	-0.56	B	E2	8.96	8.96	8.96	-6.26%
M52A	AA	1092.681	-1.09	A2	D	8.96	5.625	5.625	-19.43%
M89A	BC	5226.396	-5.23	B	C1	8.96	12.66	8.96	-58.33%
M89B	AB	3608.045	-3.61	A1	B	8.96	8.96	8.96	-40.27%

Maximum Loading for Load Case 4

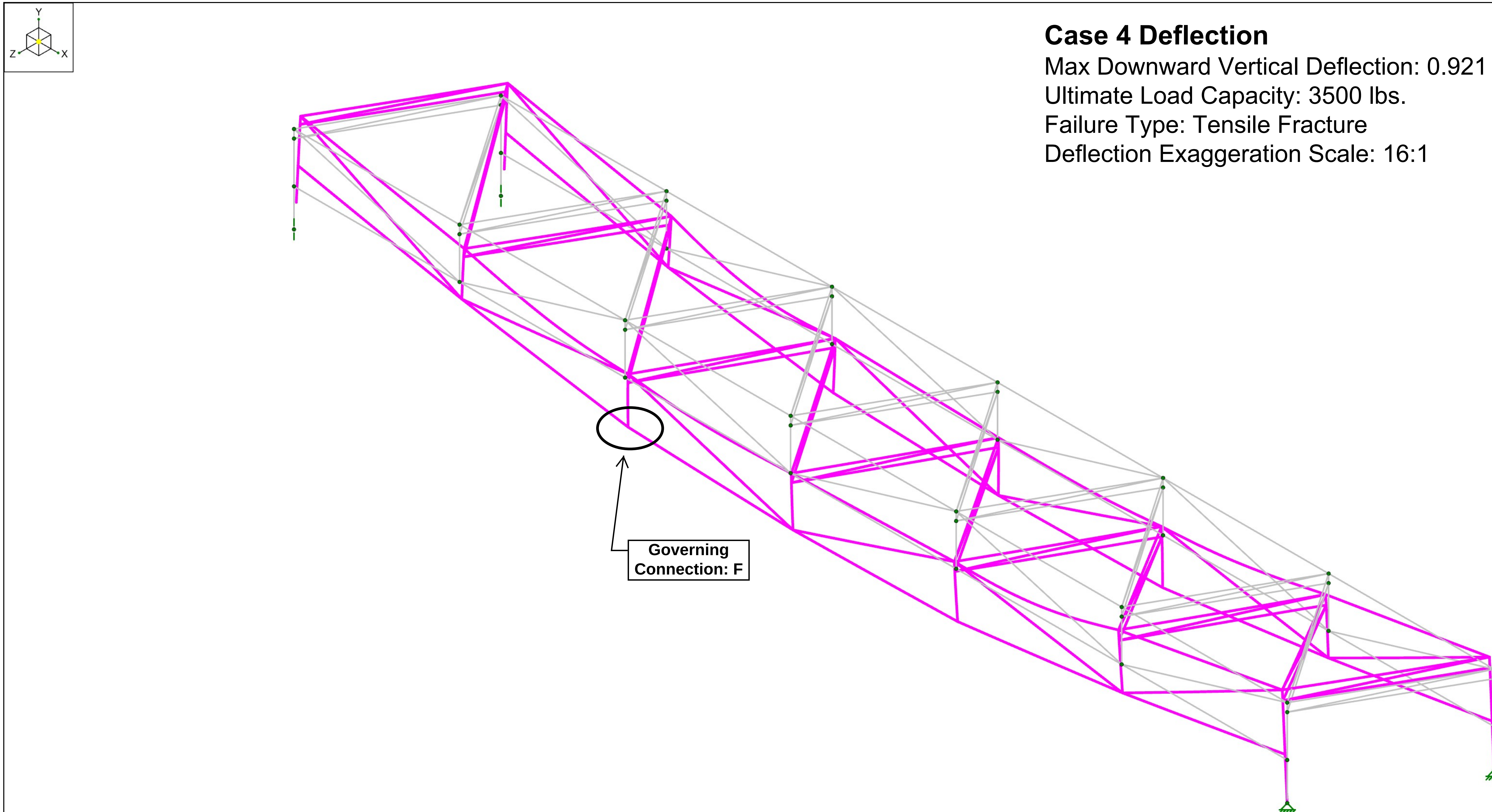


Loads: LC 7, Case 4

		SK-7
		Mar 04, 2021
		Existing bridge loaded to failure.r3d

Case 4 Deflection

Max Downward Vertical Deflection: 0.921 in
Ultimate Load Capacity: 3500 lbs.
Failure Type: Tensile Fracture
Deflection Exaggeration Scale: 16:1



