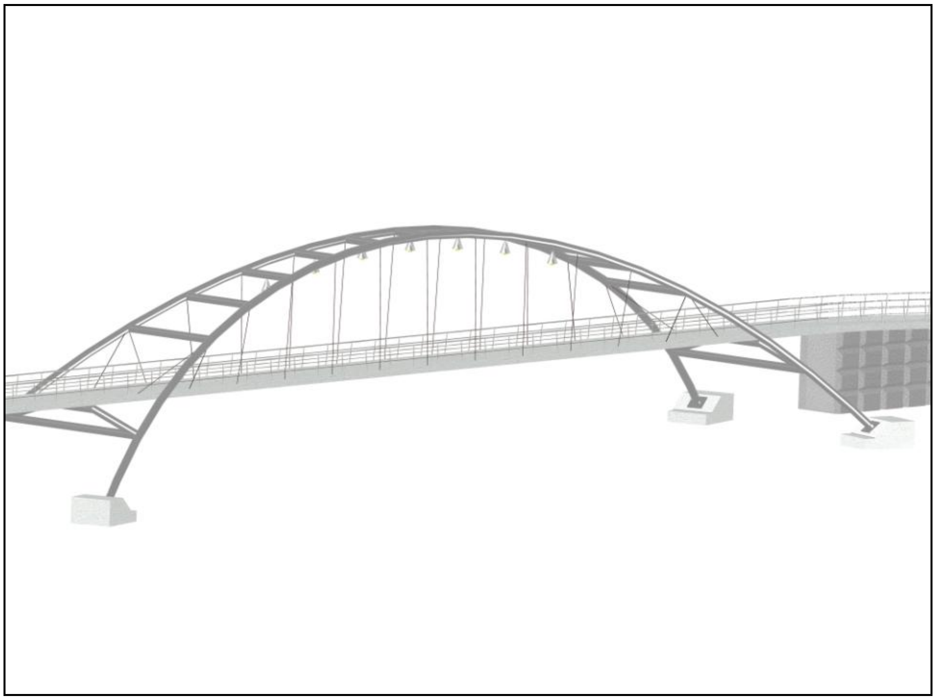
“DESIGN OF A PEDESTRIAN BRIDGE CROSSING OVER COLISEUM BOULEVARD”



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December 11, 2009

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Acknowledgments

The group would like to thank a few people for without them, this project would not have been possible. First, we would like to thank our faculty advisor, Dr. Mohammad Alhassan, for not only his help with the project, but also for preparing us to endeavor on such a project through the multiple structural courses in which he has previously instructed the group. Without the extensive background in structural analysis software that was taught in each of his courses, the group would not have been able to complete such an innovative design such as is proposed in this paper.

Another person whom the group would like to thank is Greg Justice, Senior Project Manager at the IPFW Physical Plant. It was through a meeting with him that the group gathered information on various projects that the campus has, or is planning on pursuing in the future. The information that the group received from him helped guide the group in designing a pedestrian bridge crossing over Coliseum Boulevard which had previously been applied for by the university.

Finally, the group would like to thank Kurt J. Heidenreich, P.E., S.E. taking time out of his busy schedule to meet with the group early on in our design process. Mr. Heidenreich is President/Founder of Engineering Resources, a civil engineering firm here in Fort Wayne. His company is responsible for the design of the two pedestrian bridges that are currently on the IPFW campus: the Willis Family Bridge and the Venderly Family Bridge. The group was able to reap vast amounts of knowledge about pedestrian bridge designs through the meeting with Mr. Heidenreich, and it was through his initial sketch that led the group to their final design.

Abstract

A major obstacle for pedestrians south of the IPFW campus is Coliseum Boulevard: a main arterial for the city of Fort Wayne which has an average daily traffic (ADT) of 50,000 vehicles a day. With this high of an ADT value, crossing by foot can not only be challenging, but it also can be dangerous. Thus, the civil engineering senior design group has proposed to build a pedestrian bridge over Coliseum Boulevard which would allow for easy, safe travel over this busy roadway. Cohering to the innovative design concepts of both the Willis Family Bridge and the Venderly Family Bridge which already exist on the campus, the new structure should be designed so that it too can be transformed into a landmark for the IPFW campus as the other two bridges have become.

1. Section I: Problem Statement

1.1. Problem Statement

The two higher education institutions of Indiana University-Purdue University Fort Wayne (IPFW) and Ivy Tech Community College of Indiana-Northeast have joined together to form the Crossroads partnership, an excellent opportunity that helps students achieve their goal of receiving a college degree faster by allowing the student to enroll in courses at both institutions simultaneously. Since the start of the Crossroads partnership, the number of students participating has steadily grown to the point where now there are 650 students participating in this program.

Also of interest to the city of Fort Wayne, as well as to these two campuses, is the River Greenway Trail; a great design that connects 17 parks into a 20 mile linear park system along the three rivers that Fort Wayne is well known for: the St. Joseph, St. Mary's, and Maumee Rivers. With the campuses of IPFW and Ivy Tech lying on the banks of the St. Joseph River, these campuses have both been integrated into the design of the River Greenway Trail system.

Both of these projects face a common foe, Coliseum Boulevard (State Route 930). This multilane highway is a major route in the city of Fort Wayne which poses great difficulties when trying to cross in a vehicle as well as on foot. The best way to circumvent this problem is by constructing a pedestrian bridge to cross over Coliseum Boulevard which would allow for easy travel back and forth between IPFW and Ivy Tech, as well as to connect the River Greenway Trail to Shoaff Park to the northwest of the IPFW campus. This new bridge should be aesthetically pleasing, completely functional, and within the proposed budget for the project.

1.2. Background

1.2.1. Crossroads Partnership

The Crossroads Partnership, a collaborative effort between Ivy Tech Community College of Indiana-Northeast and IPFW, is an exciting opportunity for students in northeast Indiana. In an attempt to circumvent the normal difficulties students face when transferring credits from one university to another, the two higher learning institutions have worked together to insure that certain courses are completely transferable between the two schools. By doing this, they have made it less likely for students to waste time, credits, and money as they pursue their degree.

