Christopher Caruso & Samuel Garcia
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Steel Bridge Design and Construction

Abstract

A steel bridge was designed under constraints set by the ASCE/AISC Student Steel Bridge Competition. During a 2 stage loading test of 2500 lbs, the bridge had an aggregate deflection of 0.971 in. The bridge weighs a total of 307 lbs and takes 41.3 min to assemble. Its overall performance under the specified criteria is measured at $22,163,200, earning 4th place at the regional competition. This project was made possible through sponsorship by Metals USA and Cherry Hill Steel. Funding for tools and safety equipment was provided by the Swarthmore College Engineering Department.

Key Words: Steel Bridge, Truss, AISC National Student Steel Bridge Competition, Structural Analysis, Steel Design, Finite Element Analysis

Acknowledgements

We would like to thank Professor Faruq Siddiqui, our advisor, for providing his expertise on technical matters, his hours of guidance, and his flexibility in allowing us to work in the shop when we needed it; Grant "Smitty" Smith for his assistance in fabrication of the bridge from stock parts and in transporting our raw materials; Don Reynolds for his help on our welding and willingness to give us the last-minute shop time we needed to finish the bridge on time for the competition; Professor Frederick Orthlieb for accompanying us to the bridge competition and advising us on paint and assembly-related matters; Metals USA, Incs. and Cherry Steel, Inc. for their generous material donations of steel sections, plate, and paint; David Bober, class of 2009, for his hours of help and expertise during the bridge fabrication; Members of the Swarthmore College ASCE Student Chapter for their help with fabrication, assembly, competition details, and their tireless enthusiasm.

Table of Contents

List of Tables and Figures

1.0. Introduction

1.1. Project Appeal

Designing and fabricating a bridge to meet the requirements for the 2008 ASCE/AISC Student Steel Bridge Competition presents realistic constraints and provides the opportunity to take a project from theoretical conception to practical application. Building such a bridge requires acquired knowledge of basic mechanics, steel design, and additionally required a rudimentary understanding of plate loading theory. As an experimental project, the design process is of primary important, thus proper analysis using ANSYS® and MultiFrame® is crucial. In order to find the most optimal design, a cost-benefit analysis has to be developed that takes into account not only the material costs, but the capabilities of available workers with respect to fabricating and building the bridge. AutoCAD®, the industry's drafting standard, is also crucial for creating accurate drawings for fabrication and joint detailing, and SolidWorks serves as an important three-dimensional check on all AutoCAD drawings. Also, participation in the competition provides a comparison of our design to competitors which in turn allows us to improve our design for the future.

1.2. Meeting ABET Requirements

This project satisfies ABET Criterion 3 objectives by providing economic, environmental, social, ethical, health, safety, and manufacturability constraints. Economic constraints involve a $400 budget and optimization of the bridge performance, which is measured in total cost under the competition guidelines. Environmental and social constraints include having a sustainable design, fabricating such that waste is minimized, using shared space during manufacture and construction stages, and maximizing the effectiveness of every volunteer's efforts. Health and safety constraints restricted production of parts beyond any trained workers capabilities. Manufacturing constraints included precision to 1/32” and restricting the availability of materials to only those available by local sponsors.

The project was successfully completed under these constraints. Sponsors were found for the recyclable/reusable construction materials and the painting supplies. Fabrication and construction space was made by reorganizing an old storage space and using it for this project. The paint used was a low VOC paint that was applied by brush to minimize waste. Fabrication was divided according to training and availability, allowing for maximum efficiency. Finally, AutoCAD drawings were made to measurements of 1/32” and all members were designed for simplicity of fabrication.

This project required extensive application of knowledge in engineering, science, and mathematics. As mentioned in the project appeal section, it incorporates learned knowledge and also requires the acquisition of new engineering knowledge. Modern tools for computer aided design, such as ANSYS, MultiFrame, AutoCAD, and SolidWorks, were constantly used in the decision making process.

1.3. The Problem Statement

A bridge spanning a river has met the end of its serviceability and must be replaced. A replacement bridge must be made entirely of steel and its efficiency proven by building a scale model of the bridge and testing it for the client. The bridge must also accommodate decking that will be reused from the old bridge.

Engineers are required to design, model and present their final product. The model must be a 1:10 scale of the final model. It must fit within a 21ft x 6in envelope, with ends of the decking support surface being the reference points for the span length. There is no maximum limit to the width of the bridge. However, it must minimally accommodate 3 ft-9 in decking in its width across the entire span of the bridge. The bridge needs a minimum vertical clearance of 2’-1” over the river and adjacent floodways. The bridge also needs a minimum vehicle passageway with a 3’ width by 2’ft height from the decking support surface to accommodate traffic. The outer edge distance between same side portal footings can not be wider than 4 ft and footings themselves can be no thicker than $1ft^2$. All bridge members have to fit within a 3 ft-6 in x 6 in x 6 in prismatic box. During construction, every member has to attach to the constructed portion with a fastener that consists of a bolt and nut. (See Appendix C for the complete set of rules).

2.0. Concept Design

This bridge was designed in accordance with the 2008 rules for the AISC/ASCE Student Steel Bridge Competition (Appendix C). These rules contain strict regulations on the bridge's envelope, footing placement, portal clearance, decking support surface, and various other dimensional and material aspects of the project. These are discussed in detail in the following sections: 3.1 – 3.2. In particular, our solutions to the problems presented by the competition are presented as the end-point of a multi-step evolutionary process.

2.1. Initial Concepts

The problem put forth by the competition is conceptually a simple one. A pathway over some barrier (a river and floodway in this case) must be designed to safely and reliably carry people from one side to the other. In addition, the solution structure must be tall enough to permit a certain clearance below it.

At this point, a designer under fewer constraints might decide between using a bridge or a tunnel. In our case, the competition clearly eliminated this decision and stipulates that the solution must be a bridge. However, had we been faced with such a decision, we might consider factors such as local weather patterns, typical traffic load, topography of the landing on either sides of the river, and various properties of the soil in the area (i.e., permeability, slope stability, erosion resistance, etc...). The constructability of a tunnel or a bridge would also have to be assessed with respect to all of the aforementioned factors in addition to associated costs.

With the decision made to build a bridge, problems particular to bridges must now be identified and solved. First, the bridge must satisfy certain dimensional constraints. The most significant of these include a defined "envelope" within which the actual bridge must fit (see Appendix C for more details). The most important dimensions of this envelope are the 2'1" minimum vertical clearance between the bottom of the bridge and the river surface, the 20' center-to-center distance between footings on opposite sides of the river (this represents the bridge's full span), and the maximum total bridge height of 6' above the river surface.

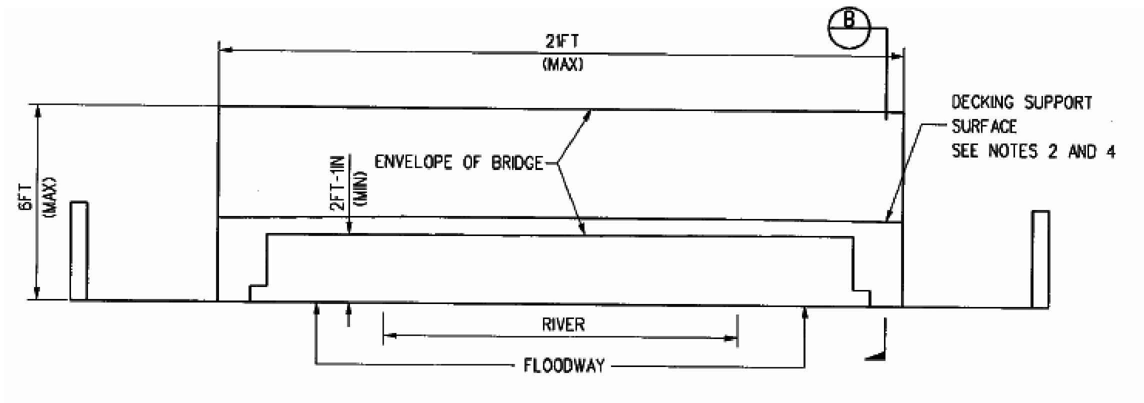


Fig. 2.1. Bridge Envelope – adapted from AISC Student Steel Bridge Competition Rules 2008.

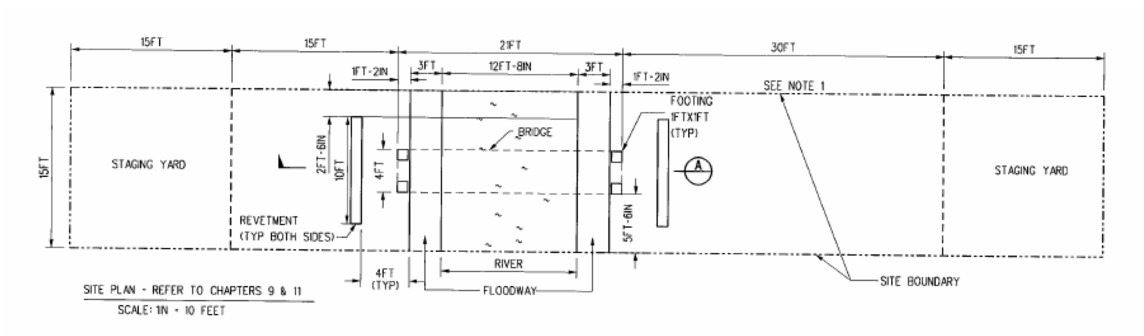


Fig. 2.2. Site plan for proposed bridge – adapted from AISC Student Steel Bridge Competition Rules 2008.

Several different bridge concepts were examined as possibilities for meeting this envelope constraint. All bridges needed to stand up on four supports and support a continuous decking support surface at a height between 2'3" and 2'6" above the river surface. We developed our first concept bridges by examining many different student steel bridges built by other schools for the same competition. We found that the most popular style was a tied- arch bridge, where two girders serve as the main decking support and work as a unit with an arch to resist the applied load. Another popular bridge style was a simple girder bridge with a stiffened deck. Among these two categories, many different varieties existed, thus we decided it was best to explore our options within these two categories.

One of the most important judging parameters in this competition is structural efficiency, which is scaled according to a bridge's stiffness and weight. Therefore, we decided our general approach would be to minimize both of these for each of the bridge concepts and then pick the bridge the bridge with a good balance between stiffness, lightness, and ease of construction. Before fully surveying our design concepts, it is important to note that our material limitations were quite significant. Since our bridge material was to be obtained through donation from a sponsoring fabricator, we were limited to designing with standard structural steel (ASTM A500 grade B) in a limited number of sections. Our designs were modeled with this limitation in mind. The possibility of using department funding was considered as a way of increasing our design options (i.e. Chrome-Moly steel), but the quantities of steel required would have cost at least $350, leaving a very small amount of money left-over for connection hardware, tools, paint, and competition paraphernalia. Thus, we decided department funds would be better spent on these items and we would design our bridge using donated steel.

2.1.1. Girder Concepts

Girder bridges were an appealing option because of their simple construction and low weight. Difference concepts were modeled using ANSYS® Finite Element Modeling Software and deflections predicted under a mid-span load of 2500 lbs. The first girder bridges tested were simply decks constructed of 1.5" square steel tubing. The decking support surface consisted of closely spaced floor beams (18") which transferred the applied loads to the outer girders. Different combinations of thicker sections and stringers in place of floor beams shortly led us to the conclusion that girder bridges tended to have uncontrollable deflections (greater than 10" vertical) when considered with our material constraints.

This problem might have been solvable using an unreasonably deep open-web girder design, however such an approach had several important obstacles. First, it would require skilled welding to complete successfully. As this project was conceived as an ASCE Student Chapter project, it was important for fabrication to be relatively simple so that chapter members could participate effectively. The welding demands of a deep-open web girder bridge would have required at least two students with exceptional proficiency (assuming only minimal help from Smitty, our department machinist). It would have been possible each of us to develop these skills, however we anticipated we would have insufficient time to develop them and would thus have to rely on David Bober '09 to perform most of the welding. Since his help was voluntary, we determined that our bridge's fabrication would be less time-consuming and prone to scheduling problems if we minimized the amount of welding required.

A second problem with the deep open web girder bridges was that girders greater than 6" depth would have been required to minimize deflections enough for the bridge to be legal. The maximum allowable vertical deflection for the competition was 2". Furthermore, the maximum allowable member size was 6" x 6" x 42". This competition constraint caused problems with the girder designs because the only possible solutions we found were either unreasonably heavy (> 6 lbs/ft) or unacceptably stressed (max normal stress > 30 ksi). Additionally, the heavy sections were typically comprised of at least 3 smaller component sections connected by welded rods.

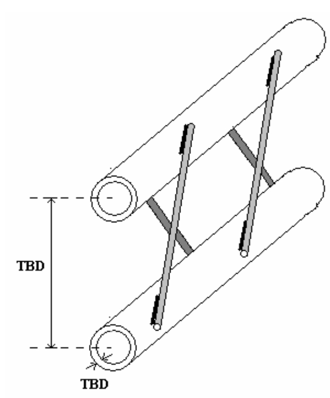


Fig. 2.3: Open Web Girder Concept Drawing.

The optimum size of the individual sections was 1.5" round tube because it allowed approximately 2.5" of clear space between the individual sections and could easily have rods welded on to their walls from any angle. However, these round tubes had limited contact metal for welds (due to their lack of flat surfaces) and were therefore inherently weaker than open web joists constructed from square stock. The optimum number of round sections for an open-web girder was found to be three arranged in an equilateral triangle. This arrangement was determined to be unsuitable because of the weld-area problem cited. Therefore, the next best option was to use square sections instead.

The smallest size available was a 1" x 1" x .065" wall section, two of which were not sufficient for an open web girder. The geometry of the square section forced us to consider using four of these sections in a square arrangement (three sections would not work because connecting rods would not lie flat against the sides of square sections arranged in a triangle). This yielded a section that weighed approximately 6 pounds per linear foot and whose deflection at midspan exceeded 2". Since our goal was to achieve a maximum model deflection of .75" (the models are conservative to more than a factor of 2 because of expected variability introduced during bridge fabrication), these difficulties made an open web girder bridge difficult to effectively design to the contest criterion using donated steel. Even though stiffness was improved by adding bracing between the legs and girders as well as lateral deck bracing, the difference was insignificant compared to the added weight.

The difficulty of fabrication and excessive weight made the open web girder bridges unsuitable for the final bridge design. We then decided to consider detailed examinations of arch and truss type bridges instead.

2.1.2. Tied Arch Concepts

Initial reviews of other competition designs revealed that this type of bridge was the most popular. This concept uses an arch to support two continuous girders, which serve as the decking support. The girders are then tied to the arch at the ends, effectively stiffening the arch and putting the girders in tension. This design allows for lighter girders but uses material in the arch in exchange for the heavier sections used in the girder bridge. Although it is a heavier, the tied arch bridge tends to be stiffer than a girder bridge.

Several different variations on this design were considered, including a tied arch bridge, a diagonal hanger supported arch-girder bridge (DHSAG bridge), and a cable-stay bridge. All of these bridges were modeled under the same loading configuration as the girder bridges, which is a 2500 lb load distributed variably as constant distributed loads on the different floor beams with beams near midspan receiving highest loads.

The tied arch bridge used two open web girders as the decking support surface and floor beams spaced at 40” to connect the girders. On each side of the bridge was a segmented arch constructed of 1.5” dia. 0.110” wall pipe. At each girder joint, vertical hangers connected the girder to the arch (see Fig.2.4). This concept was considerable stiffer (0.44” maximum vertical deflection) than any of the girder bridge designs, and was comparable in weight (350 lbs). Even though this bridge required welding because of its open web girders, it was considerably simpler than the girder bridge because only two round sections were needed in the girders instead of three or four.

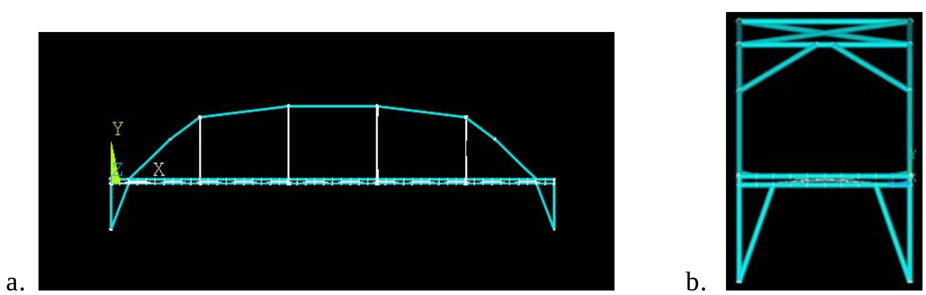


Fig. 2.4. Tied Arch Concept. a) Side Elevation. b) Longitudinal Elevation

The diagonal hanger supported arch-girder (DHSAG) bridge was similar to tied arch bridge except that its hangers were diagonal instead of vertical (see Fig.2.5). This bridge’s height was slightly lower than the tied arch and also used a more gradual arch. Additionally, the arch was tied to the girder about 6” from the end of the bridge in the case of the DHSAG as opposed to the tied arch, which was tied at the ends of the girders. The two bridges were comparable in weight and the tied arch was slightly stiffer than the DHSAG (0.44” vs. 0.48”, respectively).

However, the maximum stress in the girders of the DHSAG was significantly lower than in the tied arch because it was tied at the ends.

a. b.

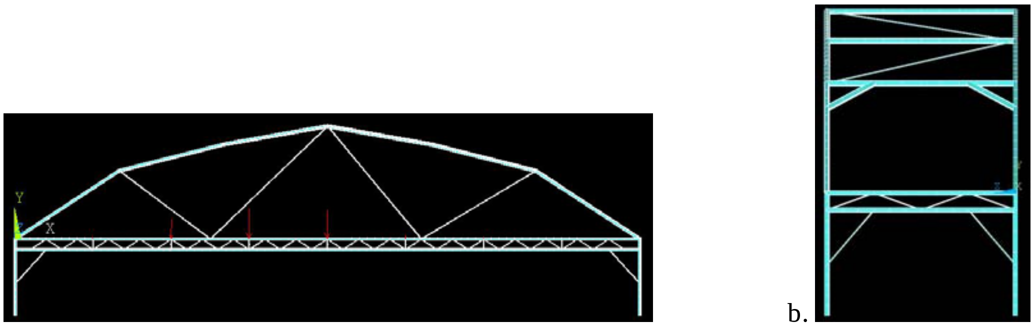
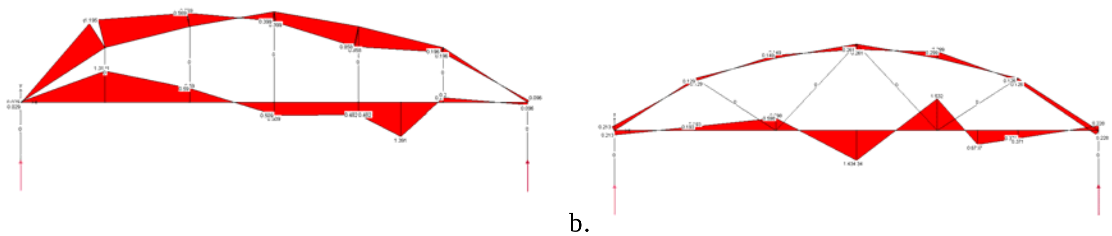


Fig. 2.5. Diagonal hanger supported arch-girder (DHSAG) concept. a) Side Elevation. b) Longitudinal Elevation

The final concept tried in this category was the cable-stay bridge. This bridge is fundamentally different from the tied arch and DHSAG because the girder is prestressed and supported vertically by cables attached to support towers. This bridge was nearly impossible to implement given the location of the footings on either side of the river and not in the middle. This type of bridge is only effective when the cable towers can be placed somewhere near ¼ span on either end of the bridge, thus allowing them to maintain equilibrium by supporting the deck on both sides. Since the decking support surface could not exceed 21' in length, and the footings were separated by 20', it would be nearly impossible to place the towers close enough to midspan for the bridge to function correctly. Additionally, cable-stay bridges are typically assembled outward from the towers and require that each cable be tensioned. Designing a system to pretension cables accurately during timed construction would be difficult, thus this type of bridge was deemed unsuitable.

Among the three bridges designed so far, the DHSAG was determined to be the best candidate for a final design concept among the other supported girder concepts for two reasons. First, the diagonal hangers and the DHSAG were superior to the vertical hangers in the tied arch because they reduced the load on the arch and stiffened the structure in the plane of the arch. Second, the vertical hangers induced more significant flexural stresses on the arch itself (see Fig. 2.6).



a. b.

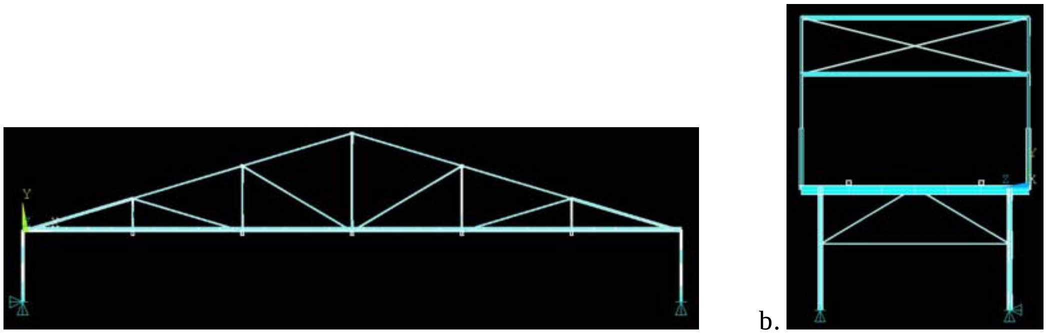
Fig. 2.6. Comparison of bending moments in tied arch concept (a) and DHSAG concept (b).

The flexural stresses on the arch led to noticeably larger sway and thus a greater need for lateral bracing. Since this bracing would likely be difficult to install during the competition, the DHSAG bridge was deemed to be a simpler solution.