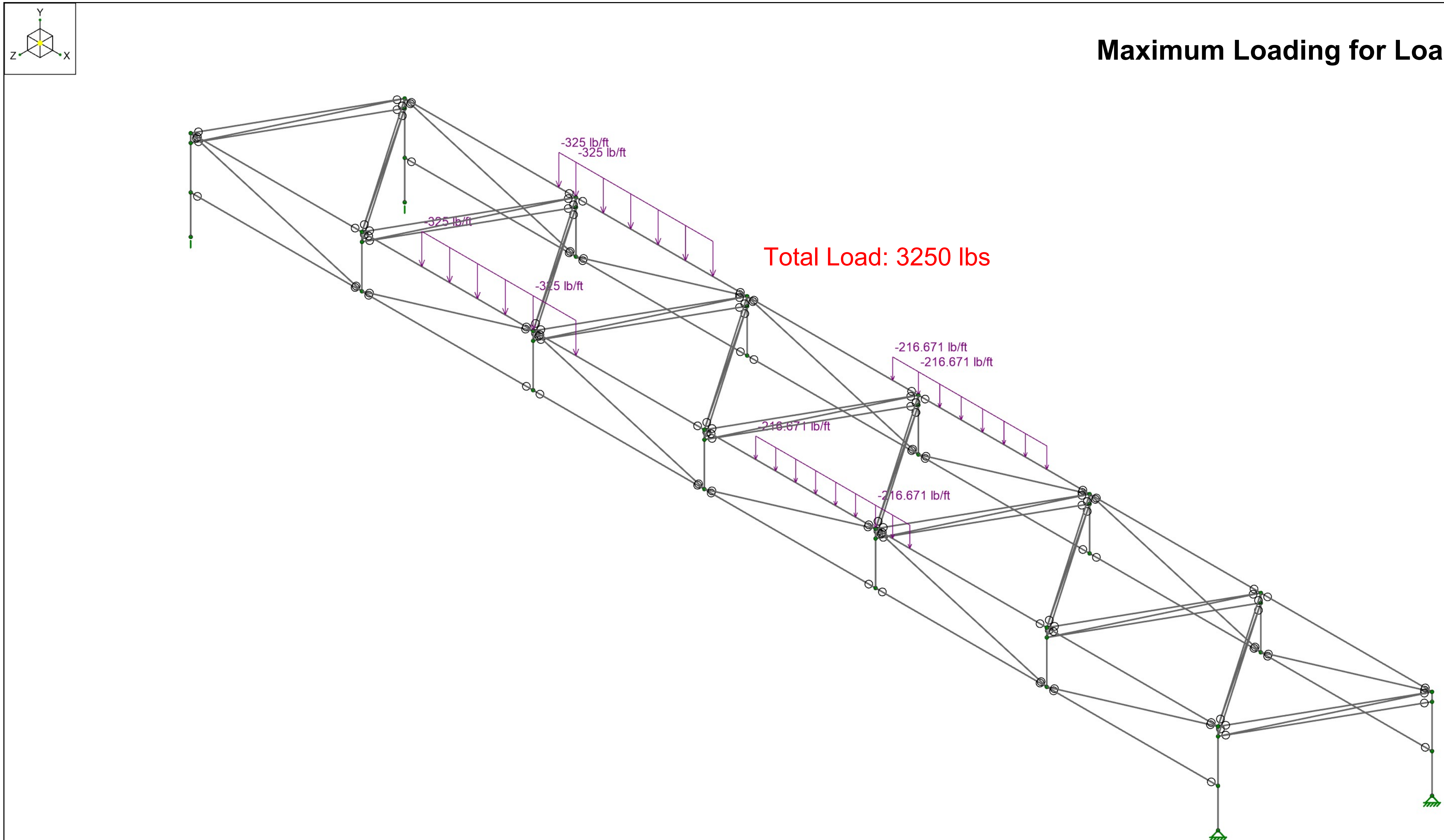
Results for LC 7, Case 4

		SK-6
		Mar 04, 2021
		Existing bridge loaded to failure.r3d

Load Case 5: Analysis of Connection Capacities versus Axial Loading

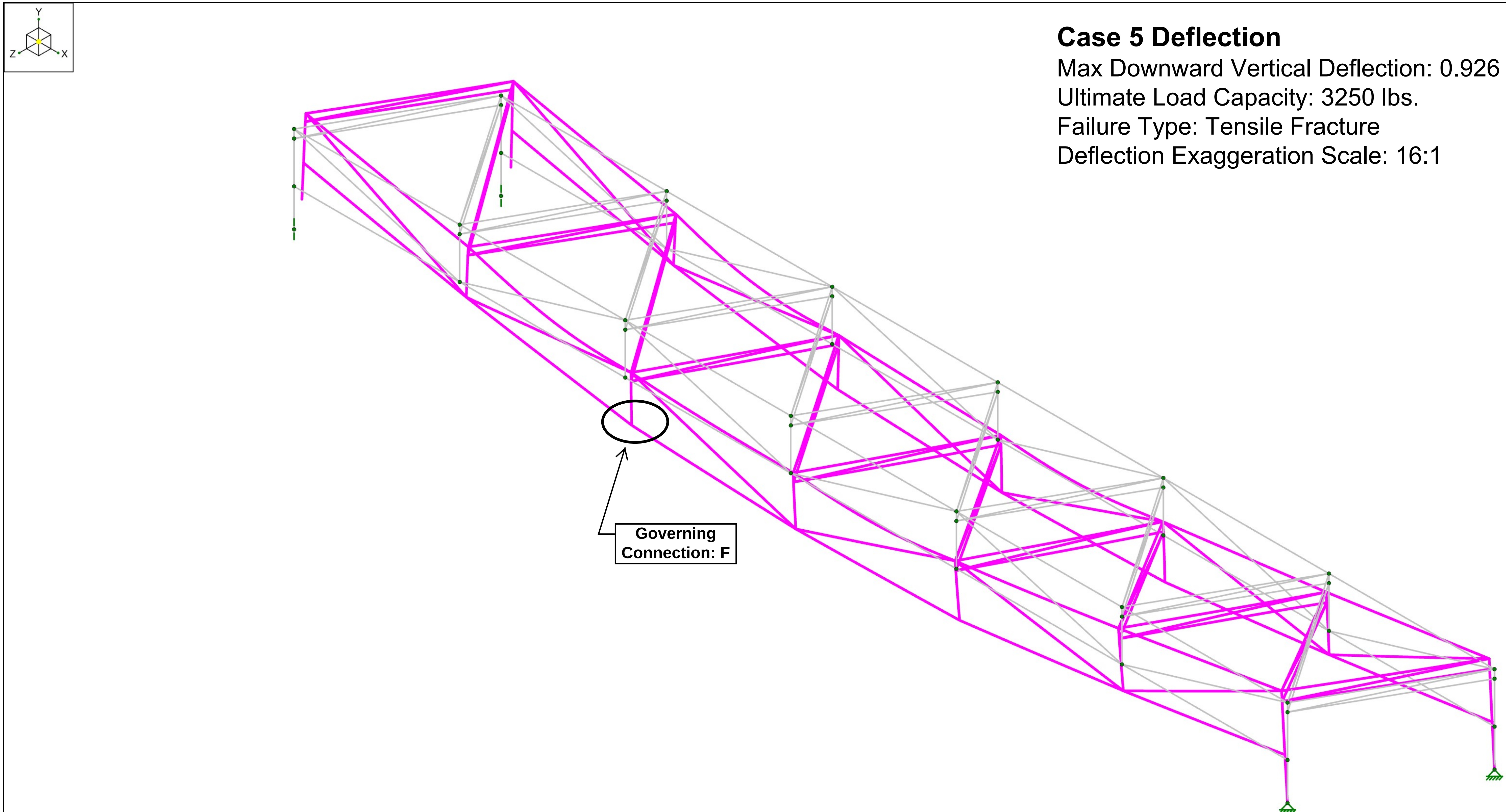
RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	Connection 1	Connection 2	Connection 1 Capacity, kips	Connection 2 Capacity, kips	Controlling Capacity	% Loaded
M56A	BC	3722.069	-3.72	B	C1	8.96	12.66	8.96	-41.54%
M58	BC	5411.187	-5.41	B	C1	8.96	12.66	8.96	-60.39%
M59A	BC	2070.774	-2.07	B	C1	8.96	12.66	8.96	-23.11%
M60A	AB	2047.09	-2.05	A1	B	8.96	8.96	8.96	-22.85%
M62A	EF	-5205.81	5.21	E2	F	8.96	5.625	5.625	92.55%
M63A	EF	-5180.23	5.18	E2	F	8.96	5.625	5.625	92.09%
M64A	EF	-4127.26	4.13	E2	F	8.96	5.625	5.625	73.37%
M65A	EF	-4087.57	4.09	E2	F	8.96	5.625	5.625	72.67%
M66A	AC	-11.925	0.01	D	E2	5.625	8.96	5.625	0.21%
M67A	AA	622.386	-0.62	A2	A2	8.96	8.96	8.96	-6.95%
M35	AA	976.767	-0.98	A2	D	8.96	5.625	5.625	-17.36%
M33	AB	3194.189	-3.19	A1	B	8.96	8.96	8.96	-35.65%
M34	BC	3203.99	-3.20	B	C1	8.96	12.66	8.96	-35.76%
M35A	BC	5608.14	-5.61	B	C1	8.96	12.66	8.96	-62.59%
M36	BC	5635.706	-5.64	B	C1	8.96	12.66	8.96	-62.90%
M37	BC	2509.386	-2.51	B	C1	8.96	12.66	8.96	-28.01%
M38	AB	2518.864	-2.52	A1	B	8.96	8.96	8.96	-28.11%
M40	AC	-23.698	0.02	D	E2	5.625	8.96	5.625	0.42%
M41	EF	-5603.75	5.60	E2	F	8.96	5.625	5.625	99.62%
M42	EF	-5632.95	5.63	E2	F	8.96	5.625	5.625	100.14%
M43	EF	-4852.34	4.85	E2	F	8.96	5.625	5.625	86.26%
M44	EF	-4872.47	4.87	E2	F	8.96	5.625	5.625	86.62%
M45	AC	-10.717	0.01	D	E2	5.625	8.96	5.625	0.19%
M45A	AA	781.492	-0.78	A2	D	8.96	5.625	5.625	-13.89%
M56	AE	-3325.3	3.33	A2	E1	8.96	10.02	8.96	37.11%
M57A	EC	2481.247	-2.48	C2	E1	8.44	10.02	8.44	-29.40%
M58A	CE	11.475	-0.01	C2	E1	8.44	10.02	8.44	-0.14%
M59	EC	-823.627	0.82	C2	E1	8.44	10.02	8.44	9.76%
M60	CE	2465.069	-2.47	C2	E1	8.44	10.02	8.44	-29.21%
M61	AE	-2622.6	2.62	A2	E1	8.96	10.02	8.96	29.27%
M64	EC	1564.162	-1.56	C2	E1	8.44	10.02	8.44	-18.53%
M65	CE	-263.21	0.26	C2	E1	8.44	10.02	8.44	3.12%
M66	EC	-1341	1.34	C2	E1	8.44	10.02	8.44	15.89%
M67	CE	2094.441	-2.09	C2	E1	8.44	10.02	8.44	-24.82%
M68	AE	-2132.02	2.13	A2	E1	8.96	10.02	8.96	23.79%
M61A	AC	-36.87	0.04	D	E2	5.625	8.96	5.625	0.66%
M67B	AE	-3833.82	3.83	A2	E1	8.96	10.02	8.96	42.79%
M41A	BE	243.125	-0.24	B	E2	8.96	8.96	8.96	-2.71%
M42A	CF	42.876	-0.04	C3	F	8.96	5.625	5.625	-0.76%
M43A	BE	249.268	-0.25	B	E2	8.96	8.96	8.96	-2.78%
M44A	CF	31.543	-0.03	C3	F	8.96	5.625	5.625	-0.56%
M45B	BE	42.465	-0.04	B	E2	8.96	8.96	8.96	-0.47%
M46	BE	15.669	-0.02	B	E2	8.96	8.96	8.96	-0.17%
M47	CF	29.25	-0.03	C3	F	8.96	5.625	5.625	-0.52%
M48	BE	478.058	-0.48	B	E2	8.96	8.96	8.96	-5.34%
M49	CF	33.067	-0.03	C3	F	8.96	5.625	5.625	-0.59%
M50	BE	658.789	-0.66	B	E2	8.96	8.96	8.96	-7.35%
M52A	AA	1123.629	-1.12	A2	D	8.96	5.625	5.625	-19.98%
M89A	BC	5441.087	-5.44	B	C1	8.96	12.66	8.96	-60.73%
M89B	AB	3707.664	-3.71	A1	B	8.96	8.96	8.96	-41.38%

Maximum Loading for Load Case 5



Loads: LC 8, Case 5

		SK-8
		Mar 04, 2021
		Existing bridge loaded to failure.r3d



Case 5 Deflection

Max Downward Vertical Deflection: 0.926 in
Ultimate Load Capacity: 3250 lbs.
Failure Type: Tensile Fracture
Deflection Exaggeration Scale: 16:1