A big draw for this program is by allowing students earn a two-year degree at Ivy Tech, and then transferring to IPFW to earn their four-year degree. Another way students can participate in the Crossroads Partnership is by taking classes at one institution and then the other, or the student could even enroll in courses at both institutions at the same time. Allowing students to take classes at both institutions simultaneously, the partnership lets students earn their degree faster than they may have previously expected.

1.2.2. Rivergreenway Trail

Located in Fort Wayne, Indiana, the Rivergreenway Trail is a 20 mile long linear park system that connects 16 parks and other attractions throughout the cities of Fort Wayne and New Haven (Figure 1). The trail is located along the three rivers that the city is well known for: the Saint Mary's, Saint Joseph, and Maumee Rivers. Although the trail is situated in an urban environment, it gives the user the pleasure of many outdoor recreational activities while offering both spectacular natural landscapes and other scenic overlooks along the three rivers. In addition to the recreational use of the trails, the Rivergreenway also creates a natural overflow to assist in holding back the river waters and hence reduce flooding (a problem that has often plagued the city of Fort Wayne).

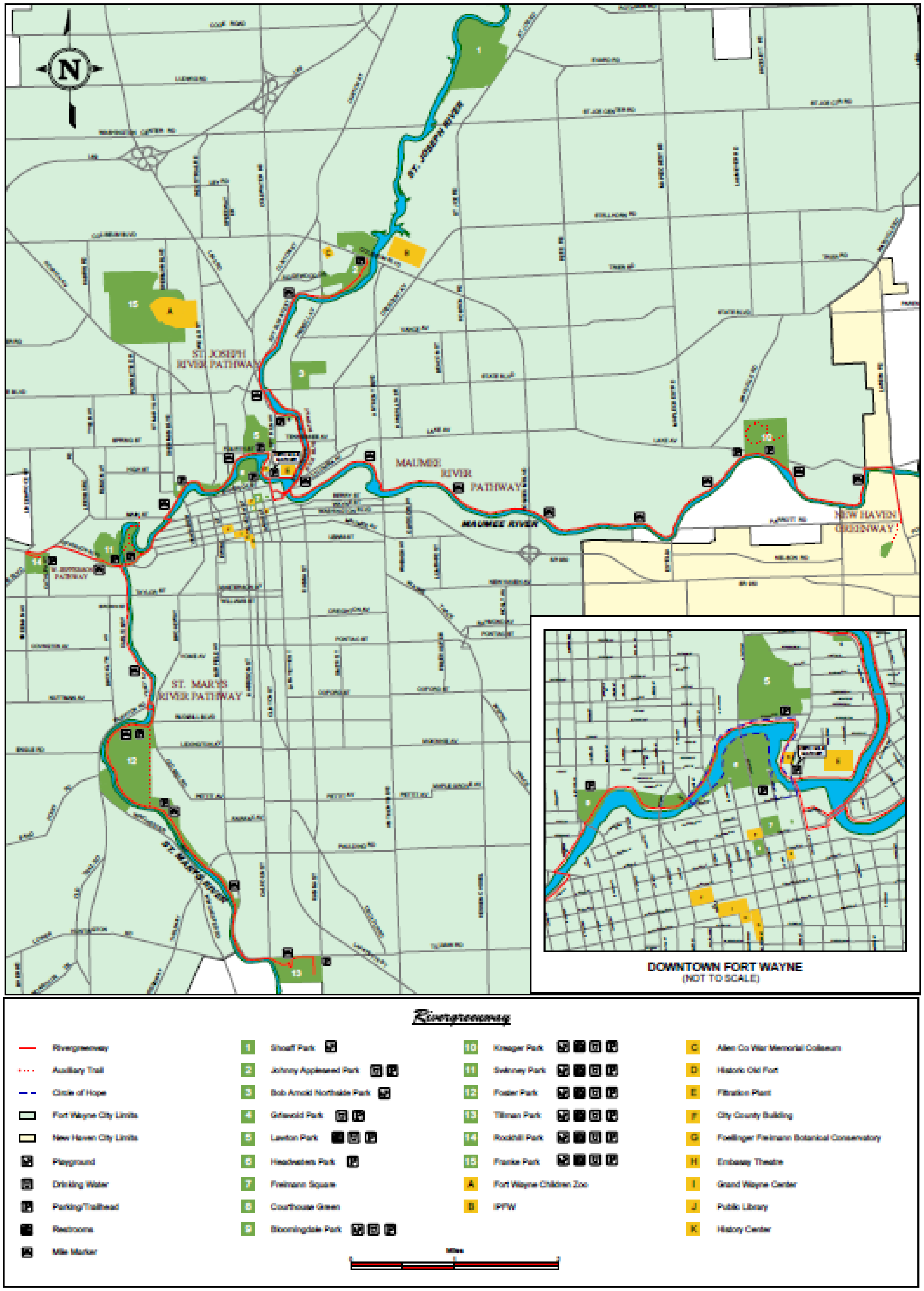

Figure 1. Rivergreenway Trail Map.

Much of the Rivergreenway has been funded by the State of Indiana and the federal Land and Conservation Fund; however, the system is owned and maintained by the Fort Wayne Parks and Recreation Department and the City of Fort Wayne Public Works Department.

As can be seen in Figure 1, the Rivergreenway Trail has yet to cross over Coliseum Boulevard. This has left Shoaff Park to the north of the IPFW campus isolated from the trail system. In the future, the city of Fort Wayne has planned for a crossing at Coliseum in the vicinity of the IPFW campus. The location of this crossing, shown on the "Project Status Map" in Figure 2, would then allow for Shoaff Park to become a part of the linear park chain.