2.1.3. Truss Bridge Concepts

The truss bridge concepts were fundamentally different from the tied arch concepts and the girder concept because their decks carried load mainly in the floor beams where the others carried load in the girders. The truss bridge concepts did not have girders functioning as the decking support surface but rather used a system of stringers to transfer the applied decking load to the floor beams. The floor beams then transferred the applied loads to each of the joints along the bottom chord of the truss panels. The result is that deck sag is far more prevalent in these bridges than in the girder bridges, calling for a different approach to deflection reduction. Most of the truss concepts tried had identical decks but differed mainly in their truss geometries. The concepts modeled include a parallel chord Pratt truss, a parallel chord Howe truss, and a triangular Pratt truss.

The first step was to compare the stiffness of the Pratt and Howe truss in two-dimensions. Simple 2-D models confirmed that the Pratt arrangement was slightly stiffer than the Howe, however we still made full-scale bride models in both arrangements to ensure continuity between the 2-D and 3-D results. Truss bridges are fundamentally different from tied arch bridges because they feature a deck that transfers load only to the joints of the truss. The deck therefore requires stringers to serve as the decking support surface (see Fig. 2.7). Load applied to the stringers is then transferred to the truss joints through floor beams.



a. b.

Fig. 2.7. Triangular Pratt truss bridge. a) Side Elevation. b) Longitudinal Elevation.

Truss bridges were considered primarily because of their simple fabrication. These bridges can be constructed using gusset plates and bolts and often do not require open-web members because flexure only occurs in the deck. The DHSAG, on the other hand, must have moment-resisting girders down the entire span, which creates a need for extensive welding.

To further simplify the construction of the Pratt truss bridge, the triangular truss was dimensioned such that all top chord members were the same length, all bottom chord member were the same length, and only two members exceeded 42”. This would reduce the number of splices needed in the final design and allow for construction using an assembly-line. Finally, the Pratt truss was found to be superior to the Howe truss because it was stiffer and the diagonal members were shorter on average, thus the triangular Pratt truss bridge was deemed the best solution.

Once the triangular Pratt truss bridge was fully modeled, deck sag was found to be the most significant problem. Since the truss panels were far stiffer than the deck, we decided to change the hangers of the DHSAG to a Pratt arrangement in an attempt to reduce deflection. This change was highly effective, bringing the vertical deflection of the DHSAG down to 0.240” (see Fig. 2.8).

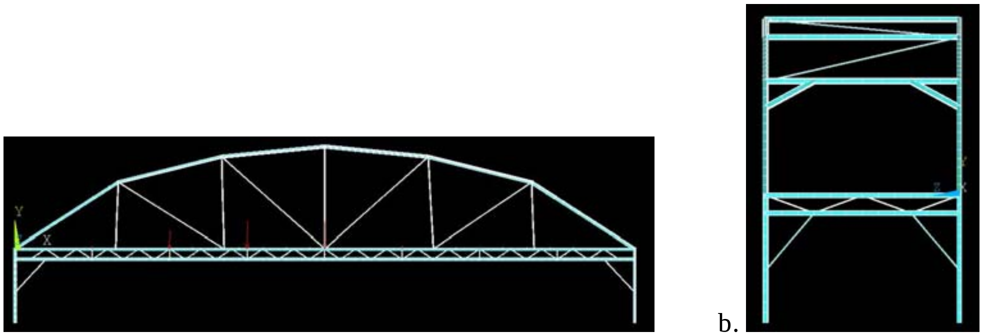
a.  b.

Fig. 2.8. DHSAG bridge: revision 1. a) Side Elevation. b) Longitudinal Elevation.

2.2. Final Concept Designs

At this point, the two final concepts being considered were the new DHSAG, modified with a Pratt arrangement of hangers, and the triangular Pratt truss bridge. Detailed assessments and technical data for both bridges will be presented in this section along with the reasoning behind the final design choice.

2.2.1. Diagonal Hanger Supported Arch-Girder Bridge

The diagonal hanger supported arch-girder (DHSAG) bridge had open web girders at this time, which we soon realized were excessively stiff with the new pratt hanger arrangement. This was determined by observing the model's behavior, particularly the fact that the girders deflected far less than the deck and we sought to have both components deflecting roughly the same amount. We then found a suitable single-section as a replacement for the open web girder, namely a 2" x 1" rectangular 0.065" wall tube. Additionally, we realized at this point that our previous bridge designs had flawed decks. In particular, the AISC rules state that the bridge's deck must be capable of supporting a 3'9" wide by 3'6" long deck with at least ½" clearance on either side. The girder spacing on the DHSAG and other supported girder bridges was between 3'9" and 4', making them unsuitable for the final design because a continuous deck support was needed. In order for the DHSAG to satisfy this criterion, it was necessary to move the girders towards the centerline so their spacing was 3'4" and the hangers connected to the girders through three inch tabs welded to the sides of the girders. After these modifications were made (see Fig. 2.9), the DHSAG was ready to be more rigorously compared to the triangular Pratt truss bridge.

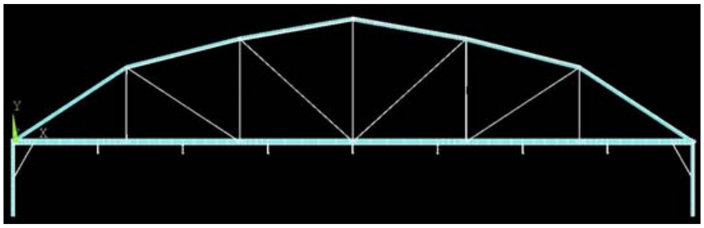
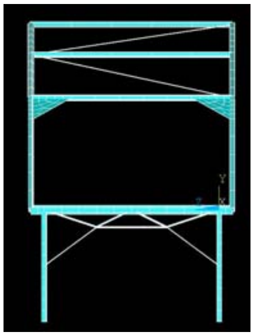
a)  b) 

Fig. 2.9. Final DHSAG concept bridge. a) Side Elevation b) Longitudinal Elevation

Other notable improvements made to the DHSAG first bridge include the use of plates as portal bracing (see Fig. 2.9). This was intended to reduce the weight of the bridge while still retaining the necessary portal stiffening for lateral stability. Additionally, note that the final DHSAG differs from its previous versions in its leg locations, namely that they attach to the end floor beams slightly inside of the floor beam ends. Also notice that the floor beams have been augmented with framing that stiffens them. No lateral bracing system had been proposed for the deck at this time, but single cross braces were used to stiffen the arch.

Finite element models were run in ANSYS® 11.0 to compare the maximum vertical deflections, maximum stresses, and weights of these final concept bridges. These three factors and constructability would wholly determine which bridge would eventually be picked for the final design. The loading for these ANSYS models was derived by creating a simplified model of a girder in Multiframe, supported with a roller at each point the girder attached to a floor beam, and assuming two distributed loads totaling 2600 lbs placed on either side of midspan. From this model, the reactions at each of the supports were then translated into equivalent distributed loads applied on the floor beams so that the girders were effectively loaded with a series of point loads equivalent to the distributed loads modeled in Multiframe. The exact location of the distributed loads was picked based on the same system that would be used in the competition, which sets a constant distance from the end of the bridge and adds a variable component determined by a die roll for each of the two loading plates.

Of the 36 possible load combinations, the one that created the largest bending moment was used in the preliminary models.

We were not concerned that this load model was not exactly accurate to how the applied loads would actually transfer to the deck in the competition because we were more concerned with comparing the behavior of our two concept models under identical loading situations. In particular, we later became aware through Professor Siddiqui that the loading grates would likely contact the deck at only four points, which were determined by their location relative to floor beams. Additionally, we were confident at the time that this loading was reasonably accurate and thus did not concern ourselves any further with it in this preliminary stage. Further discussion of how we modeled the bridge deck can be found in the Analysis section of this report (Section 4).

Fig. 2.10. Finite element analysis of DHSAG deflected (blue) and original (white) shape.

The DHSAG concept bridge had 9 floor beams, spaced at 30” along the deck. Tie-rods were modeled using 3/8” round rod and the top chord/crosses were modeled using 1.5” square 0.065” wall tube. All members were modeled using 42 ksi yield-strength steel with an elastic modulus of 29,000 ksi. An ANSYS® analysis of the final DHSAG bridge model is shown below in Fig. 2.10. The maximum vertical deflection obtained from this model was 0.4886 inches in the girder and the maximum principal stress obtained was 29.816 ksi in the trapezoidal webbing of the loaded floor beams.

2.2.2. Triangular Pratt Truss Bridge

The triangular Pratt truss model was fully developed after our realization of previous deck flaws (mentioned above in section 3.2.1), therefore its final design underwent fewer major revisions than the DHSAG to this point (see Fig. 2.8). This bridge was modeled in ANSYS® under the same loading conditions, modified slightly to properly accommodate the 7 floor beam design used in the triangular Pratt truss bridge.

The results of the ANSYS analysis of the triangular Pratt truss bridge were a maximum vertical deflection of 0.4932 inches and a maximum principal stress of 20.287 ksi in the same floor beam (see Fig. 2.11).

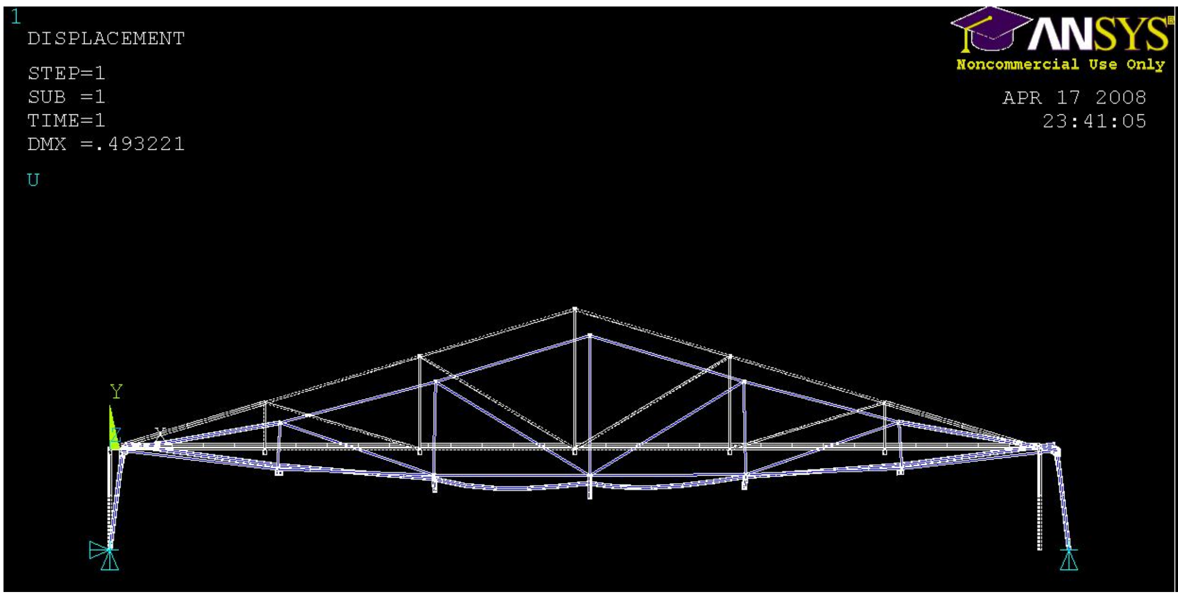


Fig. 2.11. Finite element analysis of triangular Pratt truss Bridge deflected (blue) and original (white) shape.

2.2.3. Comparison of Two Final Concept Designs

In order to effectively compare the two designs, it was necessary to establish a standard of comparison. We chose a weighted sum called the "C Factor." This C Factor consisted of ratios reflecting appropriately chosen limits of maximum stress, deflection, and weight. The definition of the C Factor is as follows:

$$C\ Factor = \frac{\delta_{\max}}{2} + \frac{\sigma_{\max}}{42} + \frac{weight}{400}$$

where: $\delta_{\max}$ = Max vertical deflection (in.),

$\sigma_{\max}$ = Max flexural stress (ksi),

$weight$ = Bridge weight (lbs).

Lower values of C Factor generally indicate better performance with respect to the specified design limits of 2” max vertical deflection, 42 ksi max flexural stress, and 400 lbs max bridge weight. Both bridges were compared based on the C Factor with data obtained from their respective ANSYS® models. This data is tabulated in Table 1 below.

Table 2.1: Tabulated performance data for DHSAG and triangular Pratt truss bridges and Calculated C Factors.

	Max Deflection (in.)	**Max Stress** (ksi)	**Weight** (lbs)	**C Factor**
DHSAG	0.47908	29.816	270	1.86398
Tri. Pratt Truss	0.49322	20.287	290	1.70124

Based on the results from Table 1, the triangular Pratt truss bridge outperforms the DHSAG by a significant amount. Although it has a larger deflection (0.49322 vs. 0.47908) and is 20 lbs heavier, the maximum stress is significantly lower (20.287 vs. 29.816), making the triangular Pratt truss bridge a safer design. Based on this data, the triangular Pratt truss bridge seemed to be the best candidate for the final design concept. However, the issue of constructability had to be addressed for both bridges before the final decision was made.

The modifications made to the DHSAG bridge relieved many of the problems in supported girder bridges due to excess welding because most of the open-web members were replaced by single section members (with floor beams as the exception). However, the 3’6” member length limitation was broken by the floor beams, forcing them to be built with a splice. This would require not only a moment resisting connection in the flange section (1.5” square 0.065” wall tube), but also connections for the webbing below. These connections seemed troublesome primarily because they might be weak in the web, incidentally in the same location as the point of maximum stress in the DHSAG bridge. The solution therefore was either to replace the open-web floor beams with solid sections, which were heavier and less stiff, or develop a strong moment resisting connection for the floor beam splices, which seemed difficult to do without introducing large stress concentrations.

An additional problem with the DHSAG bridge was that most of the hangers were longer than 3'6" as well, necessitating splices. This was problematic because the hangers were originally envisioned to be made of 3/8" round rod, which is simple to splice for members in tension. However, a careful examination of the load combinations on the DHSAG revealed that the hangers were sometimes loaded in compression, which complicated plans for the hanger splices because simple one-bolt splices could no longer be used if the load on each hanger could not be guaranteed as tensile. Furthermore, the buckling load for said hangers was found to be about 40 lbs, indicating that any significant compressive loads would immediately buckle these members, rendering these useless. This then forced us to consider using lightweight square tubing since it would be easy to obtain and had a much higher compressive strength. This again necessitated a large number of member splices, which were considerably more difficult to fabricate for square tubes and also time-consuming to assemble.

The triangular Pratt truss bridge was found to be superior in constructability first because its geometry minimized the number of member splices in the truss to 4 instead of at least 12 in the DHSAG bridge. This would significantly reduce the amount of time necessary to assemble the bridge in competition because each spliced member counts as two components in an assembly instead of one like a single un-spliced member. Reducing spliced members would therefore increase flexibility in the assembly plan. Second, the triangular Pratt truss bridge could be easily designed to isolate flexural stresses to the deck unlike in the DHSAG bridge. The floor beams in the triangular Pratt truss bridge could connect closer to the centerline of the vertical members in the truss panel, minimizing bending stresses induced in the truss panels due to eccentric load transfer. This is far more problematic in the DHSAG Bridge primarily because the hanger attachments are offset from the centerline of the girders. This imparts a significant bending moment on the hangers and significant moment on the girder. As the decking support surface, the girders were already under significant flexural stress (~20 ksi), to which adding a torsional shear stress would result in principal stresses exceeding 30 ksi. These stress levels are unacceptable anywhere in the bridge. Furthermore, the torsion on the girders also introduces a significant stability problem because the girders would tend to rotate under high loads, creating a potentially unsafe situation where the decking support surface is no longer horizontal.

After careful consideration of the constructability difficulties and added torsional stresses in the DHSAG Bridge, it was decided that the triangular Pratt truss bridge would be the best because, although it was not the stiffest and lightest design, it would be easy to fabricate. It would also allow for greater flexibility in the timed-assembly part of the competition in addition to having lower stresses, thus making it safer.

3.0. Analysis

Once the final concept design was decided, the next step was to perform an in-depth analysis of the design and optimize it. This involved first perusing the competition rules so that the bridge model could be build to proper specifications. Various adjustments had to be made at this point, including the following: moving the stringers farther away from the bridge centerline, effectively widening the decking support surface; increasing the elevation of each top chord joint so that the portal frame clearance (2' minimum) was met.

The model was constructed using the finite elements in ANSYS 11.0. The bridge members were modeled all using one material (E=29,000 ksi, $\sigma_y = 42\text{ ksi}, \sigma_u = 58\text{ ksi}$) and several different element types. Rough calculations led us to model the bridge using a 1" x 1" x 1/16" wall HSS section as the primary truss panel section. We decided to also use this section for the stringers and legs. Due to the higher flexural stresses in the floor beams, however, a larger section had to be used. Initial models used a 2" x 1" x 1/16" wall section for the floor beams, however this was eventually changed to a 2” x 1” x .110” wall section after preliminary analysis found the former to have unacceptable stresses.

3.1. ANSYS Analysis of Deck

The most important task within the modeling process was to determine the exact loading configuration the bridge would see in the competition. The competition loading involved placing 42” x 45” x 1.5” steel grates on the decking support surface and stacking 25 lb. angles on these grates to serve as load. The simplest and least conservative way of modeling this is to assume that the grate contacts the stringers (our decking support surface) in a continuous line underneath the grate, effectively transferring a distributed load to the stringers. This model is based on the assumption that the stringers are much stiffer than the grate, allowing complete conformity between the grate and the stringer and thus a distributed load transfer. This assumption, however, is not true and we must consider the stiffness of the grate in our model. Hence, in order to generate a truly accurate deck model, we assumed the grates were much stiffer than the stringers and experimented with different combinations of point couples between the grates and the stringers to find the most realistic configuration. The end objective was then to reduce the transfer of the loaded grates to four point loads that could be applied to the full bridge model as an equivalent critical-load state.

The deck was modeled using beam44 elements and cross-section properties were defined using the “sectype” and “secdata” commands. As described above, 1” x 1” x 1/16” wall sections were used for stringers and the bottom chord of the truss, while 2” x 1” x 0.110” wall sections were used for the floor beams. Additionally, the “secoffset” command was used to move the neutral axis of the floor beams up by ½” so that their top surfaces aligned with the top surfaces of the bottom chord members. The loading grates were modeled using shell93 elements and given an elastic modulus roughly a factor of 100 greater than that used for the steel (29000 ksi). The deck layout and grate locations are shown in Fig. 3.1.

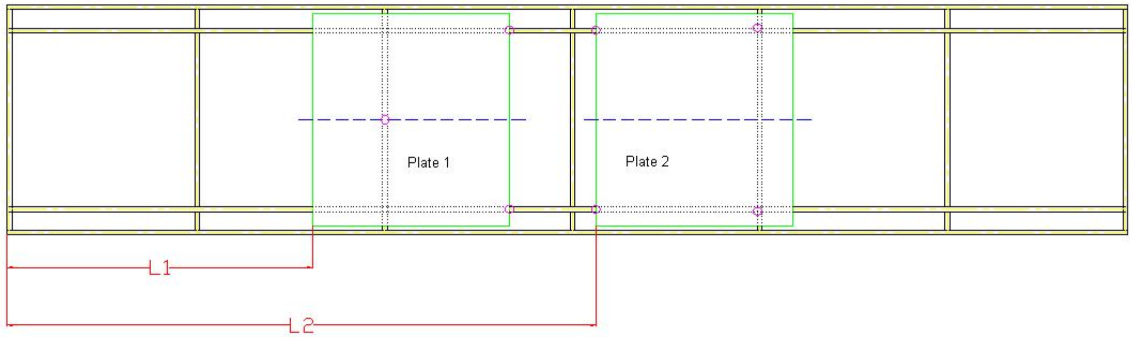


Fig 3.1. Schematic of Deck Model with assumed axes of bending for plates 1 and 2.

Since Shell 93 is an 8 node element (4 corner and 4 edge midpoint nodes), it was necessary to generate a dense mesh of nodes for the plate elements. Each grate was modeled using 6 plate elements since each grating was placed such that it overhung both stringers and sat above one floor beam. Three shell93 elements would be created on one side of the floor beam and the other three shell93 elements on the other side (see Fig 3.2). This approach was used to allow each cantilevered plate section the ability to deflect under load rather than having a single-element grate model that was less capable of conforming to the deck's geometry.

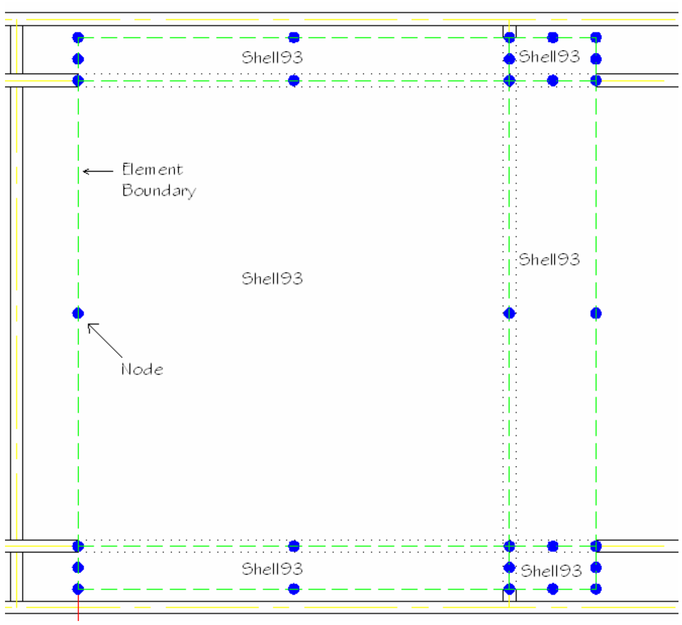


Fig. 3.2. Node locations for shell93 grate model

The connection points between the plates and stringers were picked based on the expected behavior of the grates under load. Particularly, we assumed that the grates could only bend about one axis (either parallel to the bridge's span or transverse to it). Based on the wide stringer spacing, we decided that bending about a longitudinal axis was most likely, therefore this case governed placement of node couplings. Based on this assumption, we could comfortably place two coupling points at the ends of each plate farthest from a nearby floorbeam. Couples were placed here because the grate was assumed to only contact the deck at a maximum of four points. Since the grate length was slightly longer than the distance between floor beams, we deduced that most placement cases would result one side of the plate contacting the the midpoints of the stringers, and the other end contacting the stringers directly above a floor beam. Hence, the node couples were placed at these points, as shown in Fig. 3.2.