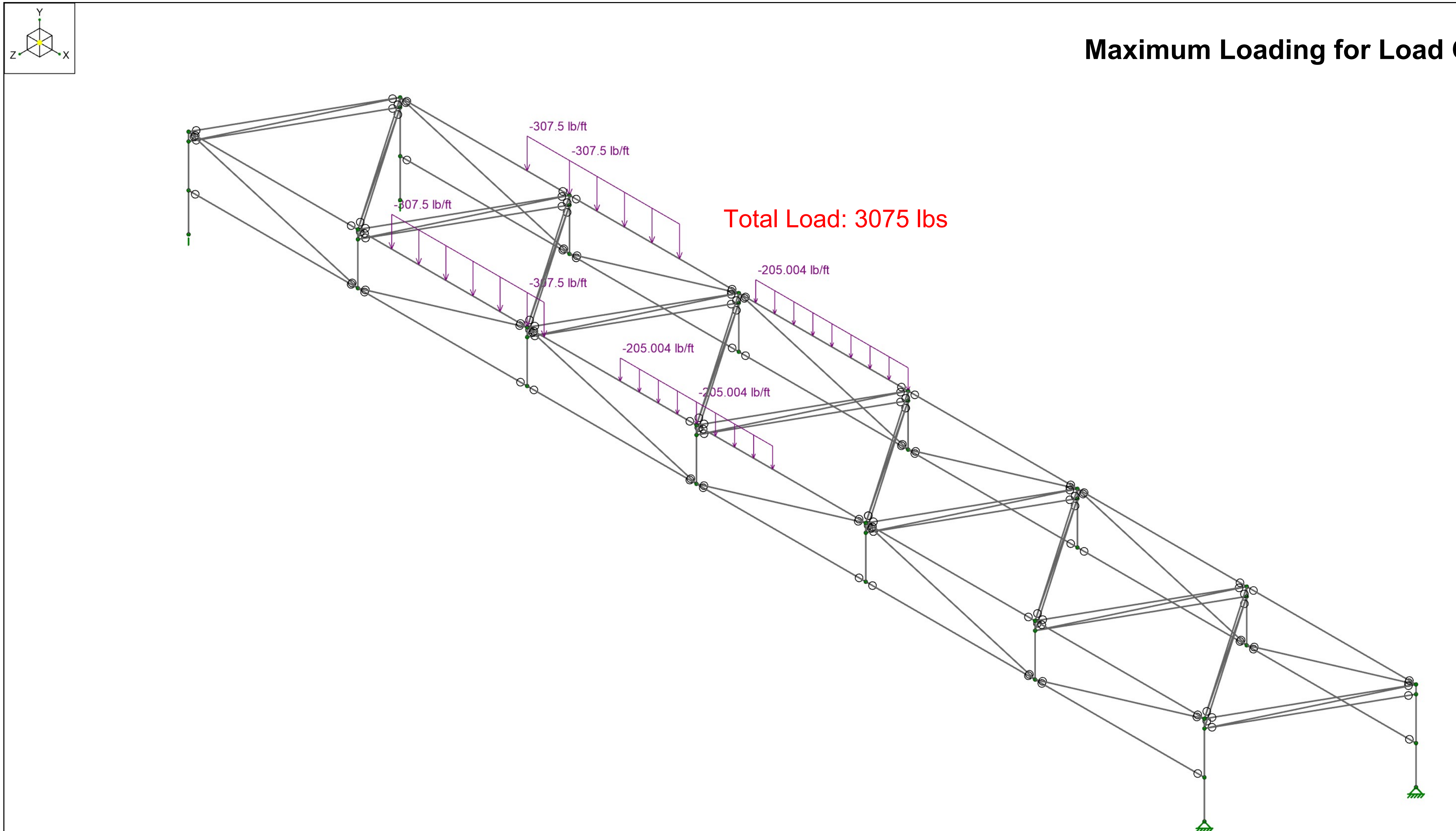
Results for LC 8, Case 5

		SK-9
		Mar 04, 2021
		Existing bridge loaded to failure.r3d

Load Case 6: Analysis of Connection Capacities versus Axial Loading

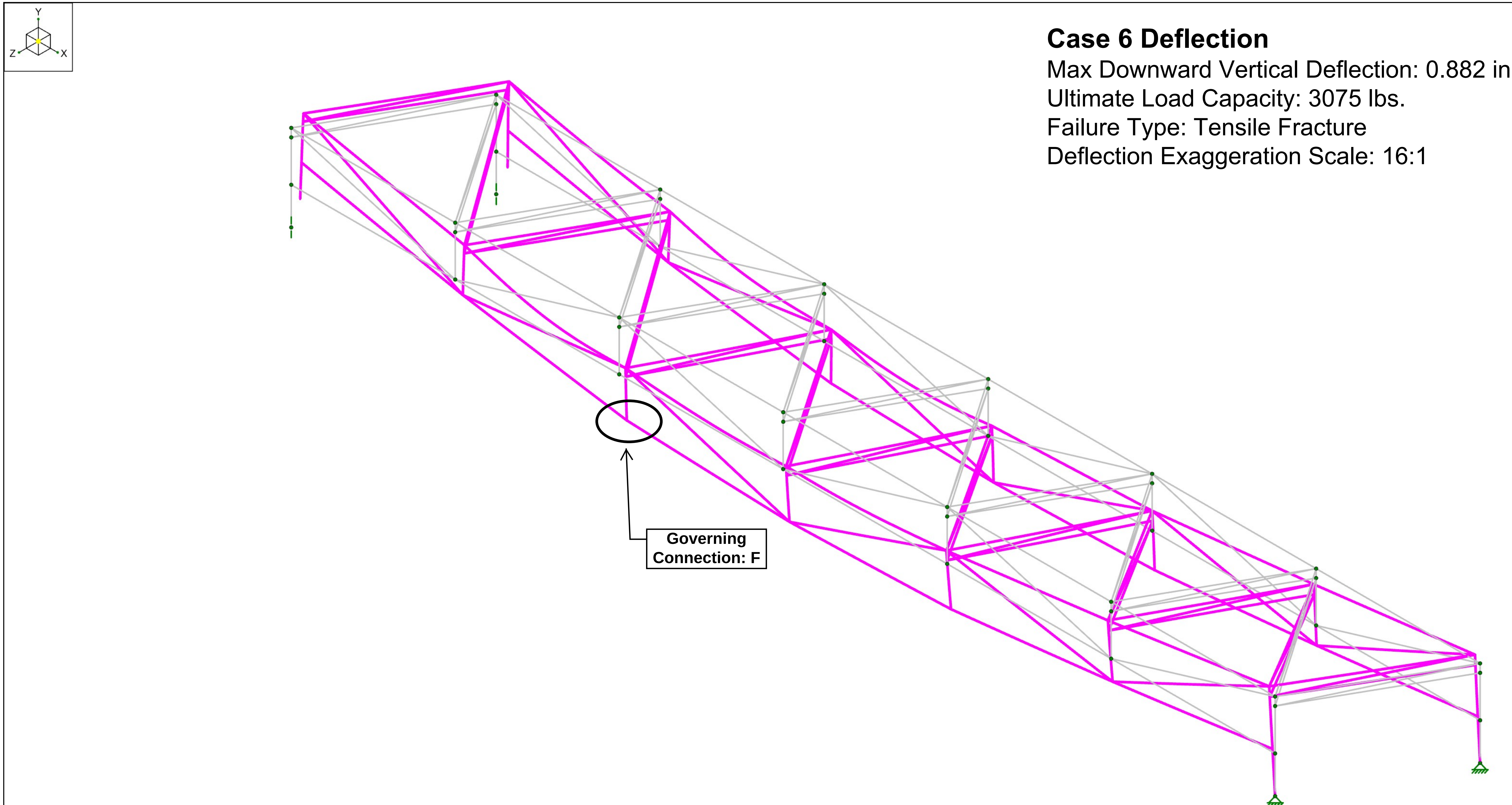
RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	Connection 1	Connection 2	Connection 1 Capacity, kips	Connection 2 Capacity, kips	Controlling Capacity	% Loaded
M56A	BC	3783.418	-3.78	B	C1	8.96	12.66	8.96	-42.23%
M58	BC	4787.659	-4.79	B	C1	8.96	12.66	8.96	-53.43%
M59A	BC	1642.104	-1.64	B	C1	8.96	12.66	8.96	-18.33%
M60A	AB	1618.082	-1.62	A1	B	8.96	8.96	8.96	-18.06%
M62A	EF	-5153.601	5.15	E2	F	8.96	5.625	5.625	91.62%
M63A	EF	-5130.056	5.13	E2	F	8.96	5.625	5.625	91.20%
M64A	EF	-3261.139	3.26	E2	F	8.96	5.625	5.625	57.98%
M65A	EF	-3225.652	3.23	E2	F	8.96	5.625	5.625	57.34%
M66A	AC	-8.062	0.01	D	E2	5.625	8.96	5.625	0.14%
M67A	AA	491.945	-0.49	A2	A2	8.96	8.96	8.96	-5.49%
M35	AA	1028.615	-1.03	A2	D	8.96	5.625	5.625	-18.29%
M33	AB	3361.154	-3.36	A1	B	8.96	8.96	8.96	-37.51%
M34	BC	3369.034	-3.37	B	C1	8.96	12.66	8.96	-37.60%
M35A	BC	5616.414	-5.62	B	C1	8.96	12.66	8.96	-62.68%
M36	BC	5639.332	-5.64	B	C1	8.96	12.66	8.96	-62.94%
M37	BC	2037.903	-2.04	B	C1	8.96	12.66	8.96	-22.74%
M38	AB	2050.405	-2.05	A1	B	8.96	8.96	8.96	-22.88%
M40	AC	-24.741	0.02	D	E2	5.625	8.96	5.625	0.44%
M41	EF	-5604.713	5.60	E2	F	8.96	5.625	5.625	99.64%
M42	EF	-5631.268	5.63	E2	F	8.96	5.625	5.625	100.11%
M43	EF	-3977.373	3.98	E2	F	8.96	5.625	5.625	70.71%
M44	EF	-3994.765	3.99	E2	F	8.96	5.625	5.625	71.02%
M45	AC	-3.333	0.00	D	E2	5.625	8.96	5.625	0.06%
M45A	AA	643.789	-0.64	A2	D	8.96	5.625	5.625	-11.45%
M56	AE	-3499.278	3.50	A2	E1	8.96	10.02	8.96	39.05%
M57A	EC	2309.698	-2.31	C2	E1	8.44	10.02	8.44	-27.37%
M58A	CE	-0.853	0.00	C2	E1	8.44	10.02	8.44	0.01%
M59	EC	-1744.424	1.74	C2	E1	8.44	10.02	8.44	20.67%
M60	CE	2040.047	-2.04	C2	E1	8.44	10.02	8.44	-24.17%
M61	AE	-2140.627	2.14	A2	E1	8.96	10.02	8.96	23.89%
M64	EC	1447.555	-1.45	C2	E1	8.44	10.02	8.44	-17.15%
M65	CE	334.559	-0.33	C2	E1	8.44	10.02	8.44	-3.96%
M66	EC	-1592.474	1.59	C2	E1	8.44	10.02	8.44	18.87%
M67	CE	1646.364	-1.65	C2	E1	8.44	10.02	8.44	-19.51%
M68	AE	-1685.551	1.69	A2	E1	8.96	10.02	8.96	18.81%
M61A	AC	-39.5	0.04	D	E2	5.625	8.96	5.625	0.70%
M67B	AE	-3891.979	3.89	A2	E1	8.96	10.02	8.96	43.44%
M41A	BE	343.936	-0.34	B	E2	8.96	8.96	8.96	-3.84%
M42A	CF	42.231	-0.04	C3	F	8.96	5.625	5.625	-0.75%
M43A	BE	520.849	-0.52	B	E2	8.96	8.96	8.96	-5.81%
M44A	CF	17.393	-0.02	C3	F	8.96	5.625	5.625	-0.31%
M45B	BE	24.881	-0.02	B	E2	8.96	8.96	8.96	-0.28%
M46	BE	13.749	-0.01	B	E2	8.96	8.96	8.96	-0.15%
M47	CF	16.853	-0.02	C3	F	8.96	5.625	5.625	-0.30%
M48	BE	372.884	-0.37	B	E2	8.96	8.96	8.96	-4.16%
M49	CF	33.964	-0.03	C3	F	8.96	5.625	5.625	-0.60%
M50	BE	709.773	-0.71	B	E2	8.96	8.96	8.96	-7.92%
M52A	AA	1164.924	-1.16	A2	D	8.96	5.625	5.625	-20.71%
M89A	BC	4820.941	-4.82	B	C1	8.96	12.66	8.96	-53.81%
M89B	AB	3767.289	-3.77	A1	B	8.96	8.96	8.96	-42.05%

Maximum Loading for Load Case 6



Loads: LC 9, Case 6

		SK-9
		Mar 04, 2021
		Existing bridge loaded to failure.r3d



Case 6 Deflection

Max Downward Vertical Deflection: 0.882 in
Ultimate Load Capacity: 3075 lbs.
Failure Type: Tensile Fracture
Deflection Exaggeration Scale: 16:1

Results for LC 9, Case 6

		SK-8
		Mar 04, 2021
		Existing bridge loaded to failure.r3d

Appendix C - Analysis of Updated Design

Contents:

- Calculations for Revision 1 Connection Capacities A - D
- Calculations for Revision 2 Connection Capacities A - D
- Excel Table Legend
- For Load Case 1 - 6:
  - Excel Determination of Failure Areas [REFER TO LEGEND]
  - RISA Loading Graphics for Predicted Max Loading
  - RISA Deflection Graphics
  - Excel Determination of Failure Areas [REFER TO LEGEND]

PROPRIETARY AND CONFIDENTIAL

		UNLESS OTHERWISE SPECIFIED:		NAME	DATE
		DIMENSIONS ARE IN INCHES TOLERANCES: FRACTIONAL± ANGULAR: MACH± BEND ± TWO PLACE DECIMAL ± THREE PLACE DECIMAL ±	DRAWN		
			CHECKED		
			ENG APPR.		
			MFG APPR.		
		INTERPRET GEOMETRIC TOLERANCING PER:	Q.A.		
		MATERIAL	COMMENTS:		
NEXT ASSY	USED ON	FINISH			
APPLICATION		DO NOT SCALE DRAWING			

TITLE:

SIZE	DWG. NO.	REV
B	Appendix C	
SCALE:	WEIGHT:	SHEET 1 OF 1

Connection A Revision 1 Capacity Calculations

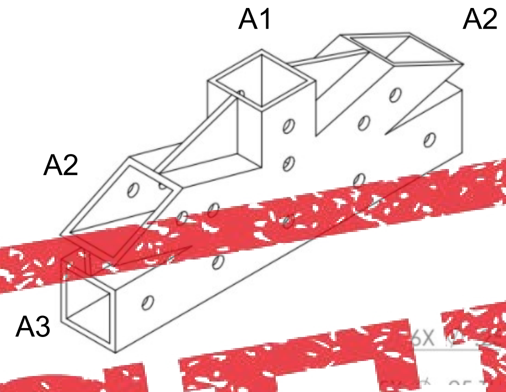
Calculations

Plate Properties (Table 1)		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength (Fy)	50	ksi
F_u	60	ksi

Tensile Stength (YLS) (Table 2)		
Phi_t*Pn	46.35	kip
Ag	1.03	in^2
Reduction Factor	0.9	
Tensile Stength (FLS)		
Phi_t*Pn	44.241	kip
An = Ae	0.983	in^2
Reduction Factor	0.75	

Block Shear Stength at A1		
Trial 1		
Phi*Rn	20.60	kip
Anv	0.2775	in^2
Ubs	0.5	
Ant	0.5825	in^2
Reduction Factor	0.75	
Trial 2		
Phi*Rn	56.87	kip
Anv	1.864	in^2
Ubs	0.5	
Ant	0.291	in^2
Reduction Factor	0.75	

Block Shear Stength at A2		
Trial 1		
Phi*Rn	14.49	kip
Anv	0.3081	in^2
Ubs	0.5	
Ant	0.2744	in^2
Reduction Factor	0.75	
Trial 2		
Phi*Rn	11.672	kip
Anv	0.206	in^2
Ubs	0.5	
Ant	0.271	in^2
Reduction Factor	0.75	



REVISED

Results

Overall Strengths of Connection B (kip)	
Tensile Strength (YLS)	46.35
Tensile Strength (FLS)	44.24
Block Shear Strength at A1	20.60
Block Shear Strength at A2	11.67

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Block Shear Phi*Rn	J4.3	16.1-138

Connection B Revision 1 Capacity Calculations

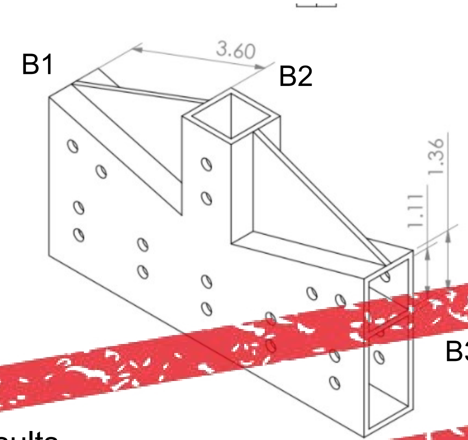
Calculations

Plate Properties (Table 1)		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength	50	ksi
F_u	60	ksi

Tensile Stength (YLS) (Table 2)		
Phi_t*Pn	57.516	kip
Ag	1.278	in^2
Reduction Factor	0.9	
Tensile Stength (FLS)		
Phi_t*Pn	110.813	kip
An = Ae	1.231	in^2
Reduction Factor	0.75	
Number of Plates	2	

Block Shear Stength at B1		
Trial 1		
Phi*Rn	18.45	kip
Anv	0.4219	in^2
Ubs	0.5	
Ant	0.31375	in^2
Reduction Factor	0.75	
Trial 2		
Phi*Rn	10.86	kip
Anv	0.1406	in^2
Ubs	0.5	
Ant	0.31375	in^2
Reduction Factor	0.75	

Block Shear Stength at B2		
Trial 1		
Phi*Rn	14.29875	kip
Anv	0.268125	in^2
Ubs	0.5	
Ant	0.31375	in^2
Reduction Factor	0.75	
Trial 2		
Phi*Rn	13.3875	kip
Anv	0.234375	in^2
Ubs	0.5	
Ant	0.31375	in^2
Reduction Factor	0.75	



Results

Overall Strengths of Connection A (kip)	
Tensile Strength (YLS)	57.52
Tensile Strength (FLS)	110.81
Block Shear Strength at B1	10.86
Block Shear Strength at B2	13.39

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Block Shear Phi*Rn	J4.3	16.1-138