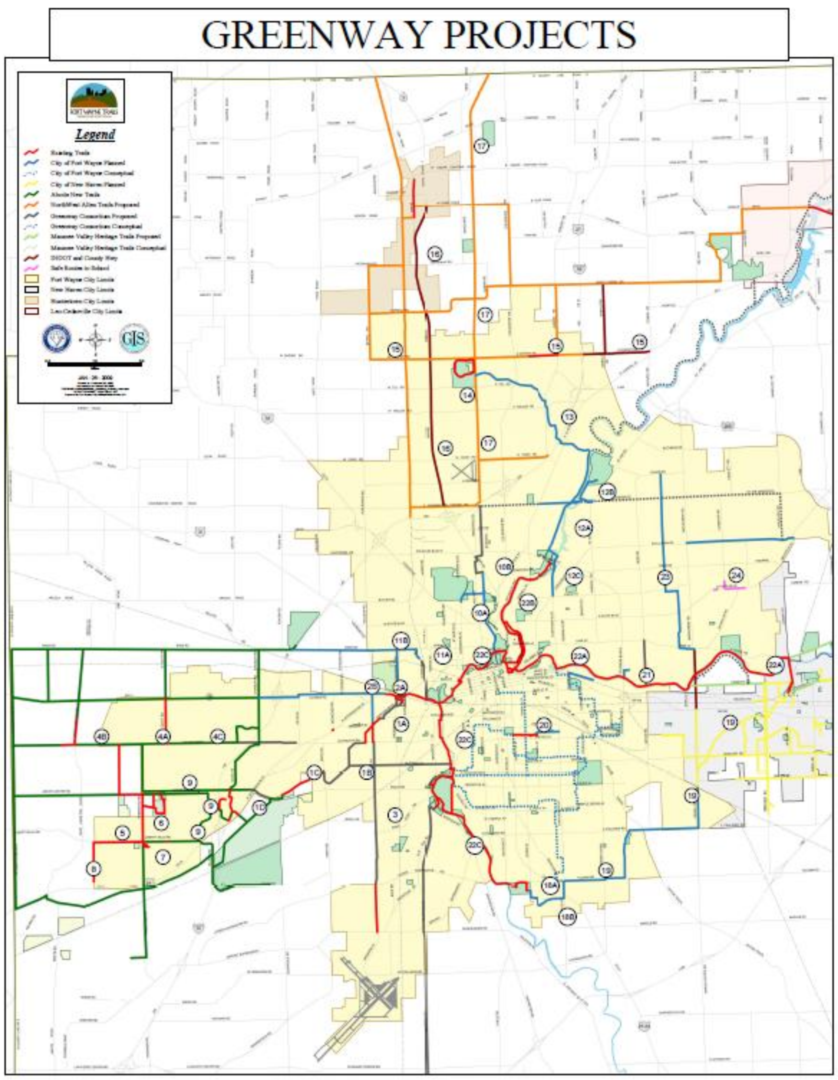

Figure 2. Rivergreenway Project Status Map as of 4/29/09.

1.3. Requirements, Specifications, and Given Parameters

The following is a list of the specifications for the bridge project:

a) The bride must clear span Coliseum Boulevard due to minimal width of median in the roadway
b) Need to cohere to the Americans with Disabilities Act (ADA) which sets a maximum slope of 5% for the walkway (including sidewalk on approach)
c) Right of Way (R/W) is 80' from each direction of the centerline of Coliseum
d) Bridge shall be designed for a minimum life span of 50 years
e) Clearance height of at least 17.55' from the top of the existing pavement
f) Minimum live load of 85 psf
g) Design wind speed of 90 mph for a 3 s wind gust
h) Design according to American Association of State Highway and Transportation Officials (AASHTO) and Indiana Department of Transportation (INDOT) requirements
i) Width of bridge to be 10' wide (controls the design vehicle to be used)

1.4. Design Variables

In addition to meeting all requirements and specifications, there are also numerous design variables that must be considered for this project, which include:

1.4.1. Aesthetic Considerations (Bridge Type)

In addition to being able to safely support any and all expected loads on the bridge, the structure should also have an innovative design to mesh with the other two pedestrian bridges on the IPFW campus. The main types of bridges that will be considered for this project are:

a) Truss
b) Suspension
c) Cable Stayed
d) Arch

1.4.2. Construction Materials

Materials used in the structural design of the bridge will be the most common materials used in the field of civil engineering. These include:

a) Steel
b) Reinforced Concrete
c) Soil (the most widely used material in civil engineering)

1.4.3. Coliseum Expansion

Even though the right of way of Coliseum Boulevard is taken into consideration, before construction commences, it should be determined if there are any plans for Coliseum Boulevard to be expanded in the future.

1.4.4. Connect to Ivy Tech

With the main classroom building for Ivy Tech being close to the road, the design could include an additional or even incorporated structure that would connect the bridge with the building. This would allow for ease of use for students as now they would be directly in the Ivy Tech building once they cross Coliseum Boulevard.

1.4.5. Covered or Open

Another design variable is if the design of the bridge will included a covered path, or if it will remain uncovered. If the walkway remains uncovered, a system such as some sort of fencing will need to be put in place to provide safety to both the pedestrians using the bridge as well as vehicles passing underneath.

1.5. Limitations and Constraints

1.5.1. Cost

With the tough economic times that this country is now facing, cost has become an ever increasing factor when considering construction of any new structure. The proposed design must be optimized in order to satisfy all requirements while minimizing the cost of the structure.

1.5.2. Natural Conditions

There are many factors of the local environment that must be taken into account during the design of the bridge which include:

a) Soil bearing pressure
b) Natural contours for slope on each side of the bridge
c) Weather conditions

1.5.3. Construction Issues

Although this project does not include the detailed construction process, there are aspects of construction that must be taken into account during the design stages. A few of these are:

a) 50,000 vehicles a day on Coliseum (main arterial) – need to minimize the adverse effects of closing the road down for long periods of time
b) Steel lengths – want to make sure that the design members are able to be shipped by tractor trailer to the jobsite. Will need to make sure that the members are less than 100' long and 14' tall (when loaded on trailer).