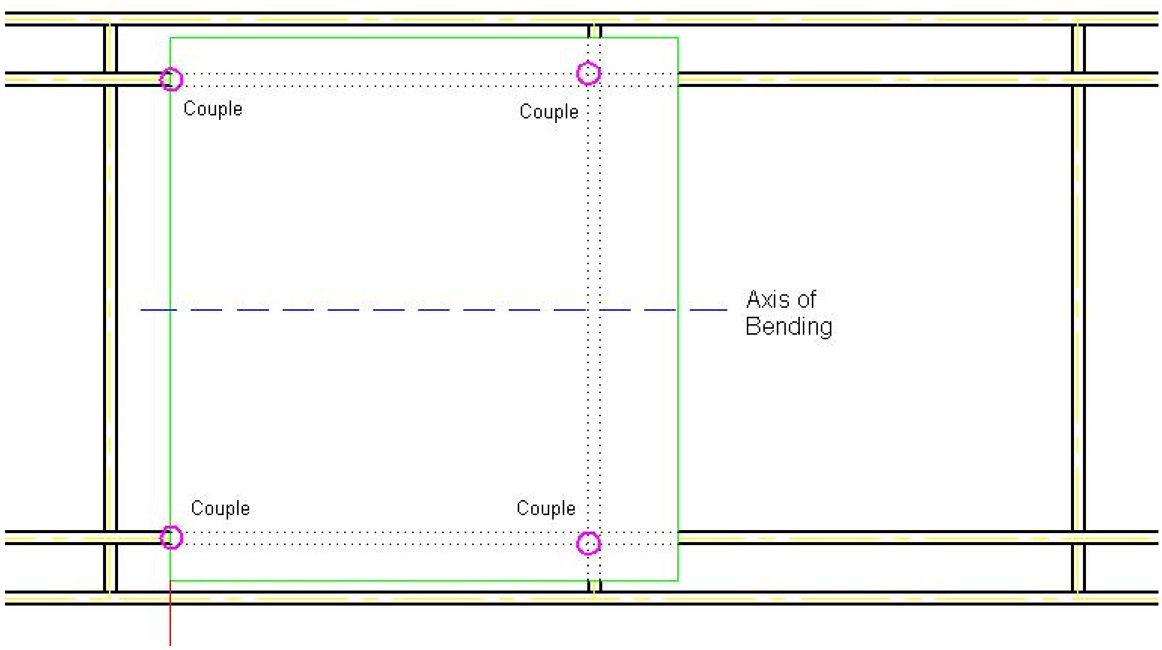


Fig. 3.3. Location of coupling points between stringers and deck grating in deck model.

This model was analyzed using a 1300 lb pressure load applied to the top surface of the plate. From the results of the model, we were able to read the forces transmitted through each node couple and thus use them in the full bridge model as the applied loads. This allowed us to leave the grate models out of the full bridge model so that any associated complications were avoided.

A significant problem we encountered while modeling the stand-alone deck is that the structure was unstable unless all the member connections were assumed to be rigid. However, our actual deck plans used all simple supports (single bolt connections at each end of the stringers and floor beams), thus the validity of the stand-alone deck model was questionable. It was then decided that the best course of action would be to generate the full bridge model and incorporate the grate models there to determine the actual point loads for the final analysis.

3.2. Full Bridge ANSYS Model

The finite element model generated for the entire bridge structure was based on the deck model in that it used the beam44 element for deck components, including stringers and floor beams. The support system (legs) and portal frame were also modeled using beam44 elements since they would be treated as rigid frame structures in the final analysis of the structure. Finally, the top chord, bottom chord, truss branches, and support rods were all modeled using link8 elements, which are tension/compression-only elements used to model two-force members. The truss components, stringers, portal frame, and support legs were all modeled using the 1" x 1" x 1/16" wall HSS cross-section, while the floor beams were modeled using the offset 2" x 1" x 0.110" wall HSS cross-section.

The use of moment releases in this model was important because this was how the support conditions of each member would be modeled. Since we were modeling the bridge as a true truss bridge, it was necessary that every component of the truss panels be strictly two-force members, thus our reasoning for modeling them with link8 elements. In the plane of the portal frame, however, it was necessary that all members be connected rigidly to form a frame structure. Since the truss and portal frame shared several members (i.e., the top chord), it was necessary that beam44 elements be used and appropriate moment releases be applied to allow rotation in the truss plane but not in the portal frame plane.

With a full superstructure and support system, we then found it was possible to model the deck as a simply supported structure as we originally intended. Connecting, through a common node, the floor beams (beam44 elements) with the vertical truss branches (link8 elements) was sufficient for modeling the simply supported condition of the floor beams. For the stringers, however, moment releases were needed through node couples since both the stringers and the floor beam elements were capable of transmitting moment.

Finally, shell93 elements were used to model the load grates and coupled to the stringers in four places as illustrated above in fig. 3.3. The competition loads were applied to the grates and the forces transmitted through the couples were determined. Once the most critical location of plates on the deck support was found (the location that produced the highest combination of flexural and shear stresses in the structures), these corresponding couple forces were applied to the final ANSYS model and the grate models were removed. From this model, the maximum bending moment was 0.273 kip-ft, the maximum shear was 0.656 kips, the minimum axial load was -2.30 kips, and the maximum axial load was 2.10 kips. The bridge's maximum vertical deflection was 0.364 in., located at the midspan floor beam (see Fig. 3.4).

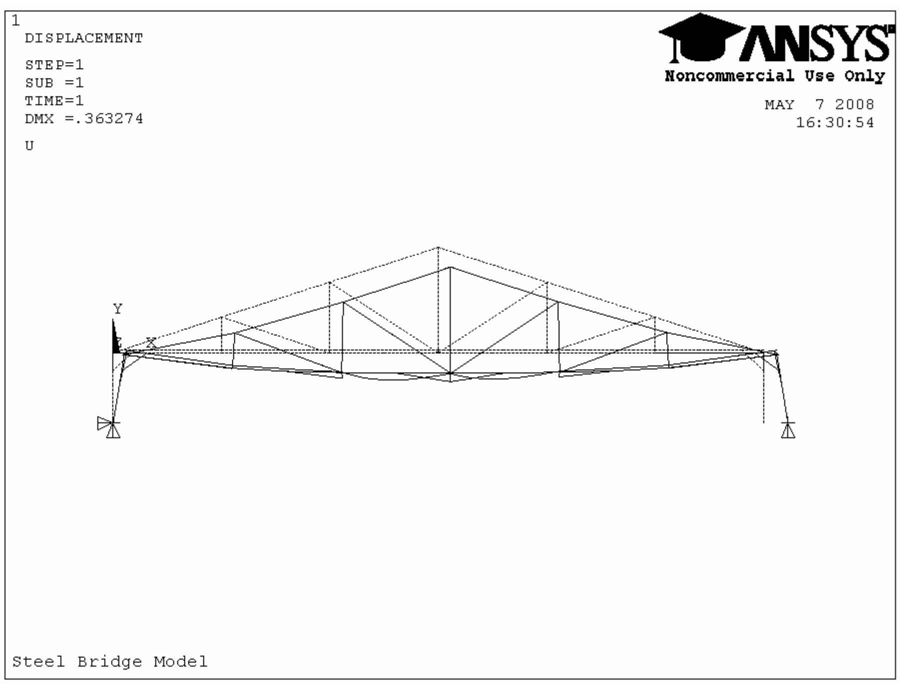


Fig. 3.4. Finite element model side view – deflected shape (solid line) and original shape (dashed line).

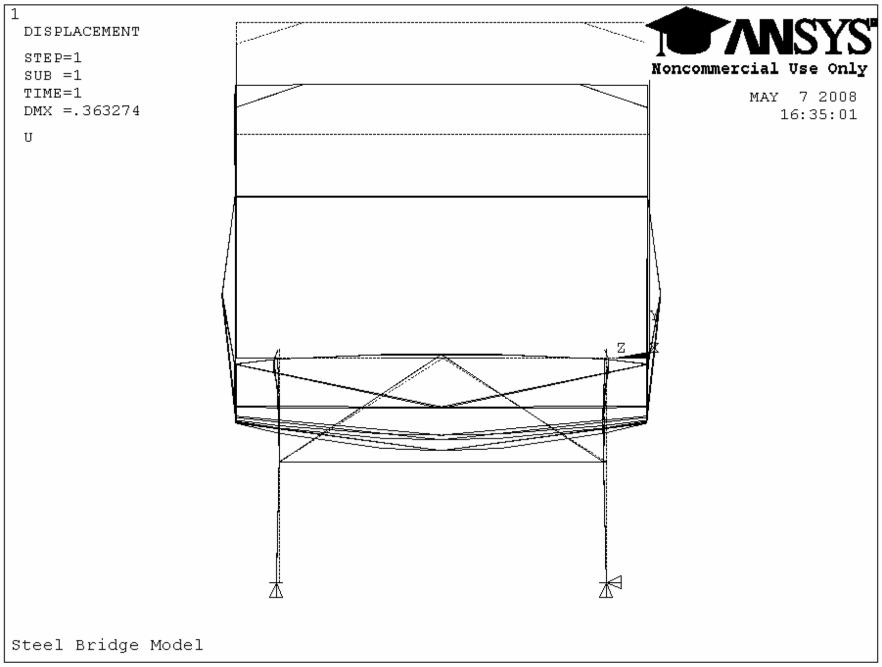


Fig. 3.5. Finite element model portal view – deflected shape (solid line) and original shape (dashed line).

3.3. Full Bridge MultiFrame Model

Once the final results were obtained from the ANSYS model, a check model was created in Multiframe 3D. The exact same section specifications and loading configuration were used and the results obtained were comparable to those from ANSYS. More particularly, the results from ANSYS were roughly 8% more severe than those from Multiframe.

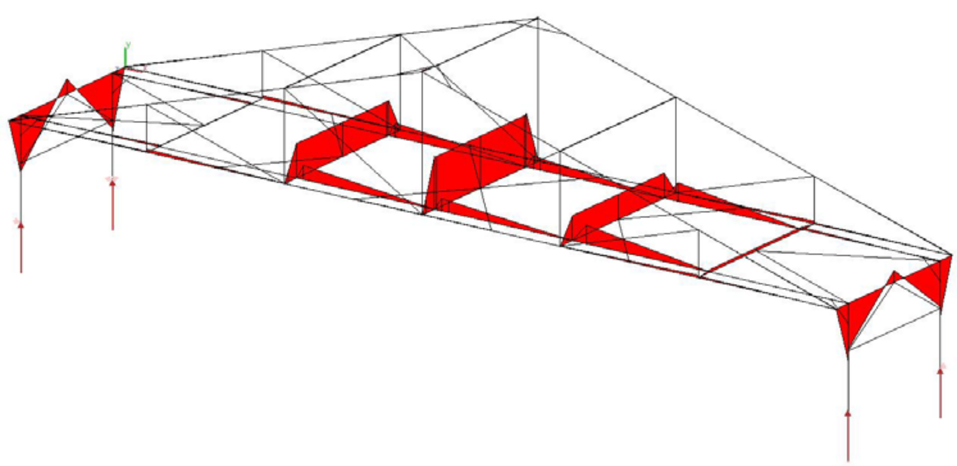


Fig. 3.6. Multiframe model bending moment diagram. Maximum moment (0.273 kip*ft) occurred in the end floor beams.

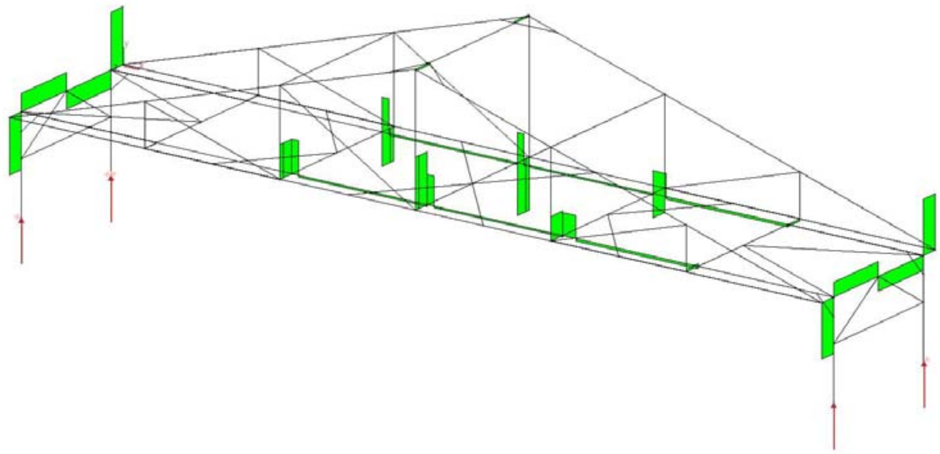


Fig. 3.7. Multiframe model shear diagram. Maximum shear (0.656 kips) occurred in the end floor beams.

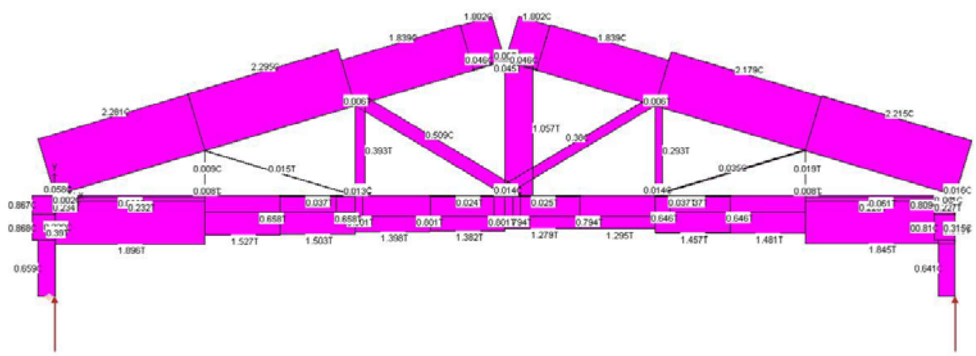


Fig. 3.8. Multiframe model axial load diagram. Maximum axial load(1.90 kips) occurred in the bottom chord next to the floor beams, while minimum axial load (-2.30 kips) occurred in the top chord next to the floor beams.

Once these critical values of moment, shear, and axial load were determined, they were used to manually check the normal and shear stresses in the critical members. The maximum normal stress was 14.96 ksi, the minimum normal stress was 15.22 ksi, and the maximum shear stress was 2.15 ksi. Since the design goal was not to exceed 20 ksi normal stress or 10 ksi shear stress anywhere in the structure, the analysis showed our structure was sufficient for resisting the critical case of load configuration.

4.0. Member Design and Sizing

Once the finite elemnt analysis was complete and cross-checked against Multiframe, the task of member detailing and sizing had to be complete. Entailed in this task were performance of member size checks, material quantity estimates, and completion of design drawings for proper dimensioning.

Member capacities were checked using the AISC 2005 LRFD steel design provisions for HSS sections. Since no live load was assumed to act on the structure, the governing load combination was 1.4D. Due to implementation of a simply supported deck, there were no members under combined loading (i.e., axial load and bending moment) in this structure. Example design computations are reproduced in section 4.1 in the format of a design exercise. This will be useful as an instructional tool for future steel bridge teams.

Most members in the structure measured less than 42" in length, however there are several others that break this rule and thus had to be composed of two members spliced together. These splices were created using a sleeve concept, where a length of section, concentric and slightly larger than the spliced members, would be welded to the end of one of the spliced members. The other end of the sleeve was then bolted into the end of the other splice member so that each splice used only two bolts and was still capable of transmitting moment.

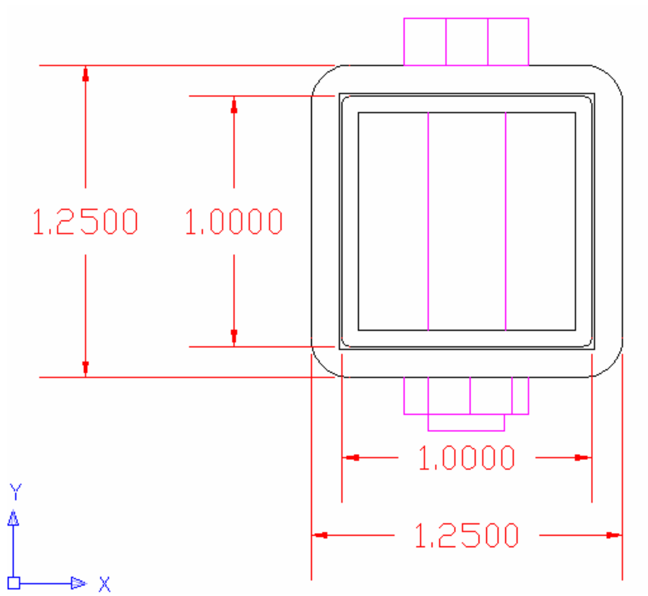


Fig. 4.1. 1" x 1" splice – end view.

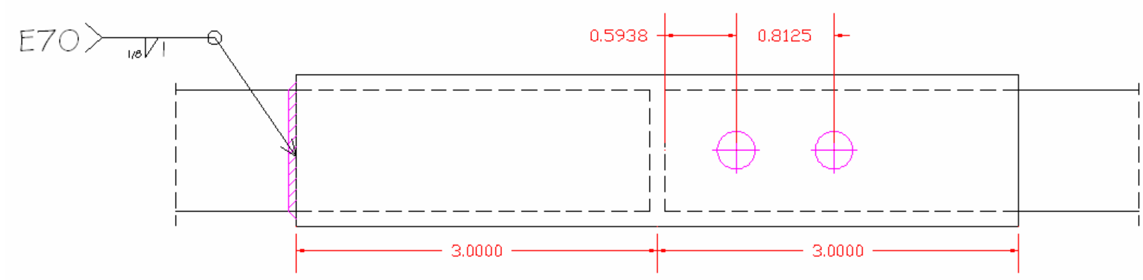


Fig. 4.2. 1" x 1" splice – side view.

Splices were also required for floor beams because they were 48" long. It was especially important that these splices be moment resisting since the floor beams were under flexure. Since Metals USA, Inc., could only provide us with 2.5" x 1.5" x .120" wall HSS for floor beam sleeve section, it was necessary to use 1/8" plate to shim the outside of the floor beam so a tight fit could be made for the sleeve. Diagrams of the floor beam splice are shown in Fig. 4.3 and Fig. 4.4.

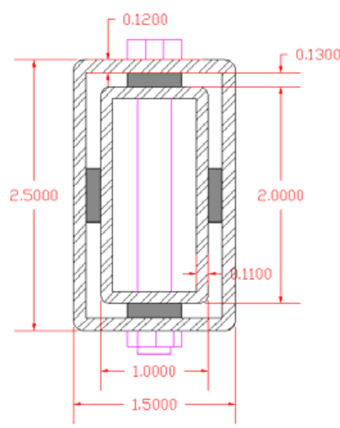


Fig. 4.3. Floor beam splice – end view.

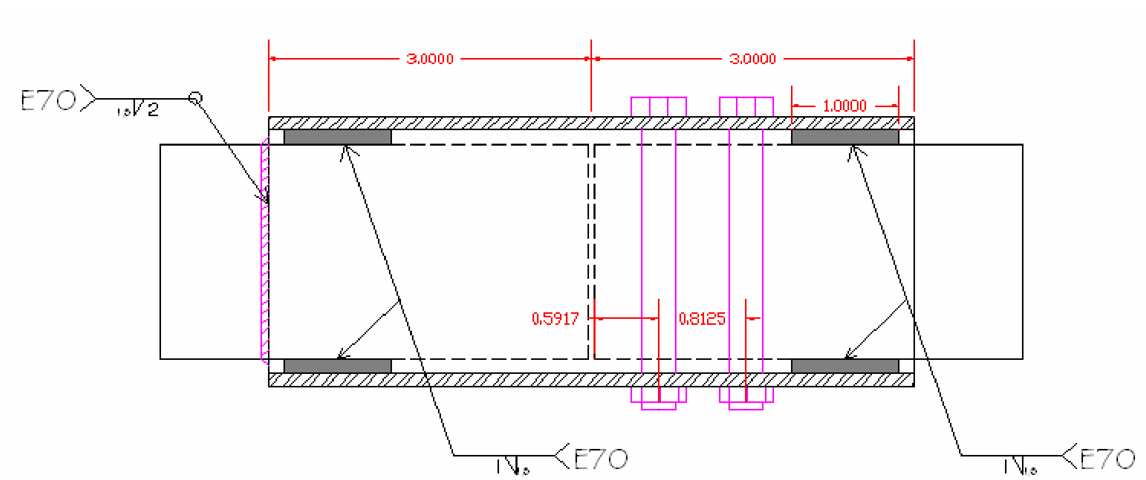


Fig. 4.4. Floor beam splice – side view.