REVISED

Connection A Revision 2 Capacity Calculations

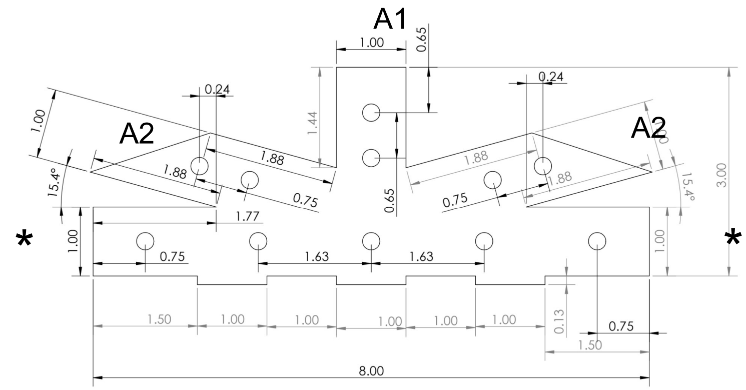
Calculations

Plate Properties (Table 1)		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength (Fy)	50	ksi
F_u	60	ksi

Tensile Stength (YLS) (Table 2)		
Phi_t*Pn	46.35	kip
Ag	1.03	in^2
Reduction Factor	0.9	
Tensile Stength (FLS)		
Phi_t*Pn	44.241	kip
An = Ae	0.983	in^2
Reduction Factor	0.75	

Block Shear Stength at A1		
Trial 1		
Phi*Rn	7.54	kip
Anv	0.208	in^2
Ubs	0.5	
Ant	0.0859	in^2
Reduction Factor	0.75	
Trial 2		
Phi*Rn	3.87	kip
Anv	0.000	in^2
Ubs	0.5	
Ant	0.172	in^2
Reduction Factor	0.75	

Block Shear Stength at A2		
Trial 1		
Phi*Rn	8.33	kip
Anv	0.2578	in^2
Ubs	0.5	
Ant	0.0609	in^2
Reduction Factor	0.75	
Trial 2		
Phi*Rn	3.867	kip
Anv	0.000	in^2
Ubs	0.5	
Ant	0.172	in^2
Reduction Factor	0.75	



Results

Overall Strengths of Connection A (kip)	
Tensile Strength (YLS)	46.35
Tensile Strength (FLS)	44.24
Block Shear Strength at A1	7.54
Block Shear Strength at A2	3.87

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Block Shear Phi*Rn	J4.3	16.1-138

*Not a failure path due to continuous chord member

Connection B Revision 2 Capacity Calculations

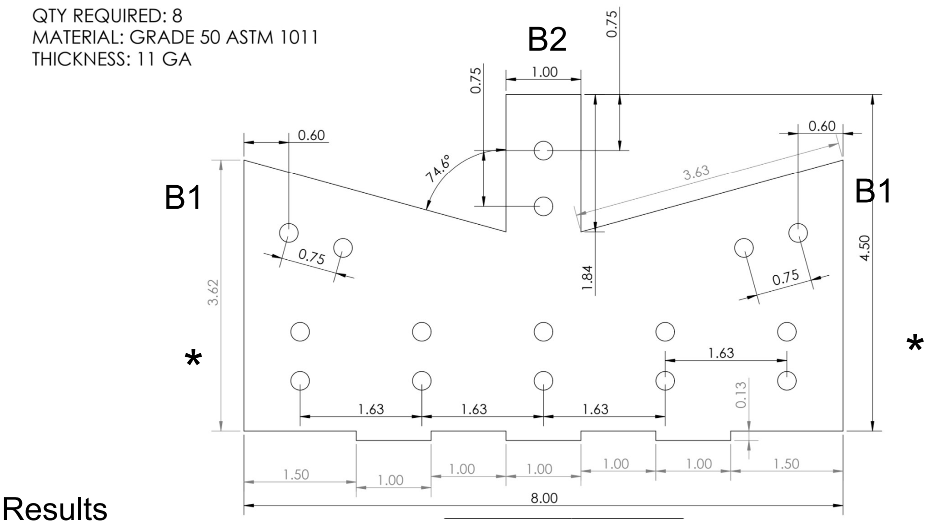
Calculations

Plate Properties (Table 1)		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength	50	ksi
F_u	60	ksi

Tensile Stength (YLS) (Table 2)		
Phi_t*Pn	57.516	kip
Ag	1.278	in^2
Reduction Factor	0.9	
Tensile Stength (FLS)		
Phi_t*Pn	110.813	kip
An = Ae	1.231	in^2
Reduction Factor	0.75	
Number of Plates	2	

Block Shear Stength at B1		
Trial 1		
Phi*Rn	9.46	kip
Anv	0.227	in^2
Ubs	0.5	
Ant	0.148	in^2
Reduction Factor	0.75	

Block Shear Stength at B2		
Trial 1		
Phi*Rn	8.895	kip
Anv	0.258	in^2
Ubs	0.5	
Ant	0.086	in^2
Reduction Factor	0.75	
Trial 2		
Phi*Rn	3.867	kip
Anv	0	in^2
Ubs	0.5	
Ant	0.171875	in^2
Reduction Factor	0.75	



Results

Overall Strengths of Connection B (kip)	
Tensile Strength (YLS)	57.52
Tensile Strength (FLS)	110.81
Block Shear Strength at B1	9.46
Block Shear Strength at B2	3.87

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Block Shear Phi*Rn	J4.3	16.1-138

*Not a failure path due to continuous chord member

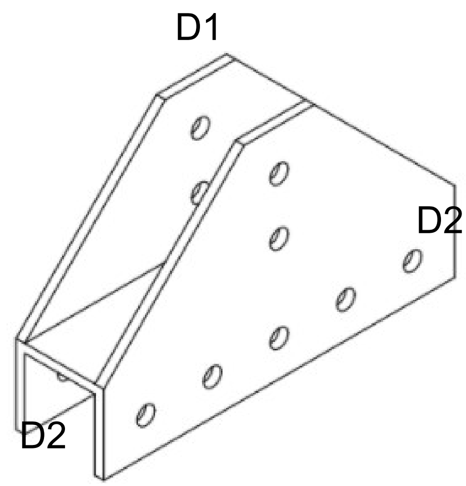
Connection D Revision 1 Capacity Calculations

Calculations

Plate Properties (Table 1)		
Steel Type	ASTM 1011	
Thickness	0.125	in
Yield Strength	50	ksi
F_u	60	ksi

Tensile Stength (YLS) (Table 2)		
Phi_t*Pn	42.750	kip
Ag	0.950	in^2
Reduction Factor	0.9	
Tensile Stength (FLS)		
Phi_t*Pn	40.500	kip
An = Ae	0.9	in^2
Reduction Factor	0.75	
Number of Plates	2	

Block Shear Stength at D2		
Trial 1		
Phi*Rn	21.94	kip
Anv	0.234	in^2
Ubs	0.5	
Ant	0.45	in^2
Reduction Factor	0.75	



Results

References From AISC Manual		
Equation	**Section**	**Page**
YLS Phi_t*Pn	D2-a	16.1-28
FLS Phi_t*Pn	D2-b	16.1-28
Block Shear Phi*Rn	J4.3	16.1-138

Determination of Failure Locations for the New Bridge Design

Max Internal Axial Forces in Web Members
Versus
Capacity of Corresponding Connections

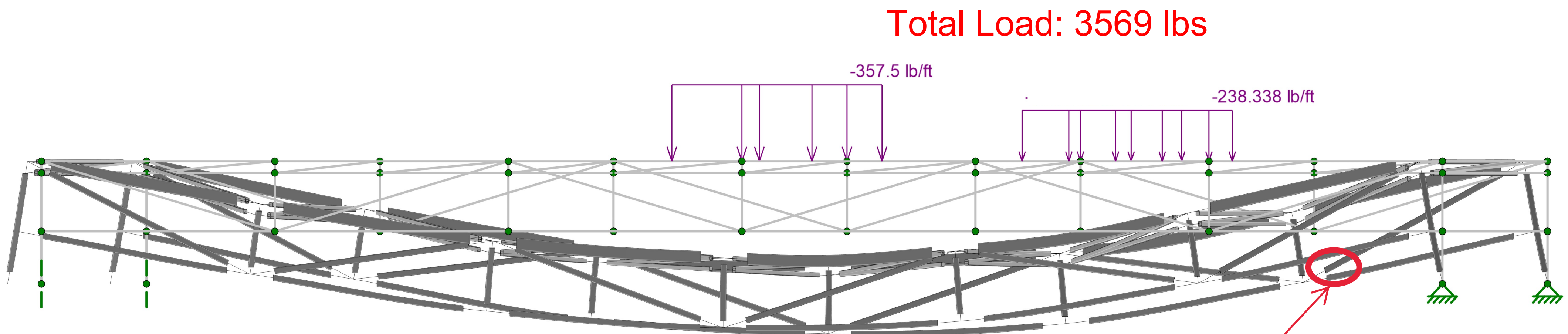
Legend	
Axial [lb] Column Color Scheme [+] : Compression *(this column only)* [-] : Tension *(this column only)*	Bottom 50% of Force Distribution Values
	Median Values [negligible axial force]
	Top 50% of Force Distribution Values
Demand vs Capacity Column Color Scheme Demonstrates % Loaded for each connection in terms of capacity	Top 50% of Values [Farthest from Failure]
	Median Values [Not Predicted to Fail]
	Bottom 50% of Values [Closest to Failure]
Plan Set ID Color Scheme *See Existing Plan Set in Appendix A for reference*	Top Chord Members
	Bottom Chord Members
	Vertical Columns at Bridge Ends
	Web Members

Load Case 1: Analysis of Connection Capacities versus Axial Loading

RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	New Connection i	New Connection j	Capacity i, kips	Capacity j, kips	Controlling Capacity, kips	% Loaded
M56A	BC	7640.563	-2.76	C1	B1	NA	9.46	9.46	-29.19%
M58	BC	19598.21	-7.08	C1	B1	NA	9.46	9.46	-74.87%
M59A	BC	9503.822	-3.43	C1	B1	NA	9.46	9.46	-36.31%
M60A	AB	9228.4	-3.34	B3	C1	NA	NA	NA	Not a Failure Path
M62A	EF	-15261.8	5.52	A1	D2	3.87	NA	3.87	142.52%
M63A	EF	-15647.7	5.66	A1	D2	3.87	NA	3.87	146.13%
M64A	EF	-17236.2	6.23	A1	D2	3.87	NA	3.87	160.96%
M65A	EF	-17011.4	6.15	A1	D2	3.87	NA	3.87	158.86%
M66A	AC	-401.658	0.15	D1	A1	21.94	3.87	3.87	3.75%
M67A	AA	2791.239	-1.01	B1	B1	9.46	9.46	9.46	-10.66%
M35	AA	1839.792	-0.66	B1	D1	9.46	21.94	9.46	-7.03%
M33	AB	6140.547	-2.22	B3	C1	NA	NA	NA	Not a Failure Path
M34	BC	6338.999	-2.29	C1	B1	NA	9.46	9.46	-24.22%
M35A	BC	18891.63	-6.83	C1	B1	NA	9.46	9.46	-72.17%
M36	BC	19349.71	-6.99	C1	B1	NA	9.46	9.46	-73.92%
M37	BC	11038.09	-3.99	C1	B1	NA	9.46	9.46	-42.17%
M38	AB	10596.92	-3.83	B3	C1	NA	NA	NA	Not a Failure Path
M40	AC	-220.841	0.08	D1	A1	21.94	3.87	3.87	2.06%
M41	EF	-12436.2	4.49	A1	D2	3.87	NA	3.87	116.14%
M42	EF	-12765.6	4.61	A1	D2	3.87	NA	3.87	119.21%
M43	EF	-16960.5	6.13	A1	D2	3.87	NA	3.87	158.39%
M44	EF	-16836.1	6.08	A1	D2	3.87	NA	3.87	157.22%
M45	AC	-329.686	0.12	D1	A1	21.94	3.87	3.87	3.08%
M45A	AA	3089.543	-1.12	B1	D1	9.46	21.94	9.46	-11.80%
M56	AE	-6252.26	2.26	B1	A2	9.46	3.87	3.87	58.39%
M57A	EC	6267.841	-2.27	B2	A2	3.87	3.87	3.87	-58.53%
M58A	CE	-6420.91	2.32	B2	A2	3.87	3.87	3.87	59.96%
M59	EC	-2377.48	0.86	B2	A2	3.87	3.87	3.87	22.20%
M60	CE	6196.073	-2.24	B2	A2	3.87	3.87	3.87	-57.86%
M61	AE	-10705.8	3.87	B1	A2	9.46	3.87	3.87	99.98%
M64	EC	7917.1	-2.86	B2	A2	3.87	3.87	3.87	-73.93%
M65	CE	-4909.7	1.77	B2	A2	3.87	3.87	3.87	45.85%
M66	EC	-2646.33	0.96	B2	A2	3.87	3.87	3.87	24.71%
M67	CE	7591.58	-2.74	B2	A2	3.87	3.87	3.87	-70.89%
M68	AE	-9379.01	3.39	B1	A2	9.46	3.87	3.87	87.59%
M61A	AC	-255.632	0.09	D1	A1	21.94	3.87	3.87	2.39%
M67B	AE	-7586.7	2.74	B1	A2	9.46	3.87	3.87	70.85%
M41A	BE	3.896	0.00	C1	A1	NA	3.87	3.87	-0.04%
M42A	CF	229.162	-0.08	B3	D2	NA	NA	NA	Not a Failure Path
M43A	BE	2823.628	-1.02	C1	A1	NA	3.87	3.87	-26.37%
M44A	CF	429.379	-0.16	B3	D2	NA	NA	NA	Not a Failure Path
M45B	BE	1346.429	-0.49	C1	A1	NA	3.87	3.87	-12.57%
M46	BE	533.184	-0.19	C1	A1	NA	3.87	3.87	-4.98%
M47	CF	464.757	-0.17	B3	D2	NA	NA	NA	Not a Failure Path
M48	BE	2451.127	-0.89	C1	A1	NA	3.87	3.87	-22.89%
M49	CF	393.321	-0.14	B3	D2	NA	NA	NA	Not a Failure Path
M50	BE	-154.236	0.06	C1	A1	NA	3.87	3.87	1.44%
M52A	AA	2271.742	-0.82	B1	D1	9.46	21.94	9.46	-8.68%
M89A	BC	20277.81	-7.33	C1	B1	NA	9.46	9.46	-77.47%
M89B	AB	7484.458	-2.70	B3	C1	NA	NA	NA	Not a Failure Path