1.5.4. Additional Considerations

In additional to the information above, there are further details that must be considered in the design of the bridge.

a) Driver's ability to view the IPFW sign from the road
b) Serviceability of the structure
c) Addition of items to enhance the aesthetic properties of the bridge

1.6. SAP2000

Founded in 1975 by now company President Ashraf Habibullah, Computers & Structures, Inc. (CSI) is a worldwide leader in the development of software used in the design and analysis of civil engineering structures. Instead of producing software that can be used for a generalized range of structures, CSI tailors their programs to be tailored to specific classes of structures. SAP2000, the software used in the analysis of the pedestrian bridge, is intended for use on structures such as bridges, dams, stadiums, industrial structures, and buildings. Other titles produced by CSI include: ETABS, software used mainly for building, and the SAFE System, a powerful program used to design and analyze concrete slabs and foundations.

SAP2000's power comes in its amazing flexibility. From the simplest design of a two dimensional frame, to a complicated bridge in three dimensions, to the "Bird's Nest" (Chinese National) Stadium from the Games of the XXIX Olympiad, it can be seen that vast power that lies within this software package. Its true strength is in its various analysis options: linear, nonlinear, static and dynamic analysis of two and three dimensional structures. The advanced features of SAP2000 allow for a structure to be analyzed even when a material no longer falls in the linear range where Hooke's Law is valid (stress is no longer proportional to strain).

Students in the civil engineering program at IPFW are first introduced to SAP2000 in CE 375: Structural Analysis and then further explore the depths of the software in two more courses: CE 376: Design of Concrete Structures, and CE 475 Design of Steel Structures. In addition to these three courses, the software was used extensively in the entire Senior Design Project. Throughout these courses, the basic steps in designing a structure are taught while learning the interface of the SAP2000 software. Any structural design which can be completed in SAP2000 may be broken into four steps:

a) Modeling

Upon creating a new file, the software prompts the user with a screen asking for the user to define a new model. The user can either define a new model and grid system themselves, or they can choose a predefined template such as a beam, 2-d or 3-d truss, 3-d frame, etc. If creating a new model with a user defined 3-d grid, it is advised to carefully define a grid that allows for the model to be correctly defined. By taking a few extra minutes setting up the initial grid, the user can save a tremendous amount of time later on in the modeling of the structure.

After the grid is set, it is now time for the user to start the actual design of the structure. Prior to placing members in the model, the materials the user wishes to use in the structure must be defined. Defining materials is easily done through the software which has built into the system a database that has numerous shapes and sizes of steel and aluminum members that are used by different agencies throughout the world. Once the materials are defined, a members shape or material can simply be changed by a dialog box which will then modify the mechanical properties of the member. In addition to frame members, cables, and tendons, the user can also define shells and planes that may be used in a structure. The materials that are defined for use in the shell or plane are also easily defined through the built in database.

Once all members and materials are defined (they can be revised at any time), the structure can then be drawn on the grid system. After the correctly dimensioned structure is on the grid, pre-analysis activities are completed to accurately model the structure. These steps include: meshing any objects together so that they act like one continuous member, correctly setting any constraints, and/or restraints to joints to precisely model the joint if it may be a pinned or fixed connection for example, and applying any releases to the members in order to apply internal force releases at a given point.

The last step in modeling the structure is to determine the loads that will be applied to it. SAP2000 allows for live, dead (which includes the structures self weight), moving, earthquake, wind, etc. loads that can be analyzed both separately as well as concurrently according to AASHTO LRFD specifications. With these loads in place, the user can then proceed to the analysis of the structure.

b) Analysis

If the user has taken the time to meticulously set up an accurate model of the structure, analysis of the structure becomes streamlined. With the loading conditions already applied to the model, all the user must do is determine which load cases they would like to run (any or all of them), and then they simply press the “Run Now” button.

While SAP2000 is analyzing the structure, a dialog box is displayed on the computer screen showing the status of the analysis. It is on this screen that the program will inform the user whether the structure was successfully analyzed, or if there was an error. If the user is just performing a linear, static analysis, the program may only take a few seconds before the analysis output may be displayed; however, if a nonlinear or dynamic analysis is performed, the user may wait much longer for the structural analysis to be completed. In some cases, numerous iterations may be needed in order for an acceptable convergence value to be established.

c) Display

Following the completed analysis of the structure, the user is then able to view the mechanical behavior of the structure. For every different display options that can be selected, the user is given the option to view the results per the selected loading condition. Once the analysis finishes the default view of the structure is its deformed shape. This display can be extremely convenient to visualize the effects of the applied loads on the structure, and if the deformation agrees with the anticipated results of the loading. If there is an error in how the model is designed, it may be obvious by erratic results of the deformed shape of the structure.

The other option for the display is to show the resultant forces for the joints, frames/cables, and shells. If the “Joint” display is chosen, the forces acting on that joint for the given loading condition is displayed on the screen. These results can then be used for the design of the supporting structure, whether it be a foundation, or any other type of support required.

For the frames/cables force option, the user is given various options to view different forces acting on the member which include: axial force, torsion, shear 2-2, shear 3-3, moment 2-2, and moment 3-3. An additional option for the forces is to either show the actual values on the structure itself, or to just display a filled diagram representing the corresponding force acting on the member. Much like the deformed shape display, this view allows the user to visually determine if the structure is acting accordingly to the design load cases acting on it.

If the designed structure has any shell objects, the final display option is to view the forces acting on these shell elements. Options for the shell force diagram include: the component types may it be resultant forces, shell stresses, or concrete design; output type for visible, top, or bottom face as well as whether they are the maximum or minimum values; and, the forces in the components, whether they be the various layers of concrete, or the reinforcement steel in the concrete.

d) Design

If the mechanical behavior of the structure is deemed to be accurate, the final step in SAP2000 is the actual design of the structure. An extremely useful feature of the software is that the user can define a list of member shapes and sizes that the program can chose between to safely support the forces per the given loading combinations. This feature eliminates the need for the user to manually go back and forth choosing different sized members by a trial and error approach. Instead, the user can allow the program to optimize the steel design members. This can save the designer hours of their time.