4.1. Design Exercise:

Learn how to design for tension, compression, and flexure using steel HSS sections through the following design exercise. Consider the truss bridge shown below:

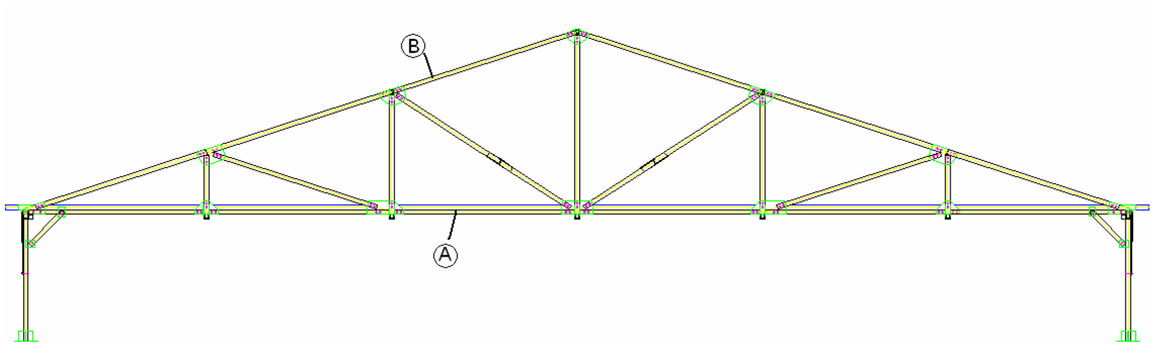


Fig. 4.5. Side elevation

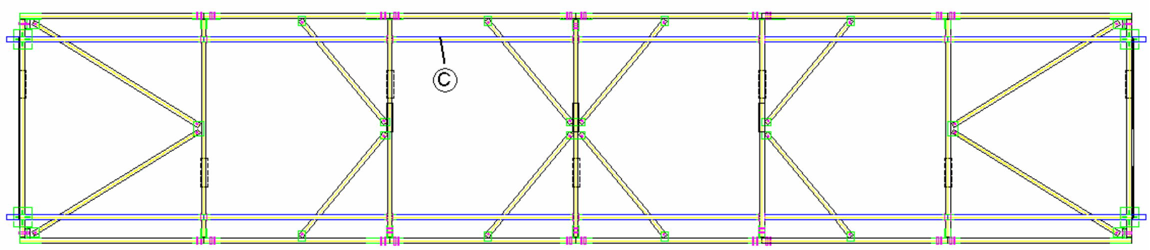


Fig. 4.6. Plan view

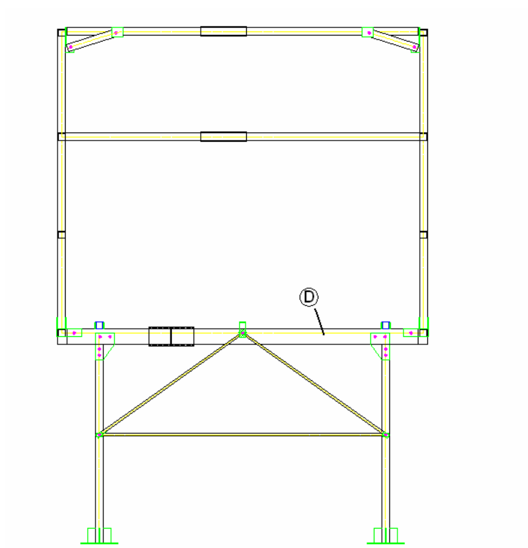


Fig. 4.7. Portal view

Calculate the following properties for the sections and bolt configurations given below. Then, use these values to complete the tension, compression, and flexure design exercises that follow.

Square Section	*Rectangular Section*
Area:___________ in^2	Area:___________ in^2
I_{xx}:______________ in^4	I_{xx}:______________ in^4
I_{yy}:______________ in^4	I_{yy}:______________ in^4
J:_______________ in^4	J:_______________ in^4
Gross Area:_____________in^2	Gross Area:_____________in^2
Effective Area:____________in^2	Effective Area:____________in^2

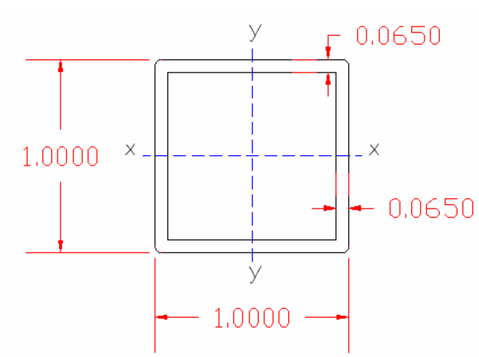


Fig. 4.8. Square HSS section dimensions dimensions

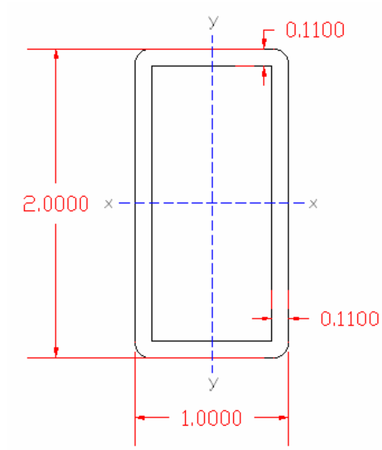


Fig 4.9. Rectangular HSS section

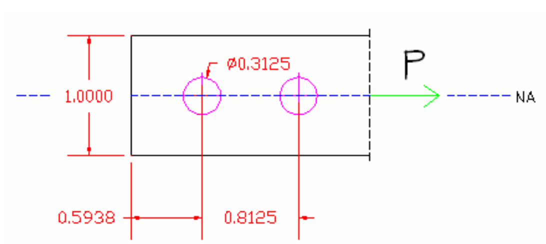


Fig. 4.10. Square HSS connection details details.

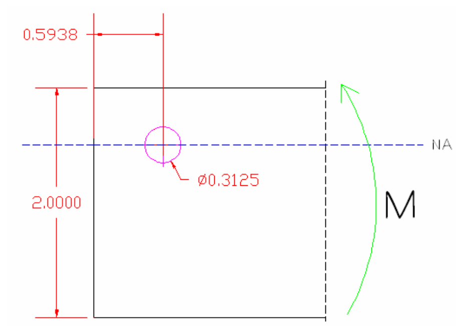


Fig. 4.11. Rectangular HSS connection

Design Member A for Tension:

Given: $P = 2.0$ kips

$f_y = 42$ ksi,

$f_u = 58$ ksi,

Determine whether a 1” x 1” x 0.065” wall HSS section is sufficient.

$P_u \le \Phi_t P_n$

Where:

P_u = Factored load,

$\Phi_t = 0.9$ = Resistance factor for tension members,

P_n = Nominal strength of member.

Determine P_u

P_u is composed only of dead load, therefore 1.4D is the governing load combination.

$$P_u = 1.4D = 1.4(2.0) = 2.8 \text{ kips.}$$

Determine P_n

P_n is determined to be the smaller of the following two expressions:

$P_n = F_y A_g$ for yielding on the gross section, where

F_y = Yield strength of steel (42 ksi),

A_g = Gross cross-section.

$$A_g = (1)^2 - \left(1-(2)(0.065)\right)^2 = 0.2431 \text{ in}^2.$$

$$P_n = (42 \text{ ksi})(0.2431 \text{ in}^2) = 10.210 \text{ kips.}$$

$P_n = F_u A_e$ for fracture on the effective section, where

F_y = Ultimate strength of steel (58 ksi),

A_e = Effective cross-section.

$$A_e = (1)^2 - \left(1-(2)(0.065)\right)^2 - (2)(.065)\left(\frac{5}{16}+\frac{1}{16}\right) = 0.1944 \text{ in}^2.$$

$$P_n = (58 \text{ ksi})(0.1944 \text{ in}^2) = 11.275 \text{ kips.}$$

Yielding on the gross section governs, thus $P_n = 10.210$ kips.

$\Phi_t P_n = (0.9)(10.210) = 9.19 \text{ kips } \ge P_u$

$\therefore$ The section is sufficient.

Design Member B for Compression:

Given: $P = -2.3$ kips

$f_y = 42$ ksi, $f_u = 58$ ksi, $E = 29{,}000$ ksi,

Determine whether a 1” x 1” x 0.065” wall HSS section is sufficient.

$P_u \le \Phi_c P_n$ where $\Phi_c = 0.85$.

Determine P_u using load combination 1.4D:

$P_u = (1.4)(-2.3 \text{ kips}) = -3.22$ kips.

Determine P_n:

Since this section is compact, $P_n = F_{cr} A_g$.

Determine A_g:

$$A_g = 1^2 - (1 - 2(.065))^2 = 0.2431 \text{ in}^2$$

Determine F_{cr} with $Q = 1$:

$$F_{cr} = 0.658^{\lambda_c^2} F_y \quad \text{for } \lambda_c \le 1.5$$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \quad \text{for } \lambda_c > 1.5.$$

Compute λ_c:

$$\lambda_c = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}}$$

where $r = \sqrt{\frac{I}{A_g}} = \sqrt{\frac{0.036}{0.243}} = 0.385$,

$K = 1$, $L = 42"$, $E = 29000$ ksi.

$\lambda_c = 1.3215 \le 1.5$

Compute F_{cr}:

$$F_{cr} = 0.658^{\lambda_c^2} F_y$$

$$F_{cr} = (0.658)^{1.3215^2}(42) = 20.22 \text{ ksi}$$

Compute P_n:

$$P_n = F_{cr} A_g = (20.22)(.2431) = 4.916 \text{ kips.}$$

Compute Design Capacity

$$\Phi_C P_n = (0.85)(4.916) = 4.179 \text{ kips.}$$

$\Phi_c P_n > P_u$

$\therefore$ The section is sufficient.

Design Member C for Flexure:

Given: $M = 1.044$ kip*in

$f_y = 42$ ksi, $E = 29,000$ ksi,

Determine whether a 1” x 1” x 0.065” wall HSS section is sufficient.

$M_u \leq \Phi_b M_n$

Where:

$M_u =$ Factored moment,

$\Phi_b = 0.9 =$ Resistance factor for tension members,

$M_n =$ Nominal strength of member.

Determine M_u

M_u is composed only of dead load moment, therefore 1.4D is the governing load combination.

$$M_u = 1.4D = 1.4(1.044) = 1.462 \text{ kip*in.}$$

Determine M_n

Check that $\lambda < \lambda_c$:

$$\lambda = \frac{h}{t} = \frac{1}{0.065} = 15.38.$$

$$\lambda_c = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29,000}{42}} = 98.80.$$

$\lambda < \lambda_c$

$\therefore$ Section is governed by formation of a plastic hinge.

$$M_n = \frac{F_y I_{xx}}{c}$$

Where:

$F_y =$ Yield strength of steel (42 ksi),

$I_{xx} =$ Moment of Inertia about strong bending axis,

$c =$ Largest distance to outer edge of section from Neutral Axis.

$$M_n = \frac{(42)(.036)}{(.5)} = 3.024 \text{ kip*in.}$$

$\Phi_b M_n = (0.9)(3.024) = 2.722$ kips*in $\geq M_u$

$\therefore$ The section is sufficient.

Design Member D for Flexure:

Given: $M = 3.276$ kip*in

$f_y = 42$ ksi, $E = 29,000$ ksi,

Determine whether a 2” x 1” x 0.110” wall HSS section is sufficient.

$M_u \leq \Phi_b M_n$

Where:

M_u = Factored moment,

$\Phi_b = 0.9$ = Resistance factor for tension members,

M_n = Nominal strength of member.

Determine M_u

M_u is composed only of dead load moment, therefore 1.4D is the governing load combination.

$$M_u = 1.4D = 1.4(3.276) = 4.586 \text{ kip*in.}$$

Determine M_n

Check that $\lambda < \lambda_c$:

$$\lambda = \frac{h}{t} = \frac{2}{0.110} = 18.18.$$

$$\lambda_c = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29,000}{42}} = 98.80.$$

$\lambda < \lambda_c$

$\therefore$ Section is governed by formation of a plastic hinge.

$$M_n = \frac{F_y I_{xx}}{c}$$

Where:

F_y = Yield strength of steel (42 ksi),

I_{xx} = Moment of Inertia about strong bending axis,

c = Largest distance to outer edge of section from Neutral Axis.

$$M_n = \frac{(42)(.3008)}{(1.5)} = 8.4224 \text{ kip*in.}$$

$\Phi_b M_n = (0.9)(8.4224) = 7.58$ kips*in $\geq M_u$

$\therefore$ The section is sufficient.

Check Member D for Shear:

Given: $V = 0.656$ kip*in

$f_y = 42$ ksi, $E = 29,000$ ksi,

Determine whether a 2” x 1” x 0.110” wall HSS section is sufficient.

$V_u \leq \Phi_s V_n$

Where:

V_u = Factored shear,

$\Phi_v = 0.9$ = Resistance factor for members in shear,

V_n = Nominal shear capacity of member.

Determine V_u

V_u is composed only of dead load moment, therefore 1.4D is the governing load combination.

$$V_u = 1.4D = 1.4(0.656) = .918 \text{ kip.}$$

Determine V_n

$V_n = F_n A_w$

Where:

F_n = Nominal shear strength (ksi),

A_w = Area of web.

Check interaction equation to determine F_n:

$$\frac{h}{t} = \frac{2}{0.110} = 18.18.$$

$$2.45\sqrt{\frac{E}{F_y}} = 2.45\sqrt{\frac{29,000}{42}} = 64.38$$

$$\frac{h}{t} < 2.45\sqrt{\frac{E}{F_y}}$$

$$\therefore F_n = 0.6F_y = (0.6)(42 \text{ ksi}) = 25.2 \text{ ksi}$$

$$V_n = (25.2)\left[(2)(0.110)(2)\right] = 11.09 \text{ kips.}$$

$$\Phi_v V_n = (0.9)(11.09) = 9.98 \text{ kips} \geq V_u$$

$\therefore$ The section is sufficient.

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5.0. Joint Design

Constructability is a key factor to consider when deciding upon a final joint design. The type of forces a joint is allowed to transmit highly affects results obtained in finite element analysis. Therefore, the joint's ability to transmit these loads must be determined before in-depth analysis can begin. Once the joint geometry is determined, the loads on a joint can be determined and used to determine the material and minimum dimensions that joint can have. This is done using the 2005 LRFD design provisions provided by AISC Steel Construction Manual. Finally, the dimensions and bolt spacing for each joint can is determined using AutoCAD.

Since a truss bridge is being designed, the major joint forces are axial loads. Analysis through MultiFrame and ANSYS under the worst case scenario found the maximum axial load to be 2.30 kip, the maximum shear load to be 0.66 kip and the maximum bending moment to be 0.5 kip-ft. After adding a safety factor of 1.5, the design load becomes 3.45 kip. With the design load determined, thickness of a gusset plate is calculated using the LRFD Equation 2.6. The shear load and bending moment are neglected because any shear load is transferred to the truss as an axial load and the bending moment only acts parts of the decking support that do not require gusset plates.

Tensile Member Design, LRFD (2005) Equation 2.6

For yielding:

$$P_u \le 0.9F_yA_g$$

$$1.5(2.30\text{ kip}) \le 0.9(42\text{ ksi})(1''\text{x thickness})$$

$$\text{thickness} \ge \frac{3.45\text{ kip}}{0.9(42\text{ ksi})(1'')}$$

$$\text{thickness} \ge \frac{3}{32}$$

so let thickness = 1/8"

The width of the gusset is chosen to be 1 in so that it governs in failure due to its smaller cross sectional area since it has already been determined that the members can safely hold the load. The equation then allows us to determine that a 1/8 in plate would satisfy the yielding criteria.

With the appropriate gusset plate thickness determined, the minimum distance from the edge of a member to the edge of its first hole can be determined by using the AISC Equation J3-6a for shear tear out.

Shear Tear Out (AISC Equation J3-6a)

$$R_n = 1.2F_uL_ct \le 2.4dtF_u$$

where:

F_u = shear fracture stress of connected part

L_c = distance from edge of hole to connected part

t = thickness of connected part

d = bolt diameter

$$3.45 \text{ kip}=1.2(58 \text{ ksi})(L_c)(1/8 \text{ in})$$

$$L_c = \frac{3.45 \text{ kip}}{1.2(58 \text{ ksi})(1/8 \text{ in})}$$

$$L_c = 0.40 \text{ in}$$

$$\text{so let } L_c = 7/16 \text{ in}$$

Using this equation, we found that anything over 7/16 in would suffice. In seeking to reduce the amount of gusset plate used, this clearance was used.

The minimum bolt diameter can also be determined using the second half of the shear tear out equation.

Shear Tear Out (AISC Equation J3-6a)

$$R_n \leq 2.4dtF_u$$

$$d \geq \frac{3.45 \text{ kip}}{2.4(58 \text{ ksi})(1/8 \text{ in})}$$

$$d \geq 0.20 \text{ in}$$

Now, knowing that any bolt larger than 0.20 in diameter satisfies the shear tear out equation, a nominal bolt diameter can be determined using a standard stress equation for the design load in single shear. Because the competition restricts the type of bolt that can be used to a minimum of Grade 2, 57 ksi yield strength was used for the limit state.

Standard Stress Equation

$$\frac{P}{A} = \sigma_{\max}$$

$$\frac{3.45 \text{ kip}}{\left(\frac{\pi}{4}\right)d^2} \leq 57 \text{ ksi}$$

$$d \geq \sqrt{\frac{3.45 \text{ kip}}{57 \text{ ksi}\left(\pi/4\right)}}$$

$$d \geq 0.28 \text{ in}$$

This determined any Grade 2 bolt larger than 9/32 in. diameter could hold the design load. Thus, a 5/16 in. diameter was selected.

With the bolt diameter now chosen, the single bolt- single gusset plate joint now needs to be checked against fracture. This can be done using the LRFD Equation 2.6.

Tensile Member Design, LRFD (2005) Equation 2.6

For fracture:

$$P_u \le 0.75 F_u A_e$$

where $A_e = A_n U$ and $\mathrm{U} = 1.0$ for this case

$$3.45 \text{ kip} \le 0.75(58 \text{ ksi})\left(1/8 \text{ in. x } (1\text{in-}5/16\text{in})\right)(1)$$

$$3.45 \text{ kip} \le 0.75(58 \text{ ksi})\left(0.086 \text{ in}^2\right)(1)$$

$$3.45 \text{ kip} \le 3.74 \text{ kip*}$$

* Safe against fracture*

The connection must also be checked against block shear. This is done using AISC Equation J4-5.

Block Shear Check AISC Equation J4-5

Lesser of the two:

$$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt}$$

or

$$R_n = 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$$

where:

A_{nv} = net area along the shear surface or surfaces

A_{nt} = net area along the tension surface

U_{bs} = equals 1.0 (for this case)

Lesser of the two:

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt}$$

$$= 0.6(58\text{ ksi})(1/8\text{"x } 7/16\text{"}) + (1)(58\text{ ksi})(1/8\text{"x } 5.5/16\text{"})$$

$$R_n = 4.40\text{ kip}$$

or

$$R_n = 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

$$= 0.6(42\text{ ksi})(9.5/16\text{"x } 1/8\text{"}) + (1)(58\text{ ksi})(5.5/16\text{"x } 1/8\text{"})$$

$$R_n = 4.36\text{ kip}$$

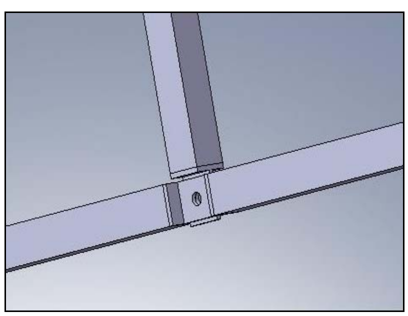
3.45 kip ≤ 4.40 kip*

Safe against block shear

With the connections satisfying the block shear requirements, we determine that 1/8" thick gusset plates and 5/16 in. diameter bolt are the minimum allowable dimensions for our joint designs. Now the joints can be evaluated for constructability and competition legality.

5.1. Joint Concepts

The initial concept for creating a pin connection was called Joint Concept #1, seen below in Figure 5.1:



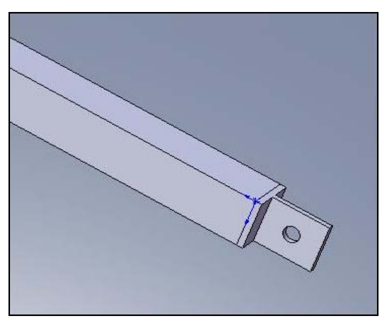


Fig 5.2. Tab Concept for single bolt pin connection

The advantages of this design include one-bolt connections and easy manufacture. The disadvantages include slight joint eccentricity and a maximum practical limit of three members per joint. Since some joints require connect more than three members, this design was deemed unsuitable.

A second design was then developed to allow for more member per joint. This concept was called Joint Concept #2 is shown in Figure 5.2 below.

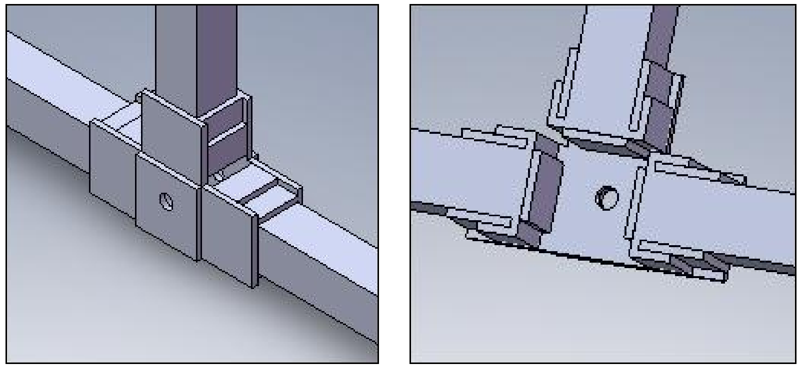


Fig. 5.3. Single Bolt Concept with 2 Gussets

The advantages of this design include quick assembly, since the joint features guide tabs, and one-bolt connections. The disadvantage of this design is its highly complicated fabrication.

A third concept (Figure 5.3) involved welding T-shaped pins onto a gusset plate. This design would a rigid connection to the gusset plate with only one bolt. The problem with this design is finding a proper way to weld the pins without warping the plate. The design calls for slotted holes, which increased manufacture time.