Case 1 Deflection - New Bridge

Max Downward Vertical Deflection: 1.097 in
Ultimate Load Capacity: 3569
Failure Type: Block Shear
Deflection Exaggeration Scale: 16:1



Loads: LC 4, Case 1
Results for LC 4, Case 1

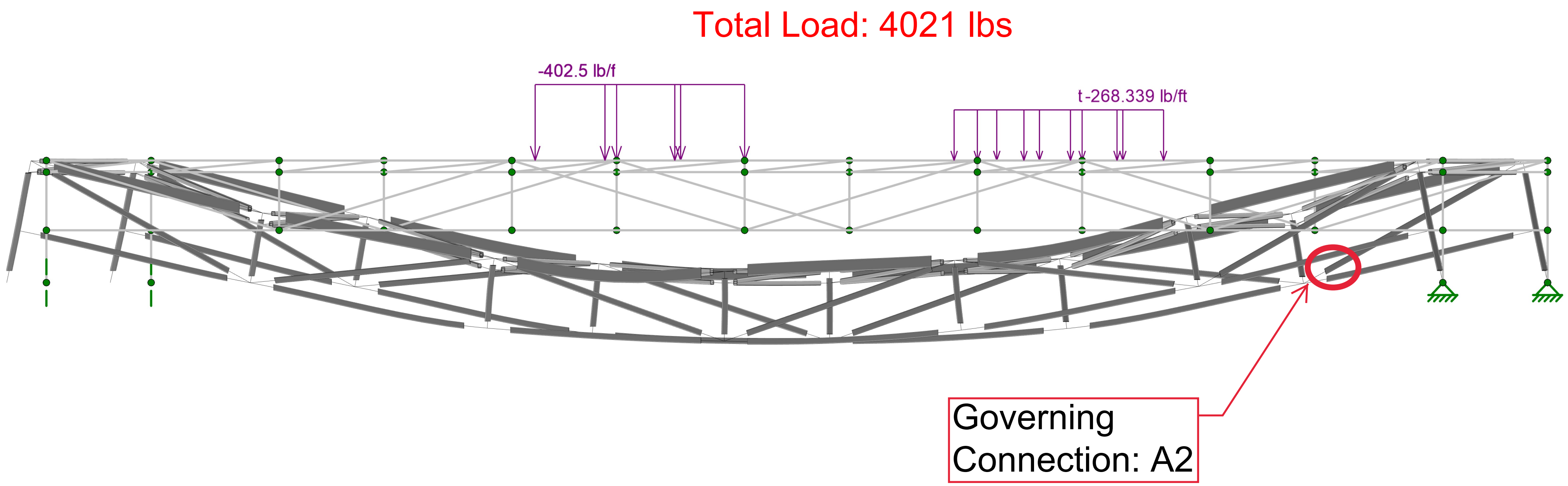
		SK-14
		Apr 07, 2021
		New Bridge Loaded to Failure.r3d

Load Case 2: Analysis of Connection Capacities versus Axial Loading

RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	New Connection i	New Connection j	Capacity i, kips	Capacity j, kips	Controlling Capacity, kips	% Loaded
M56A	BC	10382.172	-3.75	C1	B1	NA	9.46	9.46	-39.66%
M58	BC	19829.297	-7.17	C1	B1	NA	9.46	9.46	-75.75%
M59A	BC	8875.529	-3.21	C1	B1	NA	9.46	9.46	-33.91%
M60A	AB	8655.267	-3.13	B3	C1	NA	NA	NA	Not a Failure Path
M62A	EF	-19774.659	7.15	A1	D2	3.87	NA	3.87	184.67%
M63A	EF	-20035.687	7.24	A1	D2	3.87	NA	3.87	187.10%
M64A	EF	-17781.846	6.43	A1	D2	3.87	NA	3.87	166.06%
M65A	EF	-17351.957	6.27	A1	D2	3.87	NA	3.87	162.04%
M66A	AC	-408.355	0.15	D1	A1	21.94	3.87	3.87	3.81%
M67A	AA	2642.651	-0.96	B1	B1	9.46	9.46	9.46	-10.10%
M35	AA	2490.437	-0.90	B1	D1	9.46	21.94	9.46	-9.51%
M33	AB	8247.742	-2.98	B3	C1	NA	NA	NA	Not a Failure Path
M34	BC	8426.662	-3.05	C1	B1	NA	9.46	9.46	-32.19%
M35A	BC	21506.963	-7.77	C1	B1	NA	9.46	9.46	-82.16%
M36	BC	20990.469	-7.59	C1	B1	NA	9.46	9.46	-80.19%
M37	BC	11025.549	-3.98	C1	B1	NA	9.46	9.46	-42.12%
M38	AB	10566.291	-3.82	B3	C1	NA	NA	NA	Not a Failure Path
M40	AC	-359.327	0.13	D1	A1	21.94	3.87	3.87	3.36%
M41	EF	-17383.363	6.28	A1	D2	3.87	NA	3.87	162.33%
M42	EF	-18039.652	6.52	A1	D2	3.87	NA	3.87	168.46%
M43	EF	-18110.812	6.55	A1	D2	3.87	NA	3.87	169.13%
M44	EF	-18106.752	6.54	A1	D2	3.87	NA	3.87	169.09%
M45	AC	-320.025	0.12	D1	A1	21.94	3.87	3.87	2.99%
M45A	AA	3075.064	-1.11	B1	D1	9.46	21.94	9.46	-11.75%
M56	AE	-8359.505	3.02	B1	A2	9.46	3.87	3.87	78.07%
M57A	EC	9146.647	-3.31	B2	A2	3.87	3.87	3.87	-85.42%
M58A	CE	-3691.833	1.33	B2	A2	3.87	3.87	3.87	34.48%
M59	EC	-2971.025	1.07	B2	A2	3.87	3.87	3.87	27.74%
M60	CE	7485.331	-2.71	B2	A2	3.87	3.87	3.87	-69.90%
M61	AE	-10709.21	3.87	B1	A2	9.46	3.87	3.87	100.01%
M64	EC	9809.075	-3.54	B2	A2	3.87	3.87	3.87	-91.60%
M65	CE	-198.249	0.07	B2	A2	3.87	3.87	3.87	1.85%
M66	EC	-2264.359	0.82	B2	A2	3.87	3.87	3.87	21.15%
M67	CE	8602.28	-3.11	B2	A2	3.87	3.87	3.87	-80.33%
M68	AE	-8768.909	3.17	B1	A2	9.46	3.87	3.87	81.89%
M61A	AC	-365.285	0.13	D1	A1	21.94	3.87	3.87	3.41%
M67B	AE	-10168.184	3.67	B1	A2	9.46	3.87	3.87	94.96%
M41A	BE	-237.207	0.09	C1	A1	NA	3.87	3.87	2.22%
M42A	CF	546.481	-0.20	B3	D2	NA	NA	NA	Not a Failure Path
M43A	BE	2207.568	-0.80	C1	A1	NA	3.87	3.87	-20.62%
M44A	CF	447.131	-0.16	B3	D2	NA	NA	NA	Not a Failure Path
M45B	BE	965.709	-0.35	C1	A1	NA	3.87	3.87	-9.02%
M46	BE	66.984	-0.02	C1	A1	NA	3.87	3.87	-0.63%
M47	CF	493.413	-0.18	B3	D2	NA	NA	NA	Not a Failure Path
M48	BE	952.992	-0.34	C1	A1	NA	3.87	3.87	-8.90%
M49	CF	604.64	-0.22	B3	D2	NA	NA	NA	Not a Failure Path
M50	BE	102.992	-0.04	C1	A1	NA	3.87	3.87	-0.96%
M52A	AA	2938.051	-1.06	B1	D1	9.46	21.94	9.46	-11.22%
M89A	BC	20234.15	-7.31	C1	B1	NA	9.46	9.46	-77.30%
M89B	AB	10062.241	-3.64	B3	C1	NA	NA	NA	Not a Failure Path

Case 2 Deflection - New Bridge

Max Downward Vertical Deflection: 1.193 in
Ultimate Load Capacity: 4021 lbs
Failure Type: Block Shear
Deflection Exaggeration Scale: 16:1



Loads: LC 5, Case 2
Results for LC 5, Case 2

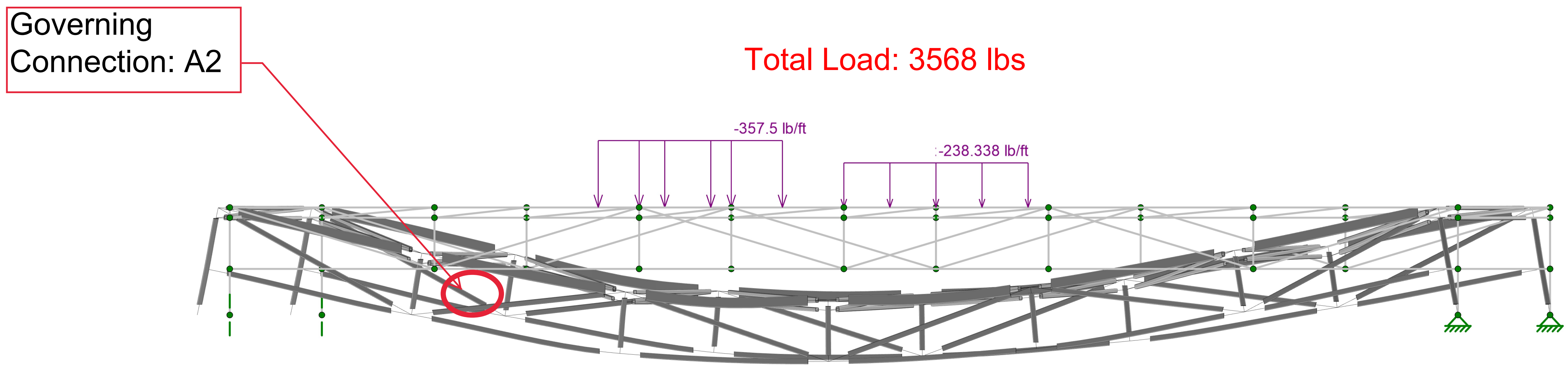
		SK-15
		Apr 07, 2021
		New Bridge Loaded to Failure.r3d