All the user has to do when they feel that they are ready to start the design of the structure is to select the correct design option (steel frame, concrete frame, or aluminum frame), and the software will go through and design the structure. After all members are analyzed, the resulting screen will show the corresponding size of the member as well as, judging by the members color, whether or not the member passed the design standards. When the design is complete, it is highly advised to run the option, “Verify Analysis vs. Design Section” to determine if the analyzed members are the same as the design sizes which will affect the dead load of the structure. If the members are found to differ, all the user needs to do is rerun the analysis as well as the design of the structure, repeating these two steps until the analysis and design members converge.

2. Section II: Conceptual Design

2.1. Location of Bridge

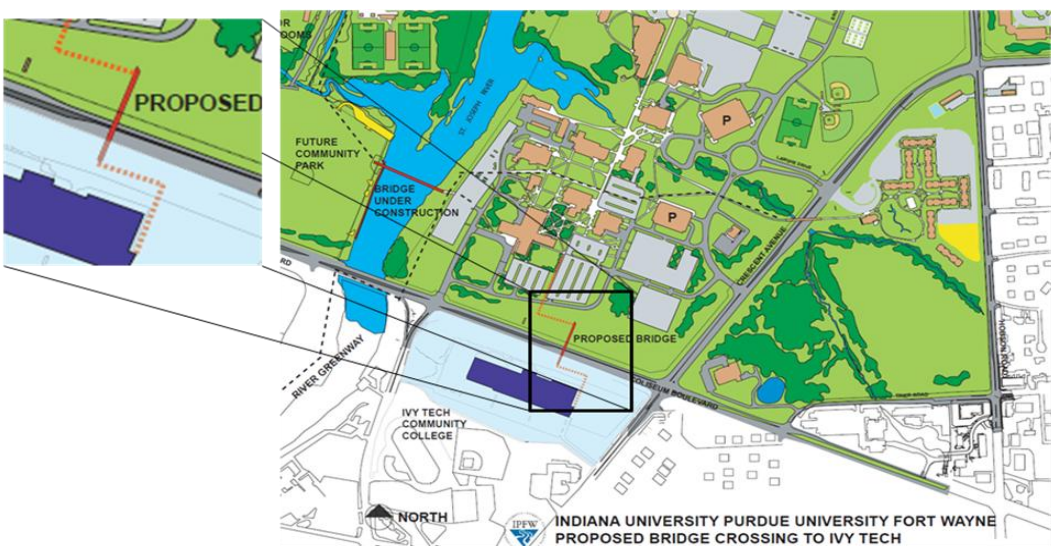

Figure 3. Proposed Location of Pedestrian Bridge (from previous TE application).

Figure 3 shows the proposed location of the pedestrian bridge as determined by the IPFW Physical Plant in the TE Application completed in August of 2008. This location helps serve many of the functions required in the bridge design. First, the location is in an open area where there are currently no structures that would need to be razed in order to construct the new bridge. Also, this point allows for a maximum use of the natural topography on the IPFW side of Coliseum (the north side) to help maintain the maximum 5% slope without having to build another structure (i.e. elevator) that would be used to lower the sidewalk from the bridge deck to ground level. Using the natural topography for the slope requirements minimizes the need for massive amounts of soil brought into the site also. For the Ivy Tech side of the bridge (south side of Coliseum), there is not enough space to allow for the sidewalk to drop directly from the bridge and to the classroom building with no curves in the sidewalk. Instead, the sidewalk will need to come off of the bridge and run parallel to Coliseum Boulevard until the at grade level is reached using the ADA requirements. An option that can be pursued for pedestrians who do not want to walk the extra distance needed to meet ADA requirements is tha a stairway may be constructed next to the bridge which can give the pedestrians a direct exit from the bridge to the Ivy Tech campus.

In addition to using the natural slope to help in the slope of the sidewalk leading to the bridge, the location shown in Figure 3 also minimizes the impact on vehicular traveler’s view of the brick IPFW sign off of the roadway. The location does block some of the view of the IPFW as travelers move west on Coliseum Boulevard, however at this location the view should not be hindered too much.

This location also allows for pedestrians to access the pedestrian bridge from the Rivergreenway Trail that is just to the west of Ivy Tech. As long as there is a sidewalk designed to connect the bridge to a sidewalk leading towards the Rivergreenway Trail. By doing this, the Rivergreenway Trail would finally be able to connect to the parks and trails to the north of Coliseum Boulevard.

2.2. Concept I: Cable-Stayed Bridge

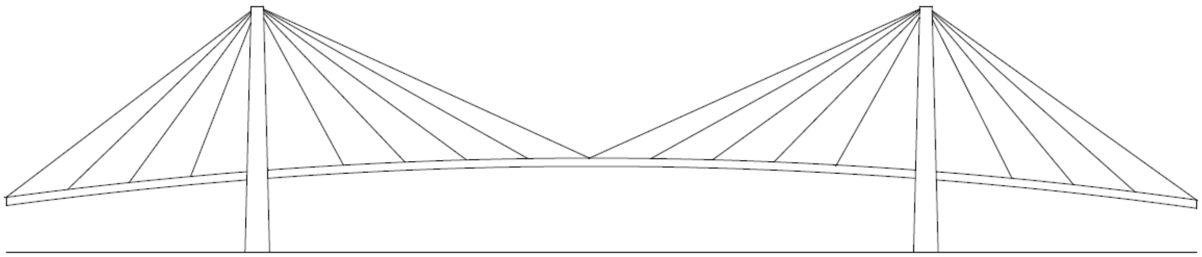

Figure 4. Conceptual design of cable-stayed bridge over Coliseum Boulevard.

Figure 5. Computer generated design of Venderly Family Bridge.