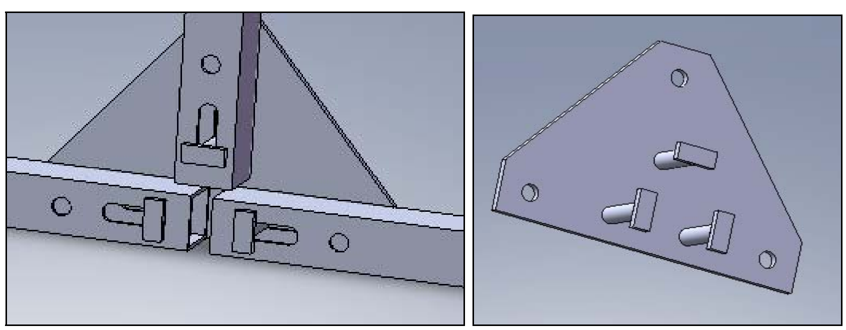


Fig. 5.4. Joint Concept #3

Since the previous designs were deemed unsuitable or difficult to fabricate, a simpler concept was considered. As seen in Figure 5.4, this new design uses two gusset plates per joint and two bolts per member in each joint. It also includes tabs to easily attach the truss panels to the deck using one bolt per floor beam end. It can also accommodate all joint configurations required by the bridge geometry while maintaining a working center for each joint. This design is also simple to fabricate and can satisfies competition criterion for legal joints, thus making it the final joint decision.

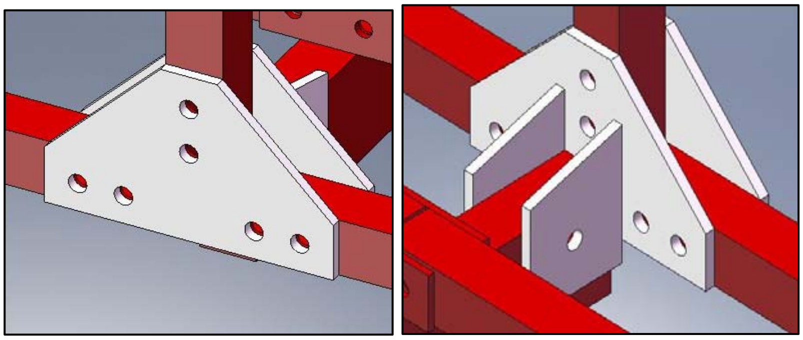


Fig. 5.5. Final Joint Design

With a final joint design picked, determination of proper bolt spacing was necessary to safely accommodate our two-bolts per member design. For this, AISC Table J3.3 would have been used, however values are not tabulated for 5/16" diameter bolts, thus the proper bolt spacing was dictated by the (2 2/3)D rule. A spacing of 13/16 in. center to center was found to be sufficient.

<u>Bolt Spacing (AISC Table J3.3)</u>

$$\text{Minimum spacing (center to center): } 2\frac{2}{3}d$$

$$= 2\frac{2}{3}(5/16)$$

$$\text{Minimum spacing } \approx 13/16 \text{ in}$$

For welded joints, the design load was the maximum shear force of 0.66 kip transferred from the floor beams to the truss panels. Using AISC Equation

J2-5 and eccentric load calculations, a sufficient weld length was to be determined through an iterative solution assuming weld widths of 1/8 in.

Weld Shear Strength (AISC Equation J2-5)

$$\mathrm{R_n} = 0.707 wLF_w$$

where:

$$F_w = 0.6F_{EXX}(1.0 + 0.50\sin^{1.5}\theta)$$

$$w = \text{width of the weld}$$

$$L = \text{length of the weld}$$

$$\begin{aligned} F_w &= 0.6F_{EXX}(1.0 + 0.50\sin^{1.5}\theta) \\ &= 0.6(70\text{ ksi})(1.0\text{+}0) \\ &= 42\text{ ksi} \end{aligned}$$

Stress due to: Direct shear

$$f_1 = \frac{P}{L} = \frac{0.66\text{ kip}}{L}$$

$$L = \frac{0.66\text{ kip}}{42\text{ ksi}} = 0.13\text{ in} \approx 3/16\text{ in}$$

Stress due to: Moment

$$\begin{aligned} f_2 &= \frac{Md}{J} \\ &= \frac{\left[(0.66\text{ kip})(1/16\text{ in+}1.0315\text{ in})\right]\left[\left(L/2\right)^2 + 1.0315^2\right]^{0.5}}{\left(\left(1/8\right)^3 L + \left(1/8\right)L^3\right)/12} \end{aligned}$$

$$L \approx 1.22\text{ in=1 7/32 in}$$

The shortest allowable weld length was found to be 19/32 in., so the weld length used in our joint designs was 2 in. since most joints allowed at least this much.

With all failure modes checked and a final design chosen, the finalization of joint geometries was completed using AutoCAD. This allowed for each joint to be specifically tailored to the exact member geometries in the bridge drawings and precise fabrication drawings to be made for each gusset plate.

6.0 Timed-Construction Plan

6.1. Construction Plan Design

The designs of the construction plan and joints had to be done simultaneously since the joint designs highly affected the order of construction. Having finalized the joint designs, could then be finalized as well.

There are three main concerns that the plan must address. First, the total time of construction needs to be minimized by reducing the number of bridge members, i.e. individual bridge pieces that can be composed of rigidly connected members. Second, the additional cost incurred by using temporary piers has to be minimized. Finally, the construction plan has to be organized to distribute the workload as evenly as possible.

6.1.1. Member Construction

The number of total construction members can be minimized by rigidly attaching each set of gusset plates to a member. This allows all rigidly connected parts to count as a single construction member, thus increase the carrying capacity of each worker since there were no free gusset plates (which would have counted as individual members had they not been rigidly connected to members). The components of the constructed member are rigidly connected with bolts, which allowed for last minute adjustments if needed. Drawings of the construction members can be found in Appendix E.

6.1.2. Cost Reduction of Temporary Piers

The necessity of temporary piers depended on whether assemblies could be made to maintain their stability during the construction process without external support. We thus found that the best way to minimize the number of piers necessary was to assemble the bottom chords components simultaneously and attach them to the construction portion of the bridge in one step, requiring only a single pier to support each bottom chord at midspan for a total of two temporary piers.

6.1.3. Construction Organization

Every part of the construction plan was feasible and each construction member was classified into sections that determined how the construction member could be installed on the constructed portion. These classifications were characterized by what personnel could install it, namely builders on land, a combination of barge and land builder, a barge alone, or both barges together.

6.2. The Plan

With the previous considerations in mind, the following plan was devised. The construction members were divided so that each staging yard contained half of the bridge's components. The only difference was that the staging yard closest to the river would have the floor beam and top bracing that connect the middles of the truss panels. The construction was broken into ten stages, which are shown below in Fig. 6.1 through Fig. 6.10. After completion of stage 10, the final step in construction is to remove the temporary piers and stop the clock.

Number of Workers: 6 workers= 4 stage yard workers + 2 barges

Stage 1

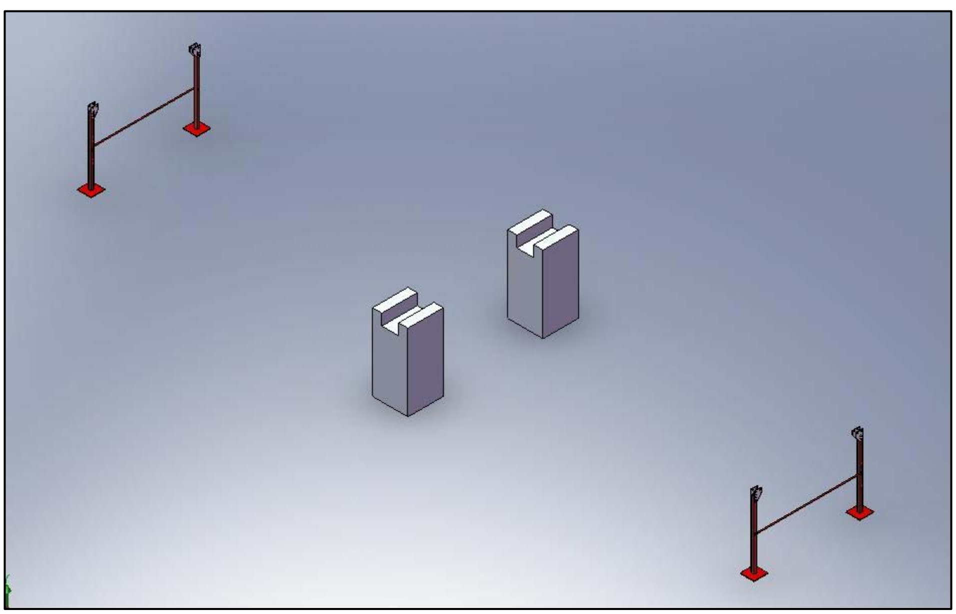


Fig. 6.6. Assembly #1 (support legs) and temporary piers

Stage 2

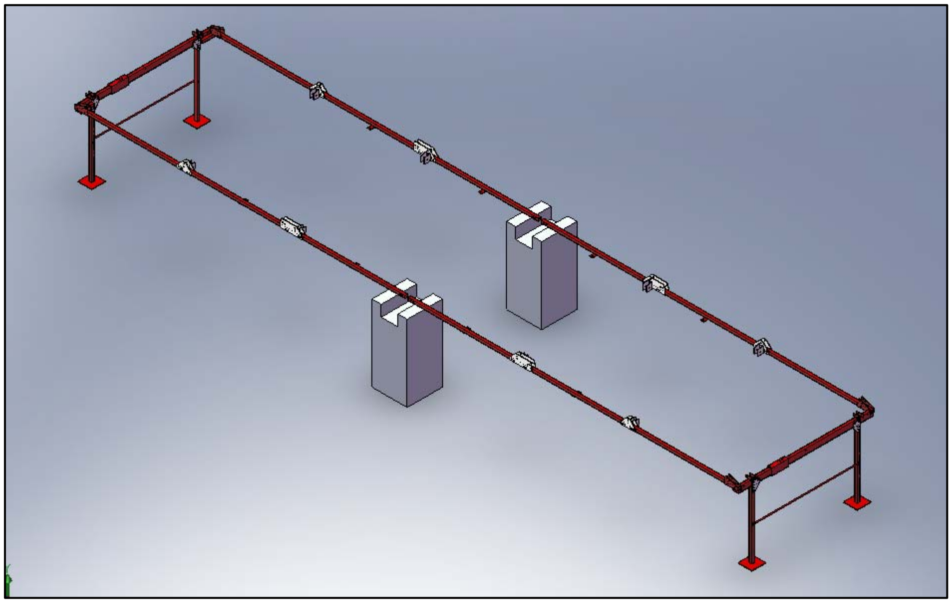


Fig. 6.7. Assembly #2 (end floor beam) and Assembly #3 (bottom chord portion)

Stage 3

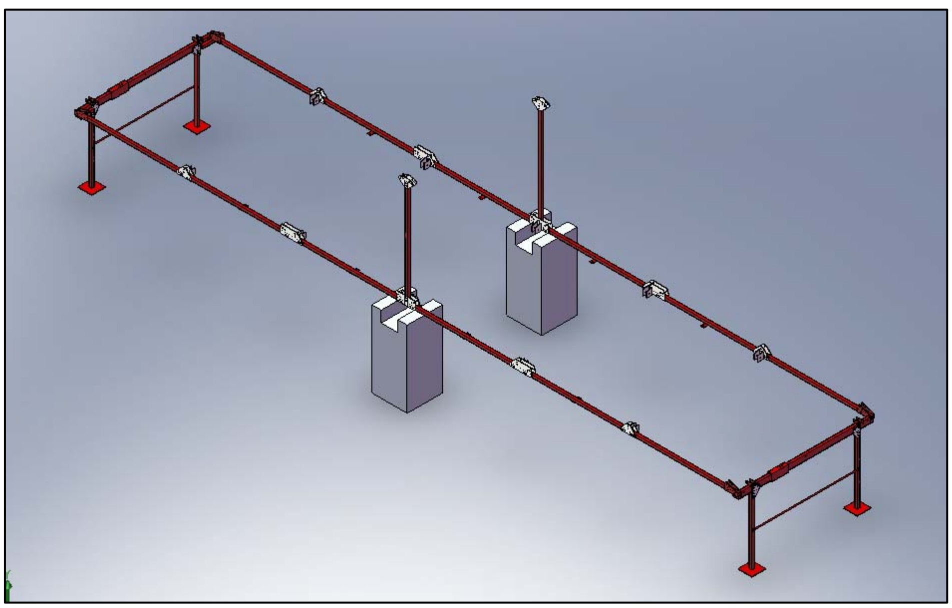


Fig. 6.8. #4 Members (Midspan vertical truss branches)

Stage 4

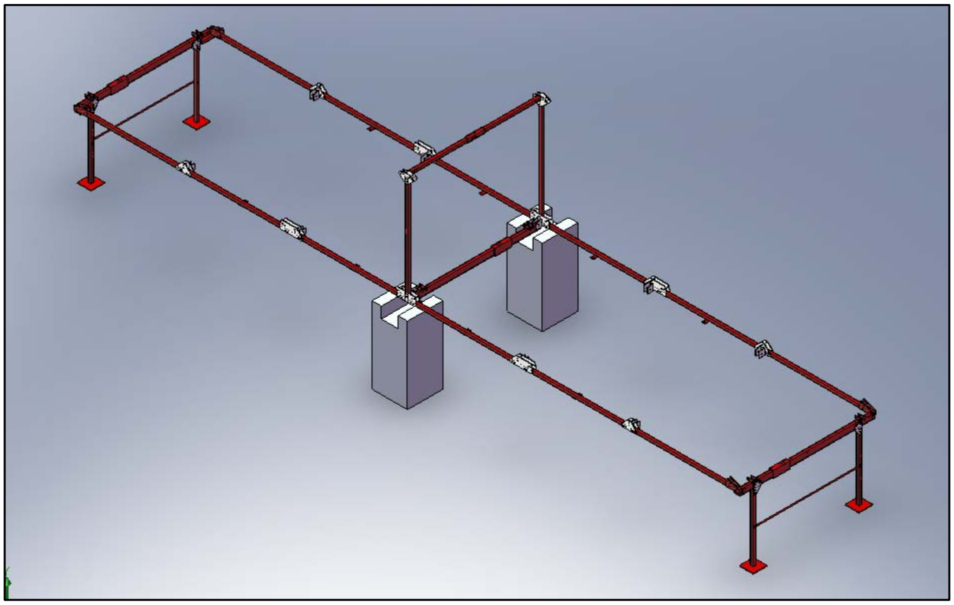


Fig. 6.9. Assembly #5 (midspan floor beam) and Assembly #6 (top cross brace)

Stage 5

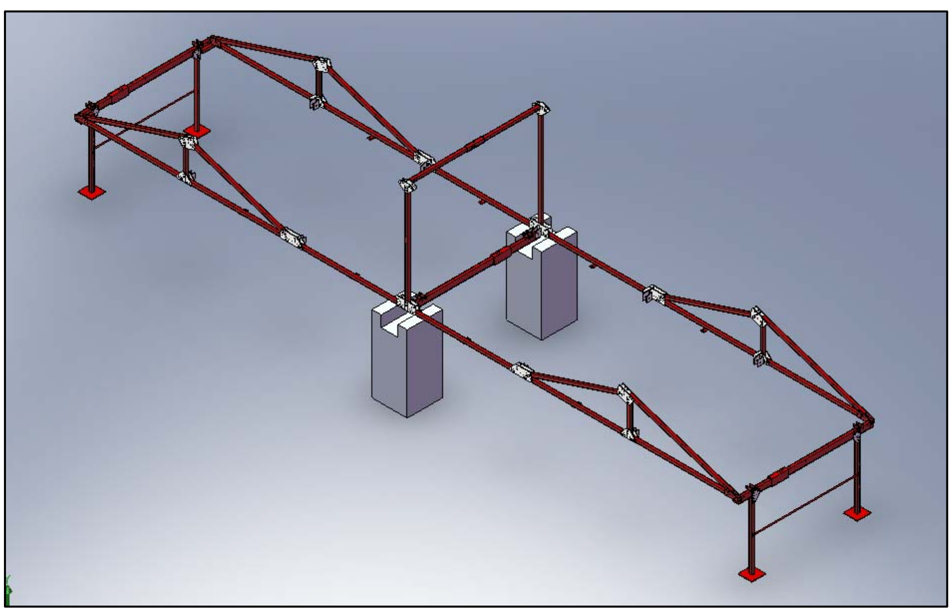


Fig. 6.10. Assembly #7 (truss panel components 1)

Stage 6

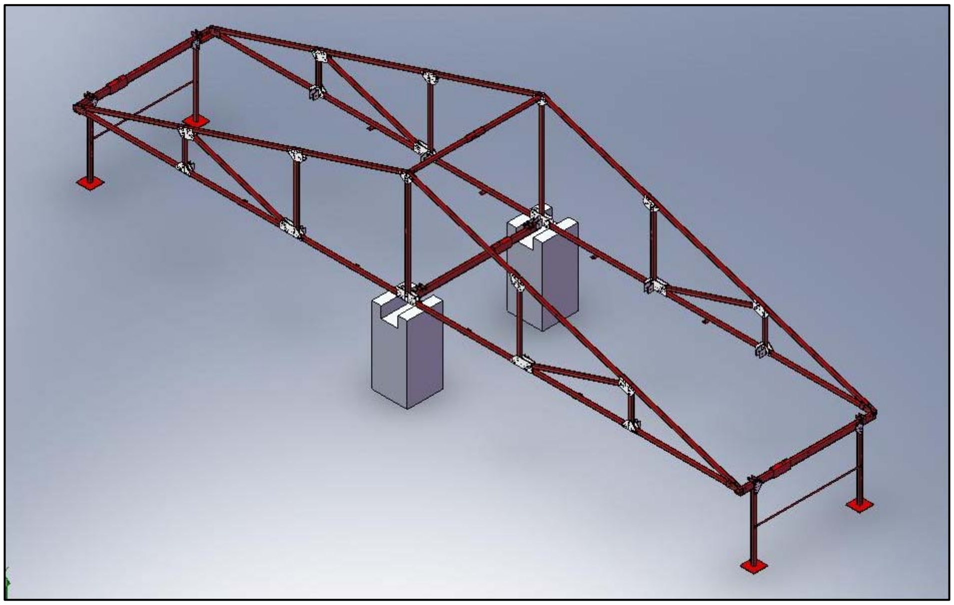


Fig. 6.11. Assembly #8 (truss panel components 2)

Stage 7

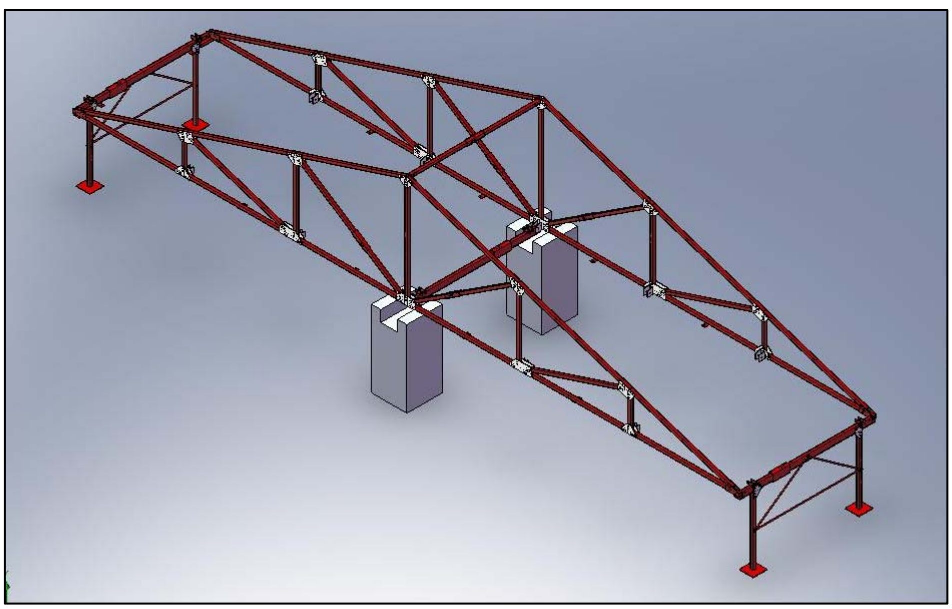


Fig. 6.12. Diagonal splices and triangular support bracing

Stage 8

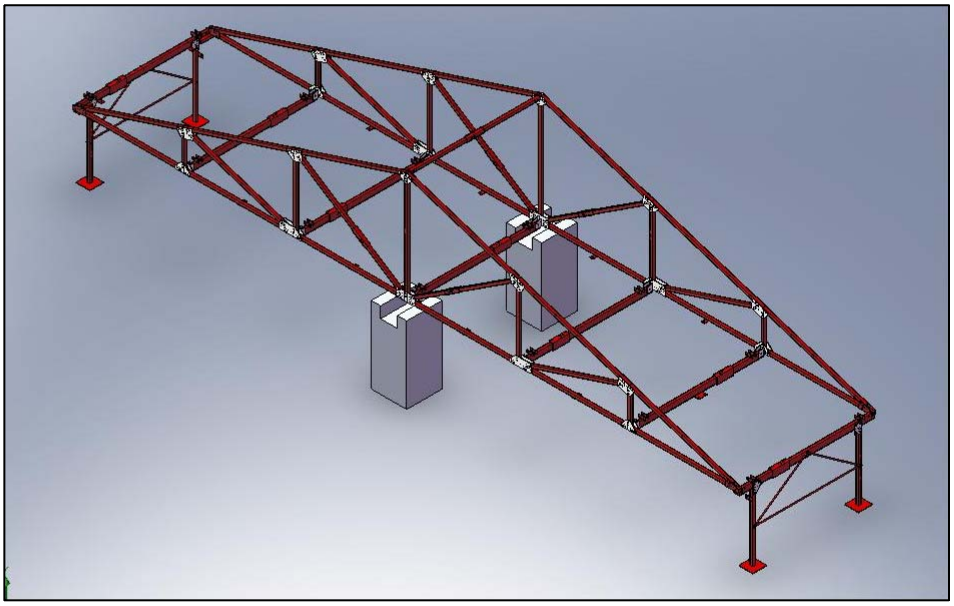


Fig. 6.13. Interior floor beams

Stage 9

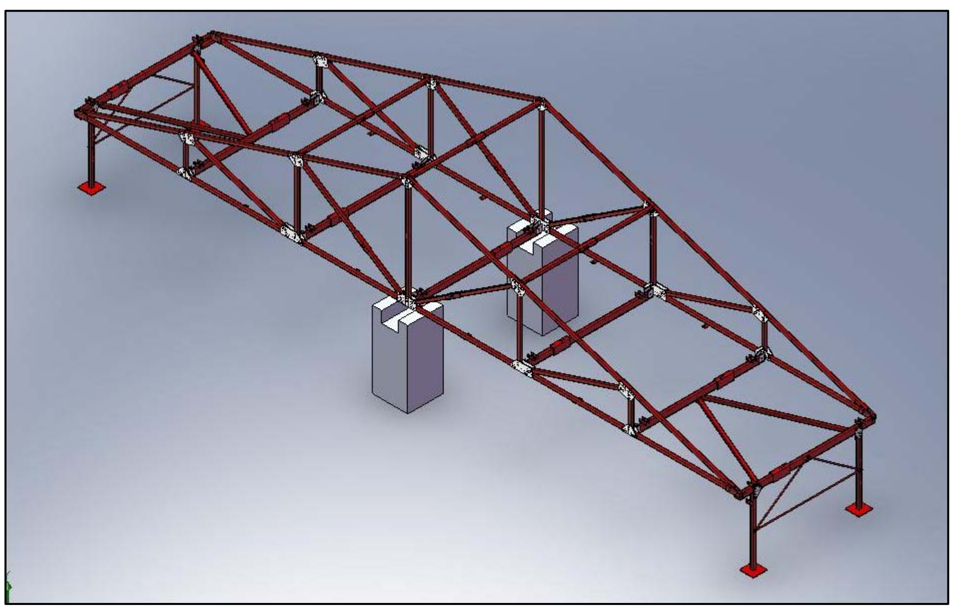


Fig. 6.14. #11-Members (eccentric deck bracing) and remaining top cross braces.