Load Case 3: Analysis of Connection Capacities versus Axial Loading

RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	New Connection i	New Connection j	Capacity i, kips	Capacity j, kips	Controlling Capacity, kips	% Loaded
M56A	BC	11080.664	-4.01	C1	B1	NA	9.46	9.46	-42.42%
M58	BC	18763.116	-6.79	C1	B1	NA	9.46	9.46	-71.83%
M59A	BC	6391.095	-2.31	C1	B1	NA	9.46	9.46	-24.47%
M60A	AB	6193.259	-2.24	B3	C1	NA	NA	NA	Not a Failure Path
M62A	EF	-18806.006	6.81	A1	D2	3.87	NA	3.87	175.97%
M63A	EF	-18834.381	6.82	A1	D2	3.87	NA	3.87	176.24%
M64A	EF	-12960.32	4.69	A1	D2	3.87	NA	3.87	121.27%
M65A	EF	-12610.764	4.57	A1	D2	3.87	NA	3.87	118.00%
M66A	AC	-266.735	0.10	D1	A1	21.94	3.87	3.87	2.50%
M67A	AA	1860.811	-0.67	B1	B1	9.46	9.46	9.46	-7.12%
M35	AA	2666.214	-0.97	B1	D1	9.46	21.94	9.46	-10.21%
M33	AB	8881.984	-3.22	B3	C1	NA	NA	NA	Not a Failure Path
M34	BC	9053.01	-3.28	C1	B1	NA	9.46	9.46	-34.66%
M35A	BC	20218.901	-7.32	C1	B1	NA	9.46	9.46	-77.40%
M36	BC	20388.677	-7.38	C1	B1	NA	9.46	9.46	-78.05%
M37	BC	7758.402	-2.81	C1	B1	NA	9.46	9.46	-29.70%
M38	AB	7592.228	-2.75	B3	C1	NA	NA	NA	Not a Failure Path
M40	AC	-385.664	0.14	D1	A1	21.94	3.87	3.87	3.61%
M41	EF	-18292.617	6.62	A1	D2	3.87	NA	3.87	171.17%
M42	EF	-18859.22	6.83	A1	D2	3.87	NA	3.87	176.47%
M43	EF	-15775.891	5.71	A1	D2	3.87	NA	3.87	147.62%
M44	EF	-15400.594	5.58	A1	D2	3.87	NA	3.87	144.11%
M45	AC	-218.11	0.08	D1	A1	21.94	3.87	3.87	2.04%
M45A	AA	2327.425	-0.84	B1	D1	9.46	21.94	9.46	-8.91%
M56	AE	-9008.555	3.26	B1	A2	9.46	3.87	3.87	84.30%
M57A	EC	9400.389	-3.40	B2	A2	3.87	3.87	3.87	-87.96%
M58A	CE	-1560.824	0.57	B2	A2	3.87	3.87	3.87	14.61%
M59	EC	-4857.785	1.76	B2	A2	3.87	3.87	3.87	45.46%
M60	CE	7964.047	-2.88	B2	A2	3.87	3.87	3.87	-74.52%
M61	AE	-7716.514	2.79	B1	A2	9.46	3.87	3.87	72.21%
M64	EC	8163.866	-2.96	B2	A2	3.87	3.87	3.87	-76.39%
M65	CE	-158.482	0.06	B2	A2	3.87	3.87	3.87	1.48%
M66	EC	-6109.675	2.21	B2	A2	3.87	3.87	3.87	57.17%
M67	CE	6364.826	-2.30	B2	A2	3.87	3.87	3.87	-59.56%
M68	AE	-6284.195	2.28	B1	A2	9.46	3.87	3.87	58.80%
M61A	AC	-358.111	0.13	D1	A1	21.94	3.87	3.87	3.35%
M67B	AE	-10711.498	3.88	B1	A2	9.46	3.87	3.87	100.23%
M41A	BE	-115.839	0.04	C1	A1	NA	3.87	3.87	1.08%
M42A	CF	573.021	-0.21	B3	D2	NA	NA	NA	Not a Failure Path
M43A	BE	2198.534	-0.80	C1	A1	NA	3.87	3.87	-20.57%
M44A	CF	306.936	-0.11	B3	D2	NA	NA	NA	Not a Failure Path
M45B	BE	-92.88	0.03	C1	A1	NA	3.87	3.87	0.87%
M46	BE	-0.535	0.00	C1	A1	NA	3.87	3.87	0.01%
M47	CF	251.587	-0.09	B3	D2	NA	NA	NA	Not a Failure Path
M48	BE	2063.062	-0.75	C1	A1	NA	3.87	3.87	-19.30%
M49	CF	590.244	-0.21	B3	D2	NA	NA	NA	Not a Failure Path
M50	BE	771.476	-0.28	C1	A1	NA	3.87	3.87	-7.22%
M52A	AA	2992.01	-1.08	B1	D1	9.46	21.94	9.46	-11.45%
M89A	BC	19059.029	-6.90	C1	B1	NA	9.46	9.46	-72.96%
M89B	AB	10599.53	-3.84	B3	C1	NA	NA	NA	Not a Failure Path

Case 3 Deflection - New Bridge

Max Downward Vertical Deflection: 1.131 in
Ultimate Load Capacity: 3568
Failure Type: Block Shear
Deflection Exaggeration Scale: 16:1



Loads: LC 6, Case 3
Results for LC 6, Case 3

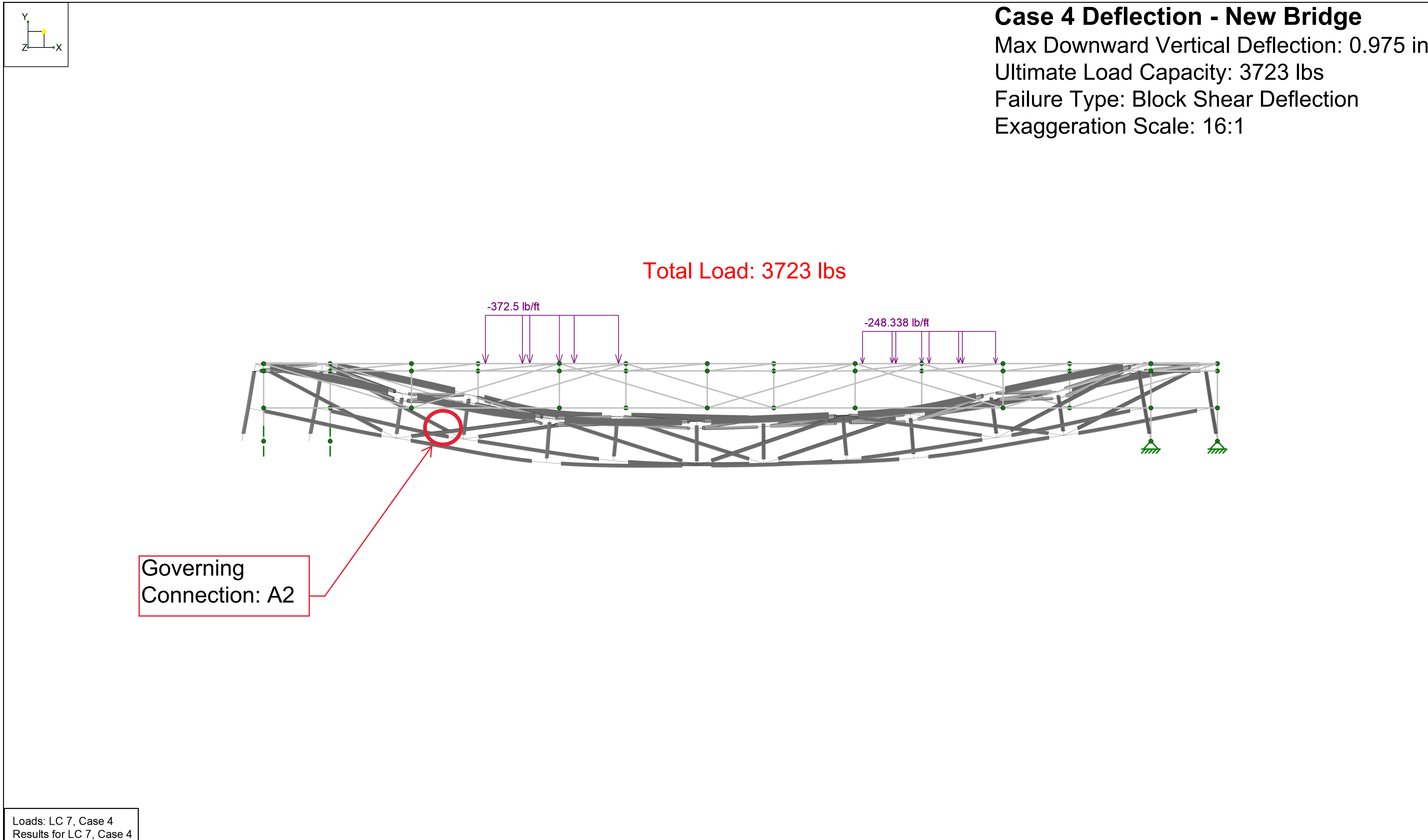
	SK-16
	Apr 07, 2021
	New Bridge Loaded to Failure.r3d

Load Case 4: Analysis of Connection Capacities versus Axial Loading

RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	New Connection i	New Connection j	Capacity i, kips	Capacity j, kips	Controlling Capacity, kips	% Loaded
M56A	BC	11073.776	-4.00	C1	B1	NA	9.46	9.46	-42.32%
M58	BC	14768.874	-5.34	C1	B1	NA	9.46	9.46	-56.44%
M59A	BC	7487.285	-2.71	C1	B1	NA	9.46	9.46	-28.61%
M60A	AB	7223.306	-2.61	B3	C1	NA	NA	NA	Not a Failure Path
M62A	EF	-15425.918	5.58	A1	D2	3.87	NA	3.87	144.09%
M63A	EF	-15198.681	5.49	A1	D2	3.87	NA	3.87	141.97%
M64A	EF	-13932.975	5.04	A1	D2	3.87	NA	3.87	130.15%
M65A	EF	-13767.685	4.98	A1	D2	3.87	NA	3.87	128.61%
M66A	AC	-275.487	0.10	D1	A1	21.94	3.87	3.87	2.57%
M67A	AA	2163.829	-0.78	B1	B1	9.46	9.46	9.46	-8.27%
M35	AA	2566.812	-0.93	B1	D1	9.46	21.94	9.46	-9.81%
M33	AB	8730.503	-3.16	B3	C1	NA	NA	NA	Not a Failure Path
M34	BC	9018.139	-3.26	C1	B1	NA	9.46	9.46	-34.46%
M35A	BC	15553.365	-5.62	C1	B1	NA	9.46	9.46	-59.43%
M36	BC	15577.78	-5.63	C1	B1	NA	9.46	9.46	-59.53%
M37	BC	9317.924	-3.37	C1	B1	NA	9.46	9.46	-35.61%
M38	AB	8945.24	-3.23	B3	C1	NA	NA	NA	Not a Failure Path
M40	AC	-321.719	0.12	D1	A1	21.94	3.87	3.87	3.01%
M41	EF	-16345.239	5.91	A1	D2	3.87	NA	3.87	152.68%
M42	EF	-16490.853	5.96	A1	D2	3.87	NA	3.87	154.04%
M43	EF	-13881.825	5.02	A1	D2	3.87	NA	3.87	129.67%
M44	EF	-13999.104	5.06	A1	D2	3.87	NA	3.87	130.77%
M45	AC	-243.208	0.09	D1	A1	21.94	3.87	3.87	2.27%
M45A	AA	2620.163	-0.95	B1	D1	9.46	21.94	9.46	-10.01%
M56	AE	-8900.962	3.22	B1	A2	9.46	3.87	3.87	83.14%
M57A	EC	7462.92	-2.70	B2	A2	3.87	3.87	3.87	-69.71%
M58A	CE	881.22	-0.32	B2	A2	3.87	3.87	3.87	-8.23%
M59	EC	-1776.842	0.64	B2	A2	3.87	3.87	3.87	16.60%
M60	CE	4935.156	-1.78	B2	A2	3.87	3.87	3.87	-46.10%
M61	AE	-9105.957	3.29	B1	A2	9.46	3.87	3.87	85.06%
M64	EC	4644.272	-1.68	B2	A2	3.87	3.87	3.87	-43.38%
M65	CE	388.99	-0.14	B2	A2	3.87	3.87	3.87	-3.63%
M66	EC	-924.696	0.33	B2	A2	3.87	3.87	3.87	8.64%
M67	CE	6415.418	-2.32	B2	A2	3.87	3.87	3.87	-59.93%
M68	AE	-7357.573	2.66	B1	A2	9.46	3.87	3.87	68.73%
M61A	AC	-327.571	0.12	D1	A1	21.94	3.87	3.87	3.06%
M67B	AE	-10705.939	3.87	B1	A2	9.46	3.87	3.87	100.01%
M41A	BE	454.703	-0.16	C1	A1	NA	3.87	3.87	-4.25%
M42A	CF	473.268	-0.17	B3	D2	NA	NA	NA	Not a Failure Path
M43A	BE	373.378	-0.13	C1	A1	NA	3.87	3.87	-3.49%
M44A	CF	301.388	-0.11	B3	D2	NA	NA	NA	Not a Failure Path
M45B	BE	1218.175	-0.44	C1	A1	NA	3.87	3.87	-11.38%
M46	BE	299.284	-0.11	C1	A1	NA	3.87	3.87	-2.80%
M47	CF	355.645	-0.13	B3	D2	NA	NA	NA	Not a Failure Path
M48	BE	257.762	-0.09	C1	A1	NA	3.87	3.87	-2.41%
M49	CF	361.488	-0.13	B3	D2	NA	NA	NA	Not a Failure Path
M50	BE	1803.337	-0.65	C1	A1	NA	3.87	3.87	-16.85%
M52A	AA	2953.367	-1.07	B1	D1	9.46	21.94	9.46	-11.29%
M89A	BC	14862.785	-5.37	C1	B1	NA	9.46	9.46	-56.80%
M89B	AB	10558.525	-3.82	B3	C1	NA	NA	NA	Not a Failure Path