2.3. Concept II: Truss Bridge

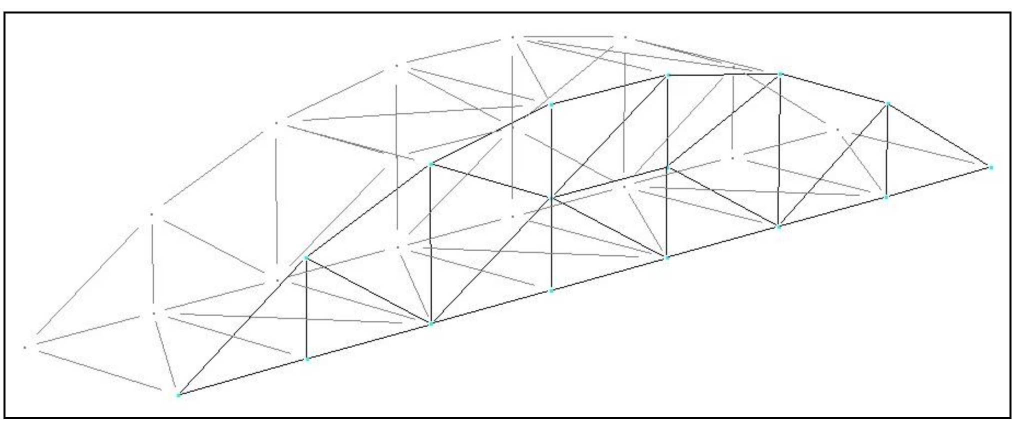

Figure 6. Design of truss bridge from CE 375 class project.

Figure 7. Example of pedestrian truss bridge utilizing weathering steel members.

2.4. Concept III: Suspension Bridge

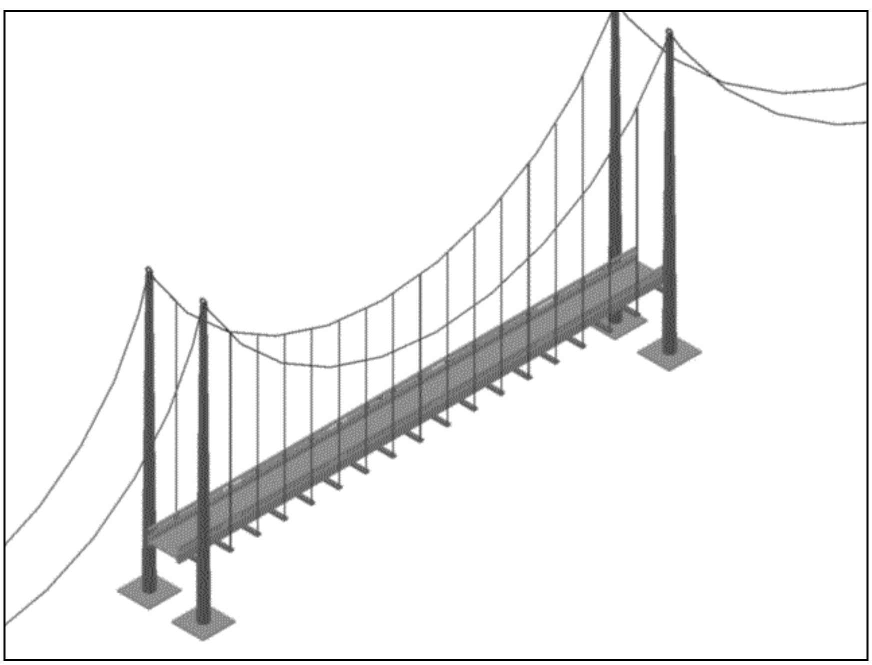

Figure 8. Computer rendering of pedestrian suspension bridge.

2.5. Concept IV: Arch Bridge

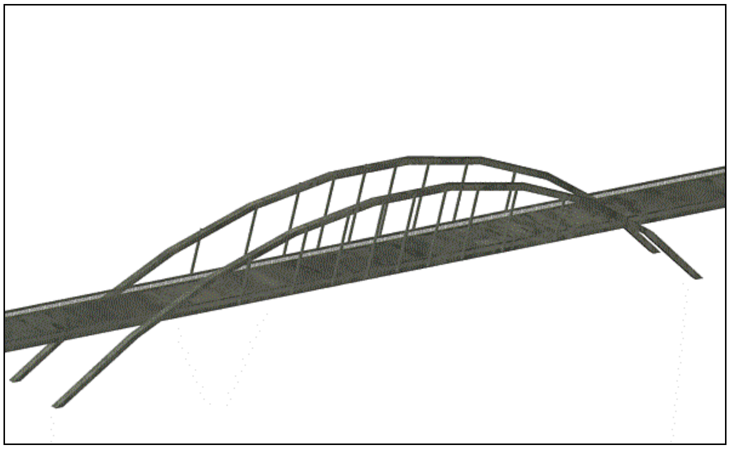

Figure 9. Computer rendering of arch bridge over Coliseum.

3. Section III: Summary of the Evaluation of Different Conceptual Designs

3.1. Concept I: Cable-Stayed Bridge

3.1.1. Advantages

a) Aesthetically pleasing
b) Ability for long clear spans
c) Modern style of bridge construction

3.1.2. Disadvantages

a) Need adequate spacing on either side of columns to reduce eccentric loading
b) Covering takes away from the appeal of the design
c) More cost effective for long spans/not for this short of a span
d) Difficult to construct
e) Already one on the IPFW campus (crossing the St. Joseph River)

3.2. Concept II: Truss Bridge

3.2.1. Advantages

a) Low cost
b) Ease of construction
c) Minimizes the amount of material needed for structure

d) Able to be covered while maintaining original appearance

3.2.2. Disadvantages

a) NOT aesthetically pleasing

b) Would not be compatible with the innovative design of the other two bridges already on the IPFW campus