Stage 10

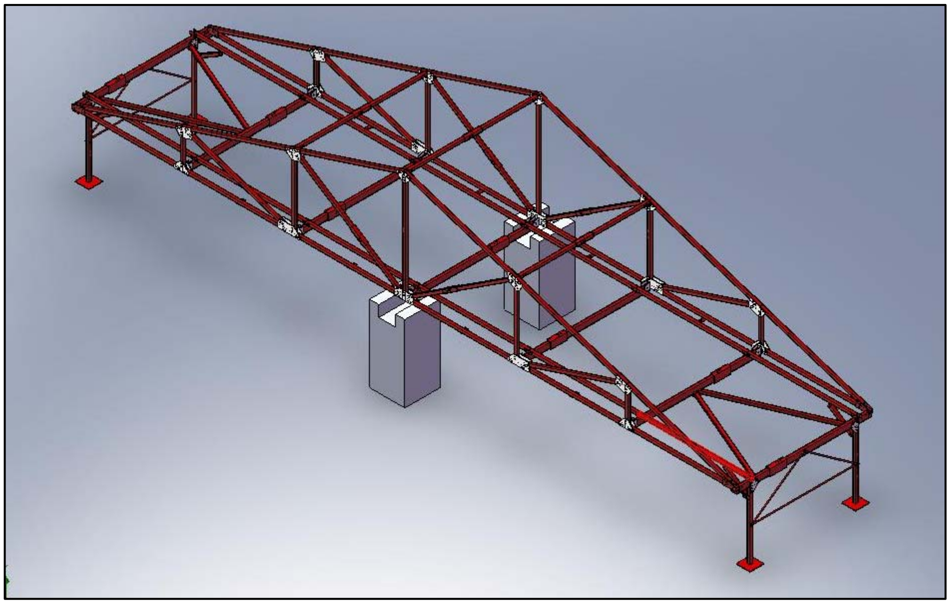


Fig. 6.15. # 12 (inside stringers), 13 (outside stringers), and 14-Members (knee support-braces)

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7.0. Lessons Learned

This project offered many opportunities to correct misconceptions about the full design projects and learn from experienced competitors. From personal experience, limitations of the design software were discovered, the problems with exact drawing were learned with respect to allowing clearances, and accurate construction time estimate techniques were learned. From experienced competitors, rule interpretations and different design philosophies were observed. In conclusion, these experiences have raised our awareness of many obstacles overlooked in the design process.

7.1. Before the Competition

7.1.1. Software

When AutoCAD drawings were finished, we intended to use them to create a 3D SolidWorks model as a final check to the design. While most of these SolidWorks members were formed using a set of concentric mates, any set of holes that were made at an angle were impossible to mate. The geometries from

AutoCAD to the SolidWorks models would match to the 5th decimal place, yet the two holes in each mating part would still refuse to line up with concentric mates. The best solution to this problem was to set the HSS's length perpendicular to the gussets edge that was perpendicular to the direction of the holes. In the case of deck and portal bracing, the members were mated at one end and just rotated into place.

7.1.2. Fabrication

While the original design called for 5/16 in. diameter holes for the splices, these turned out to be too small to accommodate the bolts when the two members forming the splice were connected. The holes had originally been reamed to slightly larger than 5/16 in. diameter to allow for a relatively tight fit. Since this made for cumbersome assembly, we resolved to re-drill these holes to 21/64 in. diameter since we found the connection was comparably stiff with this extra clearance.

The hole diameter drilled for the gusset plates was 21/64 in., which was found to provide sufficient clearance for quick assembly. However, the holes through individual members had to be drilled at 11/32 in. diameter due to variation in hole locations caused by a less-than-optimal production runs. Ideally, each member would have been cut to the same length within a lower tolerance (~1/64") so that hole locations on each member could be reliable measured from cut ends. Since we could not achieve this accuracy, we compensated by drilling larger holes. The additional clearance in these holes was largely responsible for the larger deflections observed in the competition than predicted in our models.

When initially installing the stringers, it was discovered that their bolt holes were also misaligned with their connecting tabs. This made it nearly impossible to install the stringers. Our solution was to drill the holes in the stringers to have a 3/8 in. diameter rather than the existing 11/32 inch diameter. Since the stringers were simply supported, excess hole clearance was unimportant.

In making the splices for the diagonal chords and the top bracing, the inside of the sleeve and the outside of the connecting pieces had to be sanded.

There was a slight edge from the manufacturing process of the sleeve that prevented the smaller piece from sliding in. In making the floor beams, the sleeve available was too big to tightly hold the splice members. To solve this problem, tabs of scrap 1/8 in. steel were welded to the edges of the splice members.

Finally, some of the welds that connected joints L1 and L7 to their corresponding members caused the gussets to warp, complicating the installation of members into those joints. The ends of these members were sanded down in an attempt to compensate for the reduced space, however this was not effective. The most effective solution was to physically pry the warped gusset plates apart.

7.1.3. Construction

When construction practices were begun, it became obvious that the tension in the bolts that connected the gussets to their corresponding members affected ease of construction. Tightening these bolts reduced the available clearance between the gusset plate, making it more difficult to slide other members in for connection. The solution to this was to install washers in between the gusset plates and the members they were connected to. This allowed the gussets to be legally connected while increasing the gap between the gusset plates, allowing for easy installation of other members into these joints.

Installation of the portal bracing proved to be extremely difficult to do quickly during timed construction. While three of the four braces could be installed easily, the fourth required a monumental effort. After determining the bridge had sufficient lateral stiffness without the portal bracing and interior deck bracing, the solution was to simply leave these components out in the competition to decrease our construction time. This was a risk we chose to take under the assumption that lateral stability was sufficient if the joint bolts were tightened.

7.2. After the Competition

7.2.1. Improvements to the Joint Design

The biggest problem with the joint design was the number of bolts used. Over 200 bolts were used to construct the bridge while competitor's bridges typically used less than 70. Having joints that require one bolt per member while still maintaining a working center would make the bridge more competitive. A possible solution is to have guides that will not only put the member in its final orientation but also maintain the proper working center. The most successful of the competitor's bridges could have been built without any bolts, using them only as an accessory to satisfy competition rules.

Another problem with the joint design was that the gusset plates bent under the bridge's self weight during construction. This in turn made it more difficult to put members and assemblies together. Stiffer gusset plates in the form of thicker gusset plates could have helped in maintaining proper alignment and as such reduced the struggle in lining up the bolt openings.

Using shorter bolts would have decreased the amount of time spent during installation. Shorter bolts could be tightened faster, allowing builders to work on other parts of the bridge sooner.

Finally, allowing the stringers to be connected together to form an assembly and allowing that assembly to rest in location designated by holding tabs would have also reduced our construction time.

7.2.2. Improvements to the Construction Plan

A more distinct labeling system could have prevented mistakes made during the timed construction portion of the competition. If the bridge could have been constructed in less than 30 minutes without these mistakes, the total construction cost could have been reduced by $9 million.

It also became evident that the position of the barges was important. Rather than keeping the barges outside of the constructed portion of the bridge, some of our competitor's positioned them within their structure throughout the duration of construction. As a result, the floor beams could be installed early, allowing them to help support the truss panels during construction. Another solution to this problem of truss support is to design the piers such that they hold the vertical members in place and thus reduce the need for the middle floor beam and top bracing early in construction.

Another improvement would be to have a tool to aid in supporting assemblies from a distance. This would have allowed twice the amount of work to be done by having a builder and a barge holding an assembly rather than having both barges hold the same assembly, freeing one barge to work on other parts of the bridge.

Finally, installing half of the assemblies piece by piece could have increased the barges productivity by reducing their inactive time. There were several times when the barges could have been installing single members rather than waiting for builders to bring them assemblies that take longer to assemble in staging yards.

8.0. References

AISC Manual Committee. LRFD Manual of Steel Construction (3rd edition). American Institute of Steel Construction.

Anonymous (2008). Student Steel Bridge Competition: 2008 Rules. ASCE and AISC, <http://www.aisc.org/Content/ContentGroups/Documents/University_Relations3/2008Rules.pdf> .

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Tonias, Demetrios E. and Zhao, Jim J.(2006). Bridge Engineering. McGraw Hill, New York, NY.

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9.0. Appendices

Appendix A: ANSYS Finite Element Analysis Input File

Appendix B: ANSYS Finite Element Analysis Output

Appendix C: Student Steel Bridge Competition – 2008 Rules

Appendix D: AutoCAD Drawings

Appendix A: ANSYS Finite Element Analysis Input File

```
!Final Steel Bridge Model
!Analyst:  Chris Caruso
!Date: April 15th, 2008
/Title,Steel Bridge Model
/output,FINALOUT.txt

!Kan,0   !Static analysis

/prep7  ! Entering pre-processor

nlgeom,off          !Account for large deflection
!Define gravity
acel,0,386,0

!Define Element types to be used in analysis0
!Units: Inch, Kips

et,1,beam44 ! 3D Beam element
et,3,link8     ! Tension-Member



!--Define Real Constants for Grating----!
!TK(I), TK(J), TK(K), TK(L), THETA,
ADMSUA
r,2,.5,.5,.5,.5,0,0
r,3,.25,.25,.25,.25,0,0
r,4,.2431,0          !hss area
r,5,.049087          !1/4 bar area



!Define material properties
MP,EX,1,29e3  ! Beam properties
MP,nuxy,1,0.30
MP,DENS,1,7.34e-7

MP,EX,2,29e4  ! Beam properties
MP,nuxy,1,0.30
MP,DENS,1,7.34e-7



!****** Define Rect. 1" x .065" wall Section
properties (.827 lb/ft) *******!
sectype,1,beam,hrec,,0
secdata,1,1,.065,.065,.065,.065

!****** Define Rect. 1.5" x .065" wall Section
properties (1.048 lb/ft) *******!
sectype,2,beam,hrec,,0
secdata,1.5,1.5,.065,.065,.065,.065
secoffset,user,1,.75

!****** Define Rect. 2" x 1" x .110" wall
Section properties (1.6 lb/ft) *******!
sectype,3,beam,hrec,,0
secdata,2,1,.110,.110,.110,.110
secoffset,user,1.5,.5

!****** Define 1/4" Bars (.28 lb/ft) *******!
sectype,4,beam,csolid,,0
secdata,.125,8,2


!~~ ~~~~~**************~~~~~~~~~~~~~~~
!------------------Define Nodes
!~~~~~~~~***************~~~~~~~~~~~~~~

!****** Bottom Chord & Floor Beam Nodes
*********!
NODEnum=1
counter=7
*dowhile,counter
        n,NODEnum,(7-counter)*40,0,0
        n,(NODEnum+7),(7-counter)*40,0,5
        n,(NODEnum+14),(7-counter)*40,0,17
        n,(NODEnum+21),(7-counter)*40,0,24
        n,(NODEnum+28),(7-counter)*40,0,31
        n,(NODEnum+35),(7-counter)*40,0,43
        n,(NODEnum+42),(7-counter)*40,0,48
        NODEnum=NODEnum+1
        counter=counter-1
*enddo

!****** Bottom Chord Midpoint Nodes (for
deck bracing) ******!
counter=4
NODEnum=61
*dowhile,counter
        n,NODEnum,40+(4-counter)*40+20,0,0
        n,(NODEnum+4),40+(4-
counter)*40+20,0,48
        counter=counter-1
        NODEnum=NODEnum+1
*enddo

!****** Stringer Nodes *********!
NODE=301
counter=31
*dowhile,counter
        n,(NODE),(31-counter)*8,1,5
        n,(NODE+100),(31-counter)*8,1,5
        !Moment Release Node
        n,(NODE+31),(31-counter)*8,1,43
```

```
	n,(NODE+131),(31-counter)*8,1,43
	!Moment Release Node
	NODE=NODE+1
	counter=counter-1
*enddo


!****** Middle Plate Nodes (143" - 178") 
***********!
S2=1				!Random Die Roll
MPC = 115+21+7*S2	!Define Center of 
Middle Plate
DECKH=1.75	!Defines how high deck must 
sit
n,1001,MPC,DECKH,1.5
n,1002,MPC,DECKH,3.25
n,1003,MPC,DECKH,5
n,1050,MPC,DECKH,17
n,1004,MPC,DECKH,24
n,1051,MPC,DECKH,31
n,1005,MPC,DECKH,43
n,1006,MPC,DECKH,44.75
n,1007,MPC,DECKH,46.5


n,1011,(MPC-21),DECKH,1.5
n,1012,(MPC-21),DECKH,5.75
n,1013,(MPC-21),DECKH,5
n,1052,(MPC-21),DECKH,17
n,1014,(MPC-21),DECKH,24
n,1053,(MPC-21),DECKH,31
n,1015,(MPC-21),DECKH,43
n,1016,(MPC-21),DECKH,44.75
n,1017,(MPC-21),DECKH,46.5

n,1021,(MPC+21),DECKH,1.5
n,1022,(MPC+21),DECKH,3.25
n,1023,(MPC+21),DECKH,5
n,1054,(MPC+21),DECKH,17
n,1024,(MPC+21),DECKH,24
n,1055,(MPC+21),DECKH,31
n,1025,(MPC+21),DECKH,43
n,1026,(MPC+21),DECKH,44.75
n,1027,(MPC+21),DECKH,46.5

!--Additional Plate Midpoint Nodes--!
n,1031,(MPC-10.5),DECKH,1.5
n,1032,(MPC-10.5),DECKH,5
n,1033,(MPC-10.5),DECKH,38
n,1034,(MPC-10.5),DECKH,46.5
n,1035,(MPC-10.5),DECKH,24

n,1041,(MPC+10.5),DECKH,1.5
n,1042,(MPC+10.5),DECKH,10
n,1043,(MPC+10.5),DECKH,38
n,1044,(MPC+10.5),DECKH,46.5
n,1045,(MPC+10.5),DECKH,24

!--Couple Nodes--!
n,113,(MPC-21),1,5
n,132,(MPC-10.5),1,5
n,103,MPC,1,5
n,142,(MPC+10.5),1,5
n,123,(MPC+21),1,5

n,115,(MPC-21),1,43
n,133,(MPC-10.5),1,43
n,105,MPC,1,43
n,143,(MPC+10.5),1,43
n,125,(MPC+21),1,43


!****** SidePlate Nodes (68" - 103") 
***********!
S1=6				!Random Die Roll
SPC = 40+21+7*S1	!Define Center of 
Middle Plate
DECKH=1.75	!Defines how high deck must 
sit
n,2001,SPC,DECKH,1.5
n,2002,SPC,DECKH,3.25
n,2003,SPC,DECKH,5
n,2050,SPC,DECKH,17
n,2004,SPC,DECKH,24
n,2051,SPC,DECKH,31
n,2005,SPC,DECKH,43
n,2006,SPC,DECKH,44.75
n,2007,SPC,DECKH,46.5


n,2011,(SPC-21),DECKH,1.5
n,2012,(SPC-21),DECKH,3.25
n,2013,(SPC-21),DECKH,5
n,2052,(SPC-21),DECKH,17
n,2014,(SPC-21),DECKH,24
n,2053,(SPC-21),DECKH,31
n,2015,(SPC-21),DECKH,43
n,2016,(SPC-21),DECKH,44.75
n,2017,(SPC-21),DECKH,46.5

n,2021,(SPC+21),DECKH,1.5
n,2022,(SPC+21),DECKH,3.25
n,2023,(SPC+21),DECKH,5
n,2054,(SPC+21),DECKH,17
n,2024,(SPC+21),DECKH,24
n,2055,(SPC+21),DECKH,31
n,2025,(SPC+21),DECKH,43
n,2026,(SPC+21),DECKH,44.75
n,2027,(SPC+21),DECKH,46.5

!--Additional Plate Midpoint Nodes--!
n,2031,(SPC-10.5),DECKH,1.5
```

```
n,2032,(SPC-10.5),DECKH,5
n,2033,(SPC-10.5),DECKH,43
n,2034,(SPC-10.5),DECKH,46.5
n,2035,(SPC-10.5),DECKH,24

n,2041,(SPC+10.5),DECKH,1.5
n,2042,(SPC+10.5),DECKH,5
n,2043,(SPC+10.5),DECKH,43
n,2044,(SPC+10.5),DECKH,46.5
n,2045,(SPC+10.5),DECKH,24

!--Couple Nodes--!
n,213,(SPC-21),1,5
n,232,(SPC-10.5),1,5
n,203,SPC,1,5
n,242,(SPC+10.5),1,5
n,223,(SPC+21),1,5

n,215,(SPC-21),1,43
n,233,(SPC-10.5),1,43
n,205,SPC,1,43
n,243,(SPC+10.5),1,43
n,225,(SPC+21),1,43


!****** Truss Nodes ********!
n,501,40,13,0
n,502,80,26,0
n,512,112.253,36.4375,0
n,503,120,39,0 $n,521,120,39,7.875
n,513,127.747,36.4375,0
n,504,160,26,0
n,505,200,13,0


n,506,40,13,48
n,507,80,26,48
n,517,112.253,36.4375,48
n,508,120,39,48 $n,522,120,39,40.125
n,518,127.747,36.4375,48
n,509,160,26,48
n,510,200,13,48

!*** Support System Nodes ***!
n,601,0,-26,5 $n,611,0,-12,5 $n,621,0,0,5
$n,631,0,-6.625,5
n,602,0,-26,43 $n,612,0,-12,43 $n,622,0,0,43
$n,632,0,-6.625,43
n,603,240,-26,5 $n,613,240,-12,5 $n,623,240,0,5
$n,633,240,-6.625,5
n,604,240,-26,43 $n,614,240,-12,43
$n,624,240,0,43 $n,634,240,-6.625,43

!--Pads--!
n,701,2,-26,4 $n,702,2,-26,5 $n,703,2,-26,6
$n,704,0,-26,6 $n,705,-2,-26,6 $n,706,-2,-26,5
$n,707,-2,-26,4
n,709,2,-26,3 $n,710,2,-26,2 $n,711,0,-26,2
$n,712,-2,-26,2 $n,713,-2,-26,3

n,721,2,-26,44 $n,722,2,-26,45 $n,723,2,-26,46
$n,724,0,-26,46 $n,725,-2,-26,46 $n,726,-2,-
26,45 $n,727,-2,-26,44
n,729,2,-26,43 $n,730,2,-26,42 $n,731,0,-26,42
$n,732,-2,-26,42 $n,733,-2,-26,43

n,761,242,-27,4 $n,762,242,-26,5 $n,763,242,-
26,6 $n,764,240,-26,6 $n,765,238,-26,6
$n,766,238,-26,5 $n,767,238,-26,4
n,769,242,-27,3 $n,770,242,-26,2 $n,771,240,-
26,2 $n,772,238,-26,2 $n,773,238,-26,3

n,781,242,-26,44 $n,782,242,-26,45 $n,783,242,-
26,46 $n,784,240,-26,46 $n,785,238,-26,46
$n,786,238,-26,45 $n,787,238,-26,44
n,789,242,-26,43 $n,790,242,-26,42 $n,791,240,-
26,42 $n,792,238,-26,42 $n,793,238,-26,43


!*** Couple Nodes ***!

!--Stringers to Floor Beams--!
counter=7
sNODE=301
fbNODE=8
*dowhile,counter
	cp,next,ux,(sNODE),(fbNODE),(sNOD
E+100)
$cp,next,uy,(sNODE),(fbNODE),(sNODE+100)
$cp,next,uz,(sNODE),(fbNODE),(sNODE+100)
$cp,next,rotx,(sNODE),(fbNODE),(sNODE+100
)
$cp,next,roty,(sNODE),(fbNODE),(sNODE+100
)
	cp,next,ux,(sNODE+31),(fbNODE+28),
(sNODE+131)
$cp,next,uy,(sNODE+31),(fbNODE+28),(sNOD
E+131)
$cp,next,uz,(sNODE+31),(fbNODE+28),(sNOD
E+131)
$cp,next,rotx,(sNODE+31),(fbNODE+28),(sNO
DE+131)
$cp,next,roty,(sNODE+31),(fbNODE+28),(sNO
DE+131)
!
	cp,next,all,(sNODE),(fbNODE),(sNOD
E+100)
!
	cp,next,all,(sNODE+31),(fbNODE+28),
(sNODE+131)
```

```
            sNODE=sNODE+5
            fbNODE=fbNODE+1
            counter=counter-1
*enddo


!--Plates to Stringers --!
cp,next,ux,2013,213 $cp,next,uy,2013,213
$cp,next,uz,2013,213
cp,next,ux,2015,215 $cp,next,uy,2015,215
cp,next,uy,2023,223 $cp,next,uz,2023,223
cp,next,uy,2025,225

cp,next,ux,1023,123 $cp,next,uy,1023,123
$cp,next,uz,1023,123
cp,next,ux,1025,125 $cp,next,uy,1025,125
cp,next,uy,1013,113 $cp,next,uz,1013,113
cp,next,uy,1015,115



!**** Define Elements ****!

type,1 $ mat,1 $ secnum,1

!-- Bottom Chord --!
e,1,2 $e,2,61 $e,61,3 $e,3,62 $e,62,4 $e,4,63
$e,63,5 $e,5,64 $e,64,6 $e,6,7
e,43,44 $e,44,65 $e,65,45 $e,45,66 $e,66,46
$e,46,67 $e,67,47 $e,47,68 $e,68,48 $e,48,49


!-- Floor Beams --!
type,1 $secnum,3 $real,null

e,1,8  $e,8,15 $e,15,22 $e,22,29 $e,29,36
$e,36,43

NODE=2
counter=5
*dowhile,counter
            e,(NODE),(NODE+7)
$e,(NODE+7),(NODE+14)
$e,(NODE+14),(NODE+21)
$e,(NODE+21),(NODE+28)
$e,(NODE+28),(NODE+35)
$e,(NODE+35),(NODE+42)
            NODE=NODE+1
            counter=counter-1
*enddo

e,7,14  $e,14,21 $e,21,28 $e,28,35 $e,35,42
$e,42,49

!-- Stringers --!
secnum,1
e,301,302 $e,302,303 $e,303,304 $e,304,305
$e,305,406 $e,306,307 $e,307,308 $e,308,309
$e,309,310 $e,310,411
e,332,333 $e,333,334 $e,334,335 $e,335,336
$e,336,437 $e,337,338 $e,338,339 $e,339,340
$e,340,341 $e,341,442

e,311,213 $e,213,312
e,342,215 $e,215,343

e,312,313 $e,313,314 $e,314,315 $e,315,416
e,343,344 $e,344,345 $e,345,346 $e,346,447


e,316,113 $e,113,223 $e,223,317
e,347,115 $e,115,225 $e,225,348

e,317,318 $e,318,319 $e,319,320 $e,320,421
e,348,349 $e,349,350 $e,350,351 $e,351,452

e,321,123 $e,123,322
e,352,125 $e,125,353

e,322,323 $e,323,324 $e,324,325 $e,325,426
$e,326,327 $e,327,328 $e,328,329 $e,329,330
$e,330,431
e,353,354 $e,354,355 $e,355,356 $e,356,457
$e,357,358 $e,358,359 $e,359,360 $e,360,361
$e,361,462


!--- Trusses ---!
type,1 $ mat,1 $ secnum,1 $real,null

!-Left truss
e,1,501 $e,501,502 $e,502,512 $e,512,503
$e,503,513 $e,513,504 $e,504,505 $e,505,7
type,3 $real,4
e,501,2 $e,501,3 $e,502,3 $e,502,4 $e,503,4
$e,504,4 $e,504,5 $e,505,5 $e,505,6

!-Right truss
type,1 $ secnum,1 $real,null
e,43,506 $e,506,507 $e,507,517 $e,517,508
$e,508,518 $e,518,509 $e,509,510 $e,510,49
type,3 $real,4
e,506,44 $e,506,45 $e,507,45 $e,507,46
$e,508,46 $e,509,46 $e,509,47 $e,510,47
$e,510,48

!--- Top Crosses ---!
type,1 $secnum,1 $real,null
e,502,507
e,503,521 $e,521,522 $e,522,508
e,504,509
```

```
!-- Portal Bracing
type,3 $real,4
e,512,521 $e,513,521
e,517,522 $e,518,522

!-- Deck Chevrons ---!
e,1,23 $e,43,23
e,61,24 $e,65,24
e,62,25 $e,66,25
e,63,25 $e,67,25
e,64,26 $e,68,26
e,7,27 $e,49,27


!---Support System ---!
type,1 $real,null $mat,1 $secnum,1
e,601,611 $e,611,631 $e,631,8
e,602,612 $e,612,632 $e,632,36
e,603,613 $e,613,633 $e,633,14
e,604,614 $e,614,634 $e,634,42

type,3 $real,5 $mat,1
e,611,612 $e,613,614
e,611,22 $e,612,22
e,613,28 $e,614,28

!--- Knee Bracing ---!
type,3 $real,4
e,631,302 $e,632,333 $e,633,330 $e,634,361

!--Pads---!
type,2 $real,3
!e,701,703,705,707,702,704,706,601
$e,710,701,707,712,709,601,713,711
!e,721,723,725,727,722,724,726,602
$e,730,721,727,732,729,602,733,731
!e,781,783,785,787,782,784,786,604
$e,790,781,787,792,789,604,793,791
!e,761,763,765,767,762,764,766,603
$e,770,761,767,772,769,603,773,771


!---Apply Loads-----!
counter=16
ELEM=127
*dowhile,counter
!           sfe,ELEM,2,pres,0,-.6878e-3,-.6878e-
3,-.6878e-3,-.6878e-3
            counter=counter-1
            ELEM=ELEM+1
*enddo
!.5943 for 2.6 kips reaction

f,213,fy,-.413 $f,215,fy,-.413 $f,316,fy,-.237
$f,347,fy,-.237
f,113,fy,-.361 $f,115,fy,-.361 $f,321,fy,-.289
$f,352,fy,-.289

!*** Define Supports *****!
!d,710,uy,0  $d,702,uy,0  $d,704,uy,0
$d,706,uy,0  $d,710,uy,0 $d,711,uy,0
$d,713,uy,0
!d,701,uy,0 $!d,703,uy,0 $!d,705,uy,0
$!d,707,uy,0 $!d,712,uy,0 $!d,709,uy,0
!d,710,ux,0
!d,710,uz,0

!d,730,uy,0  $d,722,uy,0  $d,724,uy,0
$d,726,uy,0  $d,729,uy,0  $d,731,uy,0
$d,732,uy,0 $d,723,uy,0
!d,721,uy,0 $!d,730,uy,0
$!d,723,uy,0$!d,725,uy,0 $!d,727,uy,0
!d,730,ux,0

!d,770,uy,0  $d,762,uy,0  $d,764,uy,0
$d,766,uy,0  $d,771,uy,0  $d,773,uy,0
!d,761,uy,0 $!d,770,uy,0 $!d,763,uy,0
$!d,772,uy,0 $!d,765,uy,0 $!d,767,uy,0
$!d,769,uy,0
!d,770,uz,0

!d,790,uy,0  $d,782,uy,0  $d,784,uy,0
$d,786,uy,0  $d,791,uy,0  $d,793,uy,0
!d,781,uy,0 $!d,790,uy,0 $!d,783,uy,0
$!d,792,uy,0 $!d,785,uy,0 $!d,787,uy,0
$!d,789,uy,0

!--- Simple Constraints---!
d,601,ux,0 $d,601,uy,0 $,601,uz,0
d,602,ux,0 $d,602,uy,0
d,603,uy,0 $d,603,uz,0
d,604,uy,0



save
finish
!/check


/solu
solve
save
finish


/post1
set,1,1
```

```
!nlist,all
elist,all
rlist,all
mplist,all
dlist,all
flist,all

cplist,all

etable,smax,nmisc,1
etable,smax,nmisc,3
etable,smin,nmisc,2
etable,smin,nmisc,4
etable,mforx,smisc,1
etable,mforx,smisc,7
etable,mfory,smisc,2
etable,mfory,smisc,8
etable,mforz,smisc,3
etable,mforz,smisc,9
etable,mmomx,smisc,4
etable,mmomx,smisc,10
etable,mmomy,smisc,5
etable,mmomy,smisc,11
etable,mmomz,smisc,6
etable,mmomz,smisc,13
etable,saxl,ls,1

pretab

save

/pbc,u,,1
!/pbc,cp,,1
!/pnum,node,1
/eshape,1
plnsol,s,x
!eplot

!finish
```