Case 4 Deflection - New Bridge

Max Downward Vertical Deflection: 0.975 in
Ultimate Load Capacity: 3723 lbs
Failure Type: Block Shear Deflection
Exaggeration Scale: 16:1



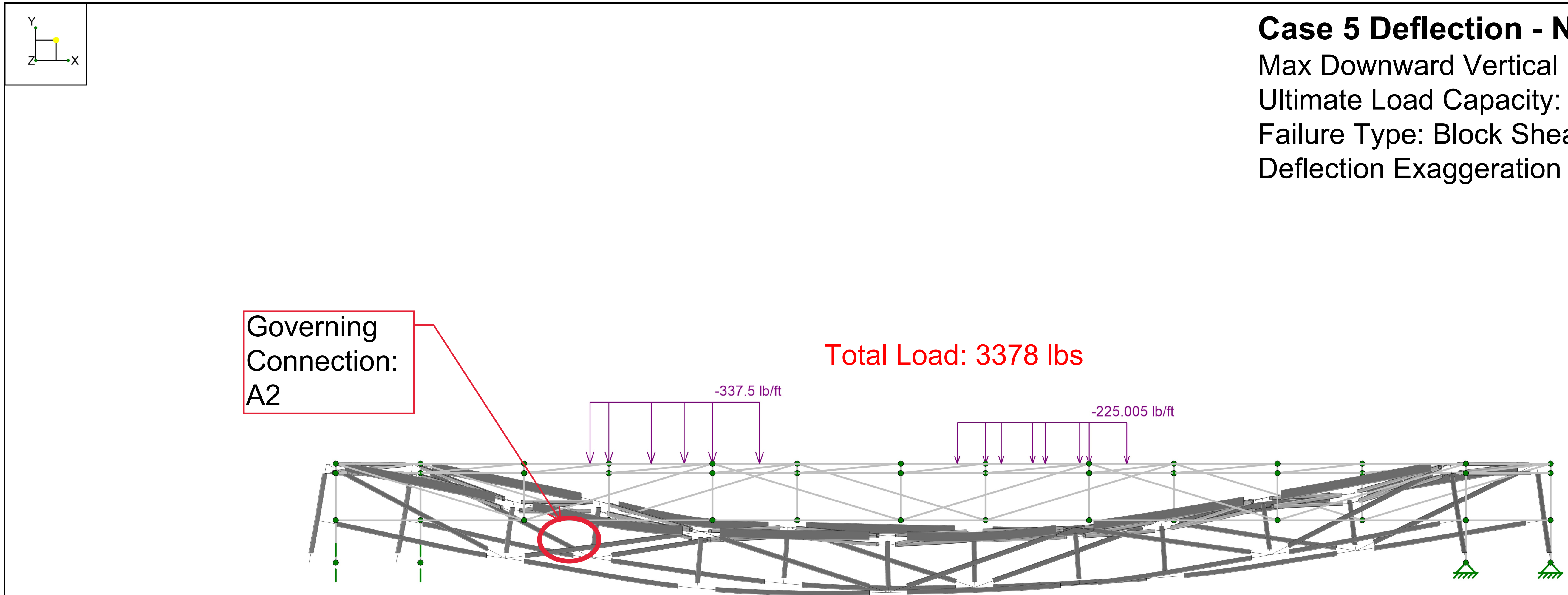
		SK-17
		Apr 07, 2021
		New Bridge Loaded to Failure.r3d

Load Case 5: Analysis of Connection Capacities versus Axial Loading

RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	New Connection i	New Connection j	Capacity i, kips	Capacity j, kips	Controlling Capacity, kips	% Loaded
M56A	BC	11072.038	-4.00	C1	B1	NA	9.46	9.46	-42.29%
M58	BC	15322.754	-5.54	C1	B1	NA	9.46	9.46	-58.52%
M59A	BC	5639.084	-2.04	C1	B1	NA	9.46	9.46	-21.54%
M60A	AB	5455.285	-1.97	B3	C1	NA	NA	NA	Not a Failure Path
M62A	EF	-14578.288	5.27	A1	D2	3.87	NA	3.87	136.10%
M63A	EF	-14411.062	5.21	A1	D2	3.87	NA	3.87	134.54%
M64A	EF	-11563.33	4.18	A1	D2	3.87	NA	3.87	107.95%
M65A	EF	-11232.965	4.06	A1	D2	3.87	NA	3.87	104.87%
M66A	AC	-210.321	0.08	D1	A1	21.94	3.87	3.87	1.96%
M67A	AA	1640.378	-0.59	B1	B1	9.46	9.46	9.46	-6.26%
M35	AA	2631.036	-0.95	B1	D1	9.46	21.94	9.46	-10.05%
M33	AB	9035.552	-3.26	B3	C1	NA	NA	NA	Not a Failure Path
M34	BC	9354.191	-3.38	C1	B1	NA	9.46	9.46	-35.73%
M35A	BC	15759.662	-5.69	C1	B1	NA	9.46	9.46	-60.19%
M36	BC	15957.235	-5.77	C1	B1	NA	9.46	9.46	-60.94%
M37	BC	7008.45	-2.53	C1	B1	NA	9.46	9.46	-26.77%
M38	AB	6844.681	-2.47	B3	C1	NA	NA	NA	Not a Failure Path
M40	AC	-322.787	0.12	D1	A1	21.94	3.87	3.87	3.01%
M41	EF	-15911.695	5.75	A1	D2	3.87	NA	3.87	148.55%
M42	EF	-15944.948	5.76	A1	D2	3.87	NA	3.87	148.86%
M43	EF	-13766.014	4.97	A1	D2	3.87	NA	3.87	128.52%
M44	EF	-13574.886	4.90	A1	D2	3.87	NA	3.87	126.73%
M45	AC	-183.031	0.07	D1	A1	21.94	3.87	3.87	1.71%
M45A	AA	2097.704	-0.76	B1	D1	9.46	21.94	9.46	-8.01%
M56	AE	-9224.655	3.33	B1	A2	9.46	3.87	3.87	86.12%
M57A	EC	6654.795	-2.40	B2	A2	3.87	3.87	3.87	-62.13%
M58A	CE	64.795	-0.02	B2	A2	3.87	3.87	3.87	-0.60%
M59	EC	-2331.791	0.84	B2	A2	3.87	3.87	3.87	21.77%
M60	CE	6832.697	-2.47	B2	A2	3.87	3.87	3.87	-63.79%
M61	AE	-6976.627	2.52	B1	A2	9.46	3.87	3.87	65.13%
M64	EC	3790.114	-1.37	B2	A2	3.87	3.87	3.87	-35.38%
M65	CE	-903.042	0.33	B2	A2	3.87	3.87	3.87	8.43%
M66	EC	-3937.053	1.42	B2	A2	3.87	3.87	3.87	36.76%
M67	CE	5749.704	-2.08	B2	A2	3.87	3.87	3.87	-53.68%
M68	AE	-5544.705	2.00	B1	A2	9.46	3.87	3.87	51.76%
M61A	AC	-338.994	0.12	D1	A1	21.94	3.87	3.87	3.16%
M67B	AE	-10711.158	3.87	B1	A2	9.46	3.87	3.87	100.00%
M41A	BE	792.122	-0.29	C1	A1	NA	3.87	3.87	-7.40%
M42A	CF	416.891	-0.15	B3	D2	NA	NA	NA	Not a Failure Path
M43A	BE	826.869	-0.30	C1	A1	NA	3.87	3.87	-7.72%
M44A	CF	272.818	-0.10	B3	D2	NA	NA	NA	Not a Failure Path
M45B	BE	23.296	-0.01	C1	A1	NA	3.87	3.87	-0.22%
M46	BE	-40.708	0.01	C1	A1	NA	3.87	3.87	0.38%
M47	CF	219.577	-0.08	B3	D2	NA	NA	NA	Not a Failure Path
M48	BE	1566.982	-0.57	C1	A1	NA	3.87	3.87	-14.63%
M49	CF	310.386	-0.11	B3	D2	NA	NA	NA	Not a Failure Path
M50	BE	2054.76	-0.74	C1	A1	NA	3.87	3.87	-19.18%
M52A	AA	2973.526	-1.07	B1	D1	9.46	21.94	9.46	-11.36%
M89A	BC	15345.331	-5.54	C1	B1	NA	9.46	9.46	-58.61%
M89B	AB	10590.187	-3.83	B3	C1	NA	NA	NA	Not a Failure Path

Case 5 Deflection - New Bridge

Max Downward Vertical Deflection: 0.958 in
Ultimate Load Capacity: 3378 lbs
Failure Type: Block Shear
Deflection Exaggeration Scale: 16:1



Loads: LC 8, Case 5
Results for LC 8, Case 5

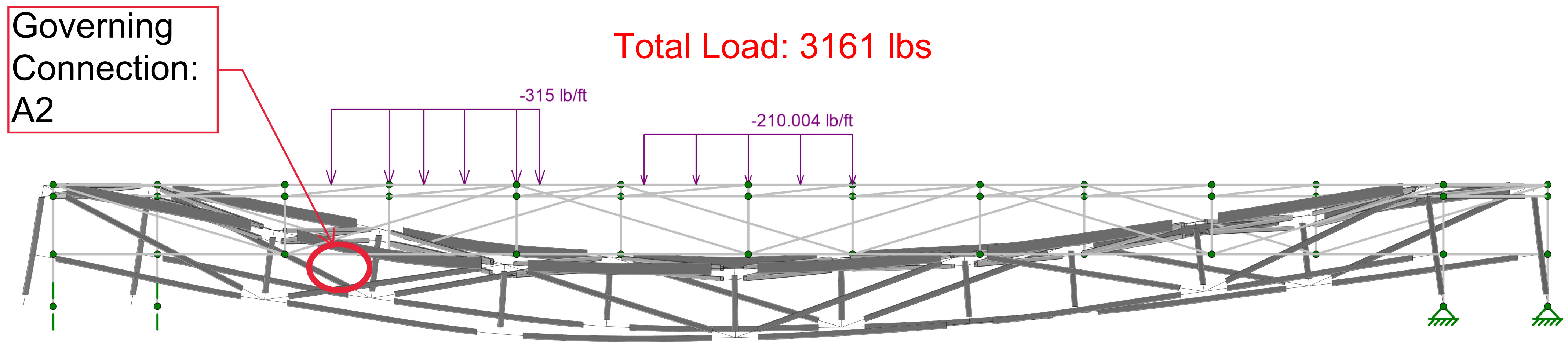
		SK-12
		Apr 07, 2021
		New Bridge Loaded to Failure.r3d