3.3. Concept III: Suspension Bridge

3.3.1. Advantages

a) Aesthetically pleasing

b) Comparable design to other bridges on campus (Willis Family Bridge)

c) Ability for long spans

3.3.2. Disadvantages

a) Needs to have adequate distance for anchorage points on either side of main supporting columns (space is limited on Ivy Tech side of the bridge)

b) Difficult and unattractive to cover if "normal" suspension bridge

c) Expensive to construct

d) Would need to close Coliseum Boulevard for an extended period of time

3.4. Concept IV: Arch Bridge

3.4.1. Advantages

a) Aesthetically pleasing

b) Design has yet to be done on the IPFW campus

c) Can easily be covered

d) Construction can be formed to minimize the impact to traffic on Coliseum Boulevard – many of the pieces can be prefabricated

e) Due to span, cost effective given the bridges requirements

f) If designed as a parabolic arch, all forces in the arch axial

3.4.2. Disadvantages

a) Large horizontal forces applied to the foundations from the arch

b) Uses large amounts of steel

3.5. Decision Matrix

To help assist the group in what bridge design they would go with for the crossing of Coliseum Boulevard, the group used a decision matrix as shown in Table 1. The matrix was designed with a set of standard guidelines used in bridge design, taken from the *Handbook of Structural Engineering*. Each item is given a priority 1 to 5 (1 = low, 2 = standard, 3 = high, 4 = very high, and 5 = extremely high) as well as a quality rating on a scale 1 to 5 (1 = poor, 2 = fair, 3 = good, 4 = very good, and 5 = excellent). The weighted average is then taken by multiplying the priority value by the quality rating with these values summed to find the total rating of the proposed design.

Table 1. Decision Matrix

Bridge Type	Structural		Constructability		Maintenance and Inspection		Construction Schedule Impact		Aesthetics		Cost		Total Rating
	Priority	Quality	Priority	Quality	Priority	Quality	Priority	Quality	Priority	Quality	Priority	Quality	
Cable-Stayed	1	5	3	2	2	2	4	5	5	5	5	2	70
Truss	1	5	3	4	2	4	4	2	5	1	5	5	63
Suspension	1	5	3	3	2	2	4	4	5	5	5	3	74
Arch	1	5	3	4	2	4	4	5	5	4	5	4	85

As is shown in the decision matrix in Table 1, the top rated bridge design is the Arch. This design had a total weighted rating of 85, compared to the next closest, the suspension bridge, which rated 74. Based upon the results of the decision matrix, the group determined to proceed with the arch as the base design of the pedestrian bridge. It is here that a note should be made about what the group found out about cost comparisons of different bridge designs in the *Bridge Engineering Handbook*. In this text, the author stated that costs between an arch bridge and a truss bridge a comparable, and that if all other factors remain equal, the best choice is usually an arch bridge due to its aesthetic superiority.

3.6. Selected Design

3.6.1. Background

An arch is an excellent choice in supporting long span structures due to their ability to reduce bending moments in the structure while carrying the load mainly in compression. A general rule of thumb is that when designing a steel bridge, “the arch system is expedient to use for spans longer than 160 ft” (Chen and Duan). By limiting the bending stresses induced on the arch structure, member sizes may be reduced since the chief load that they are supporting is the compression forces applied to it while the other forces are minor. With compression forces being the main load that the arch is supported, care must be taken in the structural design of the members to ensure that it will not buckle under the potentially large compression forces enacting on the structure. In order to reduce the chance of a catastrophic failure associated with buckling, the structural members must be sized accordingly using shapes that utilize large moments of inertia as is seen with hollow structural sections (HSS).

Depending on its given application, various types of arches may be chosen to support a given loading condition. The first, a fixed arch, is commonly used when the arch is to be constructed of reinforced concrete. Although it may require less material to construct than the other types of arches, the fixed arch does pose some potential problems since due to its geometry it is statically indeterminate to the third degree. Being statically indeterminate leaves the arch prone to additional stresses if there is any settlement of the foundation. Thus, if using a fixed arch the designer must make certain that solid foundation abutments are used to minimize the likelihood of the foundation settling.

The next type of commonly used arch is the two-hinged arch. Usually constructed of wood or timber, the two-hinged arch is less sensitive to settling since the structure is only indeterminate to the first degree. A modification to the two-hinged arch is the tied-arch. By connecting the supports with a cable, the arch can behave like a rigid unit since the tie carries the load in the horizontal direction. If the tie is used, the second support can then be a roller which allows the structure to become statically determinate.

Similar to the two-hinged arch, the three-hinged arch is basically a two-hinged arch with another hinge placed at the apex of the arch. Since there are three hinges, the structure can be disassembled which allows the arch to be statically determinate due to the fact that there are now six equations with six unknowns. With the arch being statically determinate, the structure is not affected by either settlement or temperature change leading to the three-hinged arch being an excellent option when designing an arched structure. Thus the analysis and design of this pedestrian bridge will utilize the three-hinged arch concept (Hibbeler).

3.6.2. Meeting with Greg Justice

Early on in the senior design process, the group knew that they wanted to perform the structural design of a project that would benefit the IPFW campus, but were unsure of what type of structure to design. This uncertainty led the group to schedule a meeting with Greg Justice, a Senior Project Manager at the IPFW Physical Plant. The group was surprised by the amount of projects that were currently in some phase of the construction process. With many options in front of the group, it was now time to determine which route to take: should the group design an entire building, or a pedestrian bridge?

It was at this meeting that Mr. Justice spoke with the team members about a pedestrian bridge that was in the proposal stage. Being designed to cross over Coliseum Boulevard, the bridge had just recently been sent to the university for approval; however, due to lack of funding, the bridge construction was postponed for now. Not necessarily wanting to perform the design of a building, the group made a decision to pursue the more challenging avenue of designing a pedestrian bridge. Mr. Justice was kind enough to forward us the completed Transportation Enhancement

(TE) Application for the bridge that was submitted to Purdue University. In addition to providing us access to this information, Mr. Justice also suggested we contact Kurt J. Heidenreich, P.E., S.E., whom not only collaborated with Mr. Justice on the application, but who is also the engineer that designed the other two pedestrian bridges on the IPFW campus.