Appendix B: ANSYS Finite Element Analysis Output

```
***** ANSYS - ENGINEERING ANALYSIS SYSTEM  RELEASE 11.0     *****
ANSYS Academic Teaching Introductory
00203023         VERSION=INTEL NT      21:57:37  APR 16, 2008 CP=      2.891

Steel Bridge Model


        ***** ANSYS ANALYSIS DEFINITION (PREP7) *****

ENTER  /SHOW,DEVICE-NAME  TO ENABLE GRAPHIC DISPLAY
ENTER  FINISH            TO LEAVE PREP7
PRINTOUT KEY SET TO /GOPR (USE /NOPR TO SUPPRESS)

SMALL DEFORMATION ANALYSIS

ACEL=    0.0000      386.00      0.0000

ELEMENT TYPE      1 IS BEAM44       3-D ELASTIC TAPERED BEAM
 KEYOPT(1-12)=    0  0  0    0  0  0    0  0  0    0  0  0

CURRENT NODAL DOF SET IS  UX    UY    UZ    ROTX  ROTY  ROTZ
 THREE-DIMENSIONAL MODEL

ELEMENT TYPE      2 IS SHELL93      8-NODE STRUCTURAL SHELL
 KEYOPT(1-12)=    0  0  0    0  0  0    0  0  0    0  0  0

CURRENT NODAL DOF SET IS  UX    UY    UZ    ROTX  ROTY  ROTZ
 THREE-DIMENSIONAL MODEL

ELEMENT TYPE      3 IS LINK8       3-D SPAR  ( OR TRUSS )
 KEYOPT(1-12)=    0  0  0    0  0  0    0  0  0    0  0  0

CURRENT NODAL DOF SET IS  UX    UY    UZ    ROTX  ROTY  ROTZ
 THREE-DIMENSIONAL MODEL

REAL CONSTANT SET       2  ITEMS   1 TO   6
  0.50000      0.50000      0.50000      0.50000       0.0000       0.0000

REAL CONSTANT SET       3  ITEMS   1 TO   6
  0.25000      0.25000      0.25000      0.25000       0.0000       0.0000

REAL CONSTANT SET       4  ITEMS   1 TO   6
  0.24310       0.0000       0.0000       0.0000       0.0000       0.0000

REAL CONSTANT SET       5  ITEMS   1 TO   6
  0.49087E-01   0.0000       0.0000       0.0000       0.0000       0.0000

MATERIAL        1     EX   =   29000.00

MATERIAL        1     NUXY =  0.3000000

MATERIAL        1     DENS =  0.7340000E-06

MATERIAL        2     EX   =   290000.0

MATERIAL        1     NUXY =  0.3000000

MATERIAL        1     DENS =  0.7340000E-06

  INPUT SECTION ID NUMBER              1
  INPUT BEAM SECTION TYPE          Hollow Rectangle
  INPUT BEAM SECTION NAME

  SECTION ID NUMBER IS:         1
  BEAM SECTION TYPE IS:     Hollow Rectangle
  BEAM SECTION NAME IS:
  COMPUTED BEAM SECTION DATA SUMMARY:
```

Area = 0.24310
Iyy = 0.35592E-01
Iyz = 0.14637E-17
Izz = 0.35592E-01
Warping Constant = 0.11167E-04
Torsion Constant = 0.55625E-01
Centroid Y = 0.50000
Centroid Z = 0.50000
Shear Center Y = 0.50000
Shear Center Z = 0.50000
Shear Correction-yy = 0.43954
Shear Correction-yz =-0.32151E-13
Shear Correction-zz = 0.43954

Beam Section is offset to CENTROID of cross section

INPUT SECTION ID NUMBER 2
INPUT BEAM SECTION TYPE Hollow Rectangle
INPUT BEAM SECTION NAME

SECTION ID NUMBER IS: 2
BEAM SECTION TYPE IS: Hollow Rectangle
BEAM SECTION NAME IS:
COMPUTED BEAM SECTION DATA SUMMARY:
Area = 0.37310
Iyy = 0.12831
Iyz =-0.19732E-16
Izz = 0.12831
Warping Constant = 0.49310E-04
Torsion Constant = 0.19846
Centroid Y = 0.75000
Centroid Z = 0.75000
Shear Center Y = 0.75000
Shear Center Z = 0.75000
Shear Correction-yy = 0.43222
Shear Correction-yz = 0.53756E-13
Shear Correction-zz = 0.43222

Beam Section is offset to CENTROID of cross section

BEAM SECTION WITH SECTION ID NUMBER 2 IS OFFSET TO
OFFSET Y = 1.0000
OFFSET Z = 0.75000

INPUT SECTION ID NUMBER 3
INPUT BEAM SECTION TYPE Hollow Rectangle
INPUT BEAM SECTION NAME

SECTION ID NUMBER IS: 3
BEAM SECTION TYPE IS: Hollow Rectangle
BEAM SECTION NAME IS:
COMPUTED BEAM SECTION DATA SUMMARY:
Area = 0.61160
Iyy = 0.96275E-01
Iyz = 0.30358E-17
Izz = 0.30008
Warping Constant = 0.58408E-02
Torsion Constant = 0.23568
Centroid Y = 1.0000
Centroid Z = 0.50000
Shear Center Y = 1.0000
Shear Center Z = 0.50000
Shear Correction-yy = 0.64457
Shear Correction-yz = 0.95503E-14
Shear Correction-zz = 0.23065

Beam Section is offset to CENTROID of cross section

BEAM SECTION WITH SECTION ID NUMBER 3 IS OFFSET TO
OFFSET Y = 1.5000

OFFSET Z = 0.50000

INPUT SECTION ID NUMBER 4
INPUT BEAM SECTION TYPE Circular Solid
INPUT BEAM SECTION NAME

SECTION ID NUMBER IS: 4
BEAM SECTION TYPE IS: Circular Solid
BEAM SECTION NAME IS:
COMPUTED BEAM SECTION DATA SUMMARY:
Area = 0.49049E-01
Iyy = 0.19135E-03
Iyz =-0.25411E-20
Izz = 0.19135E-03
Warping Constant = 0.48036E-38
Torsion Constant = 0.38270E-03
Centroid Y =-0.11052E-17
Centroid Z = 0.35920E-17
Shear Center Y = 0.14099E-17
Shear Center Z = 0.66301E-17
Shear Correction-yy = 0.85691
Shear Correction-yz = 0.13206E-14
Shear Correction-zz = 0.85691

Beam Section is offset to CENTROID of cross section

PRINT ELEMENT TABLE ITEMS PER ELEMENT

***** POST1 ELEMENT TABLE LISTING *****

STAT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT
ELEM	SMAX	SMIN	MFORX	MFORY	MFORZ	MMOMX	MMOMY	MMOMZ	SAXL
1	12.541	2.5777	1.8377	-0.34386E-03	0.16013E-01	0.62076E-01	0.27614	0.0000	7.5595
2	8.1763	3.9108	1.4692	0.14049E-02	0.17430E-01	0.71791E-01	0.87657E-01	0.0000	6.0435
3	6.2089	5.7342	1.4517	0.38583E-02	-0.35767E-02	0.71791E-01	0.16123E-01	0.0000	5.9715
4	6.2108	4.9182	1.3527	-0.39666E-03	0.57506E-02	0.36525E-01	0.23524E-01	0.0000	5.5645
5	5.8022	5.2746	1.3464	0.20567E-02	-0.18710E-02	0.36525E-01	-0.13897E-01	0.0000	5.5384
6	5.9596	4.4607	1.2666	-0.62031E-03	0.18244E-02	-0.50804E-01	0.32047E-01	0.0000	5.2102
7	6.7989	3.6825	1.2740	0.18331E-02	-0.70690E-02	-0.50804E-01	-0.10933	0.0000	5.2407
8	7.8256	3.5330	1.3806	-0.24515E-02	0.38453E-02	-0.58259E-01	0.91561E-01	0.0000	5.6793
9	10.485	1.0215	1.3986	0.18720E-05	-0.17670E-01	-0.58259E-01	-0.26185	0.0000	5.7531
10	12.347	2.1370	1.7606	0.30187E-02	-0.15590E-01	-0.59005E-01	-0.35410	0.0000	7.2422
11	12.541	2.5778	1.8377	-0.34313E-03	-0.16013E-01	-0.62076E-01	-0.27614	0.0000	7.5596
12	8.1764	3.9108	1.4692	0.14048E-02	-0.17430E-01	-0.71791E-01	-0.87661E-01	0.0000	6.0436
13	6.2089	5.7343	1.4517	0.38582E-02	0.35770E-02	-0.71791E-01	-0.16120E-01	0.0000	5.9716
14	6.2108	4.9182	1.3527	-0.39663E-03	-0.57510E-02	-0.36525E-01	-0.23527E-01	0.0000	5.5645
15	5.8022	5.2746	1.3464	0.20568E-02	0.18713E-02	-0.36525E-01	0.13899E-01	0.0000	5.5384
16	5.9596	4.4608	1.2666	-0.62031E-03	-0.18242E-02	0.50804E-01	-0.32044E-01	0.0000	5.2102
17	6.7988	3.6825	1.2740	0.18331E-02	0.70687E-02	0.50804E-01	0.10933	0.0000	5.2407
18	7.8256	3.5331	1.3806	-0.24515E-02	-0.38449E-02	0.58259E-01	-0.91557E-01	0.0000	5.6793
19	10.485	1.0216	1.3986	0.18748E-05	0.17670E-01	0.58259E-01	0.26184	0.0000	5.7531
20	12.347	2.1370	1.7606	0.30187E-02	0.15590E-01	0.59005E-01	0.35410	0.0000	7.2422
21	14.962	-15.224	-0.79950E-01	0.71968	-0.24901	-0.14920	-0.71197	0.0000	-0.13072
22	5.0588	-5.1956	-0.41823E-01	-0.15463	0.15936E-06	-0.22671E-04	-0.40095	0.0000	-0.68383E-01
23	2.5621	-2.6988	-0.41823E-01	-0.15342	0.15936E-06	-0.22671E-04	-0.40095	0.0000	-0.68383E-01
24	5.0588	-5.1956	-0.41824E-01	0.15464	0.15936E-06	-0.22671E-04	-0.40095	0.0000	-0.68384E-01
25	11.284	-11.421	-0.41824E-01	0.15671	0.15936E-06	-0.22671E-04	-0.40095	0.0000	-0.68384E-01
26	3.2264	-3.4878	-0.79950E-01	-0.71881	0.24901	0.14924	0.53308	0.0000	-0.13072
27	6.9372	-6.9419	-0.14165E-02	-0.70182E-02	-0.36854	-0.18427	-1.3056	0.0000	-0.23161E-02
28	1.3060	-1.2807	0.77306E-02	-0.28196E-02	0.10139	0.50699E-01	0.18137	0.0000	0.12640E-01
29	5.0437	-5.0184	0.77306E-02	-0.16066E-02	0.10139	0.50699E-01	0.89112	0.0000	0.12640E-01
30	1.3060	-1.2807	0.77301E-02	0.28192E-02	-0.10139	-0.50695E-01	0.18137	0.0000	0.12639E-01
31	5.5869	-5.5616	0.77301E-02	0.48986E-02	-0.10139	-0.50695E-01	-1.0354	0.0000	0.12639E-01
32	2.8217	-2.8264	-0.14170E-02	0.78858E-02	0.36854	0.18427	0.53708	0.0000	-0.23169E-02
33	8.1774	-8.2079	-0.93273E-02	-0.39947	-0.84345E-01	-0.42173E-01	-0.31411	0.0000	-0.15251E-01
34	6.5930	-6.6162	-0.70788E-02	-0.22888E-02	0.17506E-01	0.87526E-02	-0.33599E-02	0.0000	-0.11574E-01
35	7.2338	-7.2569	-0.70788E-02	-0.10759E-02	0.17506E-01	0.87526E-02	0.11918	0.0000	-0.11574E-01
36	6.5930	-6.6162	-0.70795E-02	0.22888E-02	-0.17506E-01	-0.87530E-02	-0.33585E-02	0.0000	-0.11575E-01
37	7.5509	-7.5740	-0.70795E-02	0.43682E-02	-0.17506E-01	-0.87530E-02	-0.21342	0.0000	-0.11575E-01

38	0.64561	-0.67612	-0.93280E-02	0.40034	0.84345E-01	0.42172E-01	0.10761	0.0000	-0.15252E-01
39	10.007	-10.019	-0.36955E-02	-0.60880	0.75550E-02	0.37776E-02	0.28320E-01	0.0000	-0.60423E-02
40	9.7973	-9.7994	-0.62892E-03	-0.33647E-02	-0.10592E-02	-0.52945E-03	0.36290E-02	0.0000	-0.10283E-02
41	9.8625	-9.8645	-0.62892E-03	-0.21518E-02	-0.10592E-02	-0.52945E-03	-0.37852E-02	0.0000	-0.10283E-02