Load Case 6: Analysis of Connection Capacities versus Axial Loading

RISA Label	Plan Set ID	Axial[lb]	Axial[kip]	New Connection i	New Connection j	Capacity i, kips	Capacity j, kips	Controlling Capacity, kips	% Loaded
M56A	BC	11028.502	-3.98	C1	B1	NA	9.46	9.46	-42.12%
M58	BC	13166.131	-4.76	C1	B1	NA	9.46	9.46	-50.28%
M59A	BC	4395.042	-1.59	C1	B1	NA	9.46	9.46	-16.79%
M60A	AB	4220.994	-1.53	B3	C1	NA	NA	NA	Not a Failure Path
M62A	EF	-14356.068	5.19	A1	D2	3.87	NA	3.87	134.03%
M63A	EF	-14345.703	5.18	A1	D2	3.87	NA	3.87	133.93%
M64A	EF	-8633.831	3.12	A1	D2	3.87	NA	3.87	80.60%
M65A	EF	-8457.977	3.06	A1	D2	3.87	NA	3.87	78.96%
M66A	AC	-149.201	0.05	D1	A1	21.94	3.87	3.87	1.39%
M67A	AA	1247.764	-0.45	B1	B1	9.46	9.46	9.46	-4.77%
M35	AA	2750.822	-0.99	B1	D1	9.46	21.94	9.46	-10.51%
M33	AB	9486.496	-3.43	B3	C1	NA	NA	NA	Not a Failure Path
M34	BC	9828.516	-3.55	C1	B1	NA	9.46	9.46	-37.54%
M35A	BC	15779.303	-5.70	C1	B1	NA	9.46	9.46	-60.26%
M36	BC	15827.434	-5.72	C1	B1	NA	9.46	9.46	-60.45%
M37	BC	5585.438	-2.02	C1	B1	NA	9.46	9.46	-21.33%
M38	AB	5472.385	-1.98	B3	C1	NA	NA	NA	Not a Failure Path
M40	AC	-333.845	0.12	D1	A1	21.94	3.87	3.87	3.12%
M41	EF	-15806.488	5.71	A1	D2	3.87	NA	3.87	147.57%
M42	EF	-15821.79	5.72	A1	D2	3.87	NA	3.87	147.71%
M43	EF	-10911.126	3.94	A1	D2	3.87	NA	3.87	101.87%
M44	EF	-10762.564	3.89	A1	D2	3.87	NA	3.87	100.48%
M45	AC	-126.703	0.05	D1	A1	21.94	3.87	3.87	1.18%
M45A	AA	1692.17	-0.61	B1	D1	9.46	21.94	9.46	-6.46%
M56	AE	-9689.814	3.50	B1	A2	9.46	3.87	3.87	90.46%
M57A	EC	6048.737	-2.19	B2	A2	3.87	3.87	3.87	-56.47%
M58A	CE	-98.839	0.04	B2	A2	3.87	3.87	3.87	0.92%
M59	EC	-5208.501	1.88	B2	A2	3.87	3.87	3.87	48.63%
M60	CE	5381.078	-1.94	B2	A2	3.87	3.87	3.87	-50.24%
M61	AE	-5599.226	2.02	B1	A2	9.46	3.87	3.87	52.27%
M64	EC	3614.23	-1.31	B2	A2	3.87	3.87	3.87	-33.74%
M65	CE	980.814	-0.35	B2	A2	3.87	3.87	3.87	-9.16%
M66	EC	-4735.22	1.71	B2	A2	3.87	3.87	3.87	44.21%
M67	CE	4188.294	-1.51	B2	A2	3.87	3.87	3.87	-39.10%
M68	AE	-4298.133	1.55	B1	A2	9.46	3.87	3.87	40.13%
M61A	AC	-365.592	0.13	D1	A1	21.94	3.87	3.87	3.41%
M67B	AE	-10711.605	3.87	B1	A2	9.46	3.87	3.87	100.00%
M41A	BE	1105.336	-0.40	C1	A1	NA	3.87	3.87	-10.32%
M42A	CF	417.808	-0.15	B3	D2	NA	NA	NA	Not a Failure Path
M43A	BE	1713.881	-0.62	C1	A1	NA	3.87	3.87	-16.00%
M44A	CF	150.75	-0.05	B3	D2	NA	NA	NA	Not a Failure Path
M45B	BE	37.999	-0.01	C1	A1	NA	3.87	3.87	-0.35%
M46	BE	52.842	-0.02	C1	A1	NA	3.87	3.87	-0.49%
M47	CF	121.215	-0.04	B3	D2	NA	NA	NA	Not a Failure Path
M48	BE	1189.617	-0.43	C1	A1	NA	3.87	3.87	-11.11%
M49	CF	341.112	-0.12	B3	D2	NA	NA	NA	Not a Failure Path
M50	BE	2103.195	-0.76	C1	A1	NA	3.87	3.87	-19.64%
M52A	AA	3053.205	-1.10	B1	D1	9.46	21.94	9.46	-11.66%
M89A	BC	13501.245	-4.88	C1	B1	NA	9.46	9.46	-51.56%
M89B	AB	10632.609	-3.84	B3	C1	NA	NA	NA	Not a Failure Path

Case 6 Deflection - New Bridge

Max Downward Vertical Deflection: 0.901
Ultimate Load Capacity: 3161 lbs
Failure Type: Block Shear
Deflection Exaggeration Scale: 16:1



Loads: LC 9, Case 6
Results for LC 9, Case 6

		SK-13
		Apr 07, 2021
		New Bridge Loaded to Failure.r3d

Appendix D - Administrative Documents

Contents:

- Proposed Staffing and Cost Estimate
- Actual Staffing and Cost Estimate

		UNLESS OTHERWISE SPECIFIED:		NAME	DATE	
		DIMENSIONS ARE IN INCHES TOLERANCES: FRACTIONAL± ANGULAR: MACH± BEND ± TWO PLACE DECIMAL ± THREE PLACE DECIMAL ±	DRAWN			TITLE:
			CHECKED			
			ENG APPR.			
			MFG APPR.			
		INTERPRET GEOMETRIC TOLERANCING PER:	Q.A.			
		MATERIAL	COMMENTS:			SIZE B / DWG. NO. Appendix D / REV
NEXT ASSY	USED ON	FINISH				
APPLICATION		DO NOT SCALE DRAWING				SCALE: / WEIGHT: / SHEET 1 OF 1

PROPRIETARY AND CONFIDENTIAL

Proposed Staffing and Cost Estimate

Task	Personnel					Sum
	SENG	ENG	EIT	LAB	AA	
Task 1: Existing Bridge Design Analysis	**0**	**15**	**27**	**0**	**39**	**81**
Task 1.1 Loading Scenarios	0	5	10	0	15	30
Task 1.2 Existing Connection Capacities	0	10	17	0	24	51
Task 1.2.1 Analysis of Previous Year's Connections	0	1	2	0	3	6
Task 1.2.2 Tensile Strength	0	3	5	0	7	15
Task 1.2.3 Bearing and Tearout Strength	0	3	5	0	7	15
Task 1.2.4 Tensile and Shear Strength of Bolts and Threaded Parts	0	3	5	0	7	15
Task 2: New Connection Designs	**15**	**55**	**30**	**0**	**0**	**100**
Task 2.1 Solutions To Existing Connection Design Flaws	5	20	10	0	0	35
Task 2.2 Designing to Withstand Minimum Loading For Each Scenario	5	20	10	0	0	35
Task 2.3 Designing to Outperform Existing Bridge Performance	5	15	10	0	0	30
Task 2.3.1 Designed Connection Calculations	5	15	10	0	0	30
Task 3: Modeling and Analysis of the New Design	**19**	**50**	**22**	**0**	**0**	**91**
Task 3.1 SolidWorks Connection Models	10	20	10	0	0	40
Task 3.2 Determination of Theoretical Failure of New Design Using RISA	3	10	4	0	0	17
Task 3.3 Prediction of New Max Load Capacity	3	10	4	0	0	17
Task 3.4 Prediction of New Failure Points	3	10	4	0	0	17
Task 4: New Plan Sets	**4**	**2**	**0**	**20**	**0**	**26**
Task 4.1 New Overall Bridge Plan Sets	2	1	0	10	0	13
Task 4.2 New Connection Plan Sets	2	1	0	10	0	13
Task 5: Construction Materials	**0**	**4**	**0**	**40**	**0**	**44**
Task 5.1 Steel Tubing	0	1	0	10	0	11
Task 5.2 Plate Steel	0	1	0	10	0	11
Task 5.3 Hardware	0	1	0	10	0	11
Task 5.4 All Other Miscellaneous Materials	0	1	0	10	0	11
Task 6: Fabrication	**20**	**40**	**30**	**80**	**10**	**180**
Task 6.1 In-House Fabrication	20	40	30	50	10	**150**
Task 6.2 Outsourced Fabrication	0	0	0	30	0	**30**
Task 7: Bridge Assembly	**10**	**10**	**10**	**15**	**5**	**50**
Task 8 Loading Bridge To Failure	**3**	**3**	**3**	**3**	**3**	**15**
Task 9 Performance Report	**9**	**15**	**30**	**0**	**15**	**69**
Task 9.1 Data From Loading and Failure	3	5	10	0	5	**23**
Task 9.2 Predicted Versus Actual Results	3	5	10	0	5	**23**
Task 9.3 Updated Design Versus The Original Design	3	5	10	0	5	**23**
Total Personnel Hours	**80**	**194**	**152**	**158**	**72**	**656**

Actual Staffing and Cost Estimate

Task	Personnel					Sum
	SENG	ENG	EIT	LAB	AA	
Task 1: Existing Bridge Design Analysis	37	37	1.75	1.75	3.75	81.25
Task 2: New Connection Designs	2	7	9	0	2	20
Task 3: Modeling and Analysis of the New Design	20.5	47.75	56	0	35.75	160
Task 4: New Plan Sets	0	12.5	12.5	0	0	25
Task 5: Construction Materials	0	0	0	8.5	0	8.5
Task 6: Fabrication	0	4	4	92	0	100
Task 7: Bridge Assembly	0	0	0	0	0	0
Task 8 Loading Bridge To Failure	0	0	0	0	0	0
Task 9 Performance Report	0	0	0	0	0	0
Total Personnel Hours	59.5	108.25	83.25	102.25	41.5	394.75

Cost Per Hour	210	150	80	100	55	-
Total Cost Per Position	$12,495.00	$16,237.50	$6,660.00	$10,225.00	$2,282.50	$47,900.00

Appendix E - Field Testing

Contents:

- Testing Photos

		UNLESS OTHERWISE SPECIFIED:		NAME	DATE	
		DIMENSIONS ARE IN INCHES TOLERANCES: FRACTIONAL± ANGULAR: MACH± BEND ± TWO PLACE DECIMAL ± THREE PLACE DECIMAL ±	DRAWN			TITLE:
			CHECKED			
			ENG APPR.			
			MFG APPR.			
		INTERPRET GEOMETRIC TOLERANCING PER:	Q.A.			
		MATERIAL	COMMENTS:			SIZE B DWG. NO. Appendix E REV
NEXT ASSY	USED ON	FINISH				
APPLICATION		DO NOT SCALE DRAWING				SCALE: WEIGHT: SHEET 1 OF 1

PROPRIETARY AND CONFIDENTIAL

Figure 1: Die rolled for load combination.

Figure 2: Early stage of loading bridge with water

Figure 3: Loading of bridge with water

Figure 4: Bridge immediately after failure

Figure 5: Releasing water from tanks after loading to failure

Figure 6: Member failure from loading.

Figure 7: Top chord deformation from loading to failure.