***** POST1 ELEMENT TABLE LISTING *****

STAT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT
ELEM	SMAX	SMIN	MFORX	MFORY	MFORZ	MMOMX	MMOMY	MMOMZ	SAXL
42	9.7973	-9.7994	-0.62893E-03	0.33647E-02	0.10594E-02	0.52982E-03	0.36304E-02	0.0000	-0.10283E-02
43	9.6872	-9.6893	-0.62893E-03	0.54441E-02	0.10594E-02	0.52982E-03	0.16343E-01	0.0000	-0.10283E-02
44	0.32795	-0.34003	-0.36955E-02	0.60967	-0.75563E-02	-0.37780E-02	-0.94599E-02	0.0000	-0.60423E-02
45	6.9522	-6.9879	-0.10914E-01	-0.31417	0.91600E-01	0.45800E-01	0.33401	0.0000	-0.17845E-01
46	5.3655	-5.3954	-0.91397E-02	-0.22888E-02	-0.17929E-01	-0.89646E-02	0.70299E-02	0.0000	-0.14944E-01
47	5.9836	-6.0135	-0.91397E-02	-0.10759E-02	-0.17929E-01	-0.89646E-02	-0.11848	0.0000	-0.14944E-01
48	5.3656	-5.3954	-0.91390E-02	0.22888E-02	0.17930E-01	0.89648E-02	0.70312E-02	0.0000	-0.14943E-01
49	6.3498	-6.3797	-0.91390E-02	0.43682E-02	0.17930E-01	0.89648E-02	0.22219	0.0000	-0.14943E-01
50	0.63274	-0.66843	-0.10914E-01	0.31504	-0.91600E-01	-0.45800E-01	-0.12399	0.0000	-0.17844E-01
51	6.7632	-6.7700	-0.20805E-02	-0.70392E-02	0.36202	0.18101	1.2787	0.0000	-0.34017E-02
52	1.2068	-1.1850	0.66820E-02	-0.28194E-02	-0.98964E-01	-0.49482E-01	-0.17504	0.0000	0.10926E-01
53	4.8562	-4.8343	0.66820E-02	-0.16065E-02	-0.98964E-01	-0.49482E-01	-0.86779	0.0000	0.10926E-01
54	1.2068	-1.1849	0.66826E-02	0.28194E-02	0.98964E-01	0.49482E-01	-0.17504	0.0000	0.10926E-01
55	5.4019	-5.3801	0.66826E-02	0.48988E-02	0.98964E-01	0.49482E-01	1.0125	0.0000	0.10926E-01
56	2.7571	-2.7639	-0.20800E-02	0.79056E-02	-0.36202	-0.18101	-0.53135	0.0000	-0.34010E-02
57	14.405	-14.660	-0.77971E-01	0.69126	0.24166	0.14484	0.69098	0.0000	-0.12749
58	4.8745	-5.0103	-0.41527E-01	-0.14852	0.76226E-07	0.22319E-06	0.38872	0.0000	-0.67899E-01
59	2.4778	-2.6136	-0.41527E-01	-0.14731	0.76226E-07	0.22319E-06	0.38872	0.0000	-0.67899E-01
60	4.8745	-5.0103	-0.41526E-01	0.14852	0.76226E-07	0.22319E-06	0.38872	0.0000	-0.67897E-01
61	10.855	-10.991	-0.41526E-01	0.15060	0.76226E-07	0.22319E-06	0.38872	0.0000	-0.67897E-01
62	3.1258	-3.3808	-0.77970E-01	-0.69040	-0.24166	-0.14484	-0.51734	0.0000	-0.12749
63	4.1477	-2.0683	0.25274	0.17167E-02	0.12664E-01	0.47367E-01	-0.20971	0.0000	1.0397
64	2.5757	-0.52710	0.24901	-0.91132E-03	0.12664E-01	0.47367E-01	-0.10840	0.0000	1.0243
65	1.1668	0.88185	0.24901	-0.36031E-03	0.12664E-01	0.47367E-01	-0.70889E-02	0.0000	1.0243
66	2.4003	-0.35173	0.24901	0.19070E-03	0.12664E-01	0.47367E-01	0.94222E-01	0.0000	1.0243
67	3.7712	-1.7226	0.24901	0.74171E-03	0.12664E-01	0.47367E-01	0.19553	0.0000	1.0243
68	3.7357	2.1790	0.71894	-0.82651E-03	0.35167E-02	0.63413E-01	-0.46589E-01	0.0000	2.9574
69	3.4024	2.5123	0.71894	-0.27550E-03	0.35167E-02	0.63413E-01	-0.18456E-01	0.0000	2.9574
70	3.2791	2.6357	0.71894	0.27550E-03	0.35167E-02	0.63413E-01	0.96777E-02	0.0000	2.9574
71	3.6124	2.3024	0.71894	0.82651E-03	0.35167E-02	0.63413E-01	0.37811E-01	0.0000	2.9574
72	3.8838	2.0310	0.71894	0.13775E-02	0.35167E-02	0.63413E-01	0.65944E-01	0.0000	2.9574
73	4.1469	-2.0676	0.25274	0.17104E-02	-0.12664E-01	-0.47367E-01	0.20971	0.0000	1.0396
74	2.5752	-0.52655	0.24901	-0.90973E-03	-0.12664E-01	-0.47367E-01	0.10840	0.0000	1.0243
75	1.1671	0.88151	0.24901	-0.35872E-03	-0.12664E-01	-0.47367E-01	0.70891E-02	0.0000	1.0243
76	2.4005	-0.35189	0.24901	0.19228E-03	-0.12664E-01	-0.47367E-01	-0.94221E-01	0.0000	1.0243
77	3.7712	-1.7225	0.24901	0.74329E-03	-0.12664E-01	-0.47367E-01	-0.19553	0.0000	1.0243
78	3.7357	2.1791	0.71894	-0.82651E-03	-0.35167E-02	-0.63413E-01	0.46589E-01	0.0000	2.9574
79	3.4024	2.5124	0.71894	-0.27550E-03	-0.35167E-02	-0.63413E-01	0.18456E-01	0.0000	2.9574
80	3.2791	2.6357	0.71894	0.27550E-03	-0.35167E-02	-0.63413E-01	-0.96778E-02	0.0000	2.9574
81	3.6124	2.3024	0.71894	0.82651E-03	-0.35167E-02	-0.63413E-01	-0.37811E-01	0.0000	2.9574
82	3.8838	2.0310	0.71894	0.13775E-02	-0.35167E-02	-0.63413E-01	-0.65945E-01	0.0000	2.9574

***** POST1 ELEMENT TABLE LISTING *****

STAT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT
ELEM	SMAX	SMIN	MFORX	MFORY	MFORZ	MMOMX	MMOMY	MMOMZ	SAXL
83	14.889	-8.1364	0.82079	-0.39359	0.12681E-02	0.32351E-01	-0.32203E-01	0.0000	3.3763
84	13.129	-6.3760	0.82079	0.19823E-01	0.12681E-02	0.32351E-01	-0.24595E-01	0.0000	3.3763
85	14.889	-8.1364	0.82079	-0.39359	-0.12682E-02	-0.32351E-01	0.32204E-01	0.0000	3.3764
86	13.129	-6.3760	0.82079	0.19823E-01	-0.12682E-02	-0.32351E-01	0.24595E-01	0.0000	3.3764
87	10.727	-3.9747	0.82079	0.20374E-01	0.12681E-02	0.32351E-01	-0.14450E-01	0.0000	3.3763
88	8.2641	-1.5114	0.82079	0.20926E-01	0.12681E-02	0.32351E-01	-0.43049E-02	0.0000	3.3763
89	5.9030	0.84970	0.82079	0.21477E-01	0.12681E-02	0.32351E-01	0.58400E-02	0.0000	3.3763
90	3.6009	3.1518	0.82079	0.22028E-01	0.12681E-02	0.32351E-01	0.15985E-01	0.0000	3.3763
91	10.727	-3.9747	0.82079	0.20374E-01	-0.12682E-02	-0.32351E-01	0.14450E-01	0.0000	3.3764
92	8.2641	-1.5114	0.82079	0.20926E-01	-0.12682E-02	-0.32351E-01	0.43048E-02	0.0000	3.3764
93	5.9030	0.84971	0.82079	0.21477E-01	-0.12682E-02	-0.32351E-01	-0.58405E-02	0.0000	3.3764
94	3.6009	3.1518	0.82079	0.22028E-01	-0.12682E-02	-0.32351E-01	-0.15986E-01	0.0000	3.3764
95	13.356	-6.6738	0.81218	-0.34419	-0.17984E-02	-0.44984E-01	0.24369E-01	0.0000	3.3409

96	12.831	-6.1490	0.81218	0.16948E-01	-0.17984E-02	-0.44984E-01	0.20772E-01	0.0000	3.3409
97	11.770	-5.0879	0.81218	0.17223E-01	-0.17984E-02	-0.44984E-01	0.13578E-01	0.0000	3.3409
98	13.356	-6.6738	0.81218	-0.34419	0.17984E-02	0.44984E-01	-0.24368E-01	0.0000	3.3409
99	12.831	-6.1490	0.81218	0.16948E-01	0.17984E-02	0.44984E-01	-0.20771E-01	0.0000	3.3409
100	11.770	-5.0879	0.81218	0.17223E-01	0.17984E-02	0.44984E-01	-0.13577E-01	0.0000	3.3409
101	9.6237	-2.9419	0.81218	0.17774E-01	-0.17984E-02	-0.44984E-01	-0.80960E-03	0.0000	3.3409
102	7.7973	-1.1155	0.81218	0.18326E-01	-0.17984E-02	-0.44984E-01	-0.15197E-01	0.0000	3.3409
103	5.9089	0.77290	0.81218	0.18877E-01	-0.17984E-02	-0.44984E-01	-0.29585E-01	0.0000	3.3409
104	3.9586	2.7232	0.81218	0.19428E-01	-0.17984E-02	-0.44984E-01	-0.43972E-01	0.0000	3.3409
105	9.6237	-2.9419	0.81218	0.17774E-01	0.17984E-02	0.44984E-01	0.80978E-03	0.0000	3.3409
106	7.7973	-1.1155	0.81218	0.18326E-01	0.17984E-02	0.44984E-01	0.15197E-01	0.0000	3.3409
107	5.9089	0.77291	0.81218	0.18877E-01	0.17984E-02	0.44984E-01	0.29584E-01	0.0000	3.3409
108	3.9586	2.7232	0.81218	0.19428E-01	0.17984E-02	0.44984E-01	0.43971E-01	0.0000	3.3409
109	3.7125	2.0682	0.70265	-0.11020E-02	-0.35730E-02	-0.51437E-01	0.53566E-01	0.0000	2.8904
110	3.5659	2.2148	0.70265	-0.82651E-03	-0.35730E-02	-0.51437E-01	0.39274E-01	0.0000	2.8904
111	3.7125	2.0682	0.70265	-0.11020E-02	0.35730E-02	0.51437E-01	-0.53565E-01	0.0000	2.8904
112	3.5659	2.2148	0.70265	-0.82651E-03	0.35730E-02	0.51437E-01	-0.39274E-01	0.0000	2.8904
113	3.2263	2.5544	0.70265	-0.27550E-03	-0.35730E-02	-0.51437E-01	0.10690E-01	0.0000	2.8904
114	3.3275	2.4532	0.70265	0.27550E-03	-0.35730E-02	-0.51437E-01	-0.17894E-01	0.0000	2.8904
115	3.6671	2.1136	0.70265	0.82651E-03	-0.35730E-02	-0.51437E-01	-0.46478E-01	0.0000	2.8904
116	3.9448	1.8359	0.70265	0.13775E-02	-0.35730E-02	-0.51437E-01	-0.75062E-01	0.0000	2.8904
117	2.3480	-0.35977	0.24166	-0.21188E-03	-0.12336E-01	-0.44960E-01	0.92474E-01	0.0000	0.99409
118	1.1290	0.85922	0.24166	0.33913E-03	-0.12336E-01	-0.44960E-01	-0.62104E-02	0.0000	0.99409
119	2.4891	-0.50093	0.24166	0.89013E-03	-0.12336E-01	-0.44960E-01	-0.10489	0.0000	0.99409
120	4.0064	-2.0183	0.24166	0.14411E-02	-0.12336E-01	-0.44960E-01	-0.20358	0.0000	0.99409
121	5.2552	-3.2372	0.24529	-0.10810E-02	-0.12336E-01	-0.44960E-01	-0.30226	0.0000	1.0090
122	3.2263	2.5544	0.70265	-0.27550E-03	0.35730E-02	0.51437E-01	-0.10690E-01	0.0000	2.8904
123	3.3275	2.4532	0.70265	0.27550E-03	0.35730E-02	0.51437E-01	0.17894E-01	0.0000	2.8904

***** POST1 ELEMENT TABLE LISTING *****

STAT ELEM	CURRENT SMAX	CURRENT SMIN	CURRENT MFORX	CURRENT MFORY	CURRENT MFORZ	CURRENT MMOMX	CURRENT MMOMY	CURRENT MMOMZ	CURRENT SAXL
124	3.6671	2.1136	0.70265	0.82651E-03	0.35730E-02	0.51437E-01	0.46477E-01	0.0000	2.8904
125	3.9448	1.8359	0.70265	0.13775E-02	0.35730E-02	0.51437E-01	0.75061E-01	0.0000	2.8904
126	2.3480	-0.35977	0.24167	-0.21188E-03	0.12336E-01	0.44960E-01	-0.92474E-01	0.0000	0.99410
127	1.1290	0.85923	0.24167	0.33913E-03	0.12336E-01	0.44960E-01	0.62103E-02	0.0000	0.99410
128	2.4891	-0.50093	0.24167	0.89014E-03	0.12336E-01	0.44960E-01	0.10489	0.0000	0.99410
129	4.0065	-2.0183	0.24167	0.14411E-02	0.12336E-01	0.44960E-01	0.20358	0.0000	0.99410
130	5.2552	-3.2372	0.24529	-0.10810E-02	0.12336E-01	0.44960E-01	0.30226	0.0000	1.0090
131	-7.6556	-11.273	-2.3008	-0.64271E-04	0.31010E-02	0.18781E-01	-0.53057E-01	0.0000	-9.4681
132	-8.2281	-10.566	-2.2844	0.30385E-02	0.31010E-02	0.18781E-01	0.77370E-01	0.0000	-9.4008
133	-6.6059	-8.4655	-1.8319	0.72009E-03	-0.32731E-02	-0.45854E-03	-0.47099E-01	0.0000	-7.5387
134	-4.8286	-10.061	-1.8098	0.13472E-01	0.17728E-01	-0.76314E-03	0.97563E-01	0.0000	-7.4455
135	-6.5190	-8.3724	-1.8101	-0.12906E-01	-0.17734E-01	0.98265E-03	-0.47149E-01	0.0000	-7.4450
136	-6.6003	-8.4770	-1.8327	0.15344E-02	0.32298E-02	0.67769E-03	0.62336E-01	0.0000	-7.5357
137	-7.2938	-10.752	-2.1935	-0.22514E-03	-0.30023E-02	-0.18023E-01	-0.51232E-01	0.0000	-9.0194
138	-6.3920	-11.792	-2.2103	0.27368E-02	-0.30023E-02	-0.18023E-01	-0.17751	0.0000	-9.0884
139	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.32748E-01
140	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.63161E-01
141	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.6991
142	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-2.1115
143	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	4.6115
144	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-1.6829
145	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.3490
146	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.65014E-01
147	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.33285E-01
148	-7.6557	-11.273	-2.3008	-0.63678E-04	-0.31010E-02	-0.18781E-01	0.53057E-01	0.0000	-9.4682
149	-8.2281	-10.566	-2.2844	0.30383E-02	-0.31010E-02	-0.18781E-01	-0.77370E-01	0.0000	-9.4008
150	-6.6059	-8.4655	-1.8319	0.72013E-03	0.32731E-02	0.45852E-03	0.47099E-01	0.0000	-7.5387
151	-4.8286	-10.061	-1.8098	0.13472E-01	-0.17728E-01	0.76312E-03	-0.97563E-01	0.0000	-7.4455
152	-6.5190	-8.3725	-1.8101	-0.12906E-01	0.17734E-01	-0.98268E-03	0.47149E-01	0.0000	-7.4450
153	-6.6004	-8.4771	-1.8327	0.15344E-02	-0.32299E-02	-0.67773E-03	-0.62337E-01	0.0000	-7.5357
154	-7.2938	-10.752	-2.1935	-0.22514E-03	0.30023E-02	0.18022E-01	0.51232E-01	0.0000	-9.0194
155	-6.3920	-11.792	-2.2103	0.27368E-02	0.30023E-02	0.18022E-01	0.17751	0.0000	-9.0884
156	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.32757E-01
157	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.63180E-01
158	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.6991
159	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-2.1115

160	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	4.6115
161	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-1.6829
162	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.3490
163	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.65014E-01
164	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.33285E-01

***** POST1 ELEMENT TABLE LISTING *****

STAT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT
ELEM	SMAX	SMIN	MFORX	MFORY	MFORZ	MMOMX	MMOMY	MMOMZ	SAXL
165	0.48724	-0.43480	0.63741E-02	0.16530E-02	0.11480E-07	-0.15287E-06	0.18772E-01	0.0000	0.26220E-01
166	0.53619	-0.24444	0.35462E-01	0.11764E-01	-0.36609E-04	0.59518E-07	-0.21442E-03	0.0000	0.14587
167	0.36356	-0.41706	-0.65029E-02	0.11106E-02	0.13080E-07	0.59518E-07	-0.21400E-03	0.0000	-0.26750E-01
168	1.0310	-0.73925	0.35462E-01	-0.11221E-01	0.36533E-04	0.59518E-07	0.73703E-04	0.0000	0.14587
169	0.46987	-0.41859	0.62321E-02	0.16530E-02	0.11739E-07	0.10670E-07	-0.17839E-01	0.0000	0.25636E-01
170	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.12440
171	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.12418
172	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.12440
173	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.12418
174	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.48640
175	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.48640
176	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.11248
177	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.11249
178	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.40811E-01
179	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.40815E-01
180	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.47621E-01
181	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.47618E-01
182	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.11521
183	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.11520
184	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.47475
185	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.47474
186	-2.9905	-2.9918	-0.72715	0.27435E-05	-0.43438E-06	-0.10012E-17	-0.60814E-05	0.0000	-2.9951
187	0.21747	-7.4542	-0.87962	0.27435E-05	0.50791E-01	0.67082E-16	0.27300	0.0000	-3.6199
188	5.3085	-12.509	-0.87523	-0.37320E-02	0.50791E-01	0.26607E-15	0.60949	0.0000	-3.6021
189	-2.9906	-2.9917	-0.72715	-0.27435E-05	-0.21838E-17	-0.14532E-17	-0.63859E-16	0.0000	-2.9951
190	0.21751	-7.4542	-0.87962	-0.27435E-05	-0.50790E-01	-0.98045E-16	-0.27300	0.0000	-3.6199
191	5.3096	-12.510	-0.87523	-0.37292E-02	-0.50790E-01	-0.39752E-15	-0.60948	0.0000	-3.6022
192	-2.8741	-2.8743	-0.69871	0.0000	0.43438E-06	0.58345E-17	0.60814E-05	0.0000	-2.8781
193	0.20712	-7.1596	-0.84508	0.36637E-14	0.48779E-01	-0.37350E-17	0.26220	0.0000	-3.4778
194	5.1019	-12.019	-0.84078	0.36236E-02	0.48779E-01	0.18442E-16	0.58536	0.0000	-3.4605
195	-2.8742	-2.8742	-0.69871	-0.11102E-15	-0.12124E-16	-0.80461E-17	-0.16466E-15	0.0000	-2.8781
196	0.20709	-7.1596	-0.84508	0.27756E-14	-0.48780E-01	-0.21320E-16	-0.26219	0.0000	-3.4778
197	5.1019	-12.019	-0.84079	0.36236E-02	-0.48780E-01	0.22290E-15	-0.58536	0.0000	-3.4605
198	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-5.9784
199	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-5.7403
200	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	5.8471
201	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	5.8471
202	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	5.6140
203	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	5.6139
204	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.21223E-01
205	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.21176E-01

***** POST1 ELEMENT TABLE LISTING *****

STAT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT	CURRENT
ELEM	SMAX	SMIN	MFORX	MFORY	MFORZ	MMOMX	MMOMY	MMOMZ	SAXL
206	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.20592E-01
207	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	-0.20592E-01

MINIMUM VALUES

ELEM	132	21	148	26	27	27	27	1	148
VALUE	-8.2281	-15.224	-2.3008	-0.71881	-0.36854	-0.18427	-1.3056	0.0000	-9.4682

MAXIMUM VALUES

ELEM	21	13	11	21	32	32	51	1	11
VALUE	14.962	5.7343	1.8377	0.71968	0.36854	0.18427	1.2787	0.0000	7.5596

*** NOTE *** CP = 4.188 TIME= 21:58:04

DELETED BACKUP FILE NAME= file.dbb.

*** NOTE ***   CP =   4.188   TIME= 21:58:04
NEW BACKUP FILE NAME= file.dbb.

ALL CURRENT ANSYS DATA WRITTEN TO FILE NAME= file.db
FOR POSSIBLE RESUME FROM THIS POINT

U   BOUNDARY CONDITION DISPLAY KEY = 1

ELEMENT DISPLAYS USING REAL CONSTANT DATA WITH FACTOR   1.00

DISPLAY NODAL SOLUTION, ITEM=S   COMP=X   AT TOP

EXIT THE ANSYS POST1 DATABASE PROCESSOR

***** ROUTINE COMPLETED ***** CP =   4.422

*** NOTE ***   CP =   4.422   TIME= 21:59:33
A total of 36 warnings and errors written to C:\Documents and
Settings\Chris\file.err.

CLEAR DATABASE AND RERUN START.ANS

RUN SETUP PROCEDURE FROM FILE= C:\Program Files\ANSYS Inc\v110\ANSYS\apdl\start110.ans

ANSYS Academic Teaching Introductory

/INPUT FILE= C:\Program Files\ANSYS Inc\v110\ANSYS\apdl\start110.ans LINE=   0

Current working directory switched to C:\Documents and Settings\Chris\My Documents\E90\ANSYS Analysis\Final Designs\Steel
Bridge

/INPUT FILE= \\Students\students\2008\A-E\ccaruso1\E90\ANSYS Analysis\Final Designs\Steel Bridge\FINAL.txt LINE=   0

TITLE=
Steel Bridge Model

/OUTPUT FILE= FINALOUT.txt