3.6.3. Meeting with Kurt J. Heidenreich, P.E., S.E.

On February 23, 2009 at 3:00 p.m. the entire group met at the offices of Engineering Resources, Inc. to talk about the proposed bridge design with company President Kurt J. Heidenreich, P.E., S.E. The group brought Mr. Heidenreich up to date about our meeting with Greg Justice at the IPFW Physical Plant, and how he had given us the bid proposal that had already been turned down. Mr. Heidenreich made it clear to the group that in this proposal, all drawings and information were of the very preliminary thought concept stage and that the ideas that he and Mr. Justice presented in the proposal were entirely “rough ideas”.

After the group informed Mr. Heidenreich about the background information the group had collected, the conversation shifted to the other two pedestrian bridges that are on the IPFW campus; both of which were designed by Mr. Heidenreich. The first bridge he did on the campus, the Willis Family Bridge, was designed to allow students to travel from the student housing complex on the Waterford Campus, over Crescent Avenue, to the heart of the IPFW campus. As any passerby is aware, this bridge has a unique design that is a keystone of the IPFW campus. Representing the suspension bridge design, the Willis Family Bridge relies upon the two cables that are suspended from the triangular-shaped supports to carry the bridge deck.

The conversation then briefly turned to the other bridge on the campus that Mr. Heidenreich designed, the Venderly Family Bridge that crosses over the Saint Joseph River. This bridge is a cable-stayed bridge consisting of the two main towers that have cables anchored into them. These cables are what support the bridge structure.

Discussion of the previously designed bridges on campus turned to the newly proposed bridge crossing over Coliseum Boulevard which is the main purpose of the meeting. It was during this that Mr. Heidenreich gave us a peak into the mind of what a structural engineer must consider before the design process commences. With aesthetic considerations always playing a pivotal role in any idea, other details such as construction techniques, impact on the environment, various loading conditions, and height requirements were discussed with the group. Mr. Heidenreich also brought out his copy of the <u>AASHTO LRFD Movable Highway Bridge Design Specifications: 2008 Interim Revisions</u> which is a massive volume of design specifications used for pedestrian bridge projects in the United States. He recommended for the future engineers in front of him to take a look at this book, and design the proposed bridge in accordance with it.

As the meeting came to a close, the group inquired of Mr. Heidenreich of why Mr. Justice and he were leaning towards the arch bridge design in comparison with the other choices. He let us know that the main reason that at the required span (160’), the arch would be the most cost effective option in comparison with the others while a truss bridge was not an option per the request of the University. Also, neither a suspension nor cable-stayed bridge were good choices since the Ivy Tech side of Coliseum offers little space for an area to anchor cables to. With this valuable insight, the group became heavily swayed in the choice of an arched pedestrian bridge as the selected conceptual design.

4. Section IV: Detailed Design of the Selected Conceptual Design

4.1. Arch without Angled Members

In order to begin the detailed design of the arch bridge, the group must first design the bridge using a normal arch designed without any modifications, and then continue to the final design. Designing the arch in the xz plane allows the group to easily perform hand calculations to verify that the structure is accurately modeled in SAP2000.

4.1.1. Modeling of Bridge

The complete steps in modeling the arched bridge design are outlined in 4.2 Arch with Angled Members. This design is only used to serve as a check of if the SAP2000 model has any issues or irregularities associated with it. Although this is not the final design, all of these settings will be in place to allow for an easy transition to the final design using the same template.

4.1.2. Arch Bridge Design

Shown in Figure 10 is the model of the arch bridge in SAP2000. This drawing displays the complete structural outline of the arch bridge that is used as a verification of the model used prior to proceeding to the final design of the pedestrian bridge.

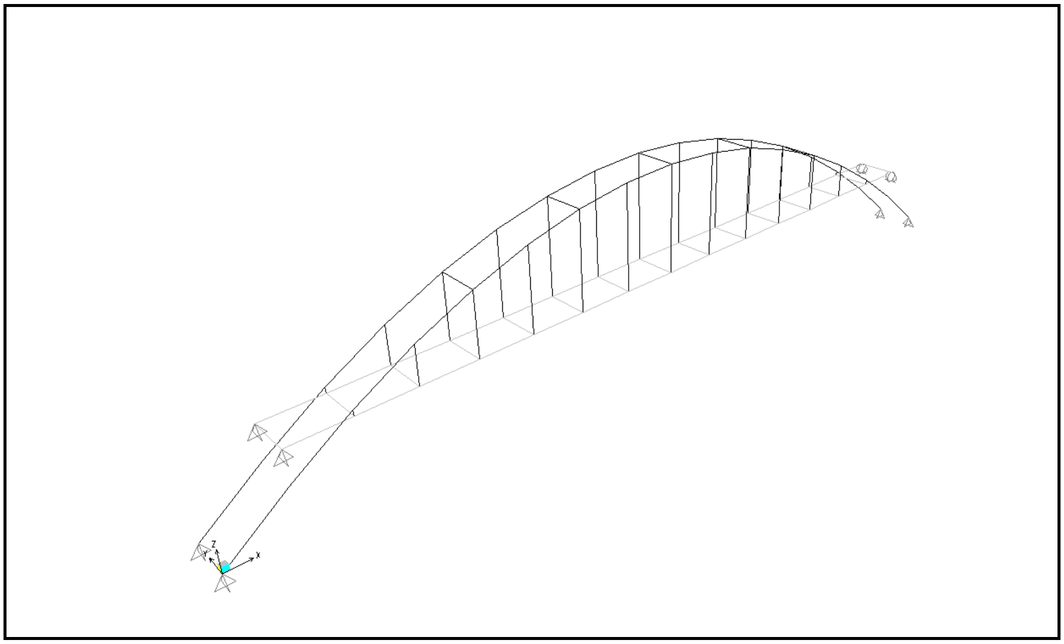

Figure 10. SAP2000 Model of Arch Bridge without angled members

4.1.3. SAP2000 Analysis

In order to verify the final model, the bridge was modeled as an in plane arch as shown in Figure 10. For this design, the bridge members are all defined as those described in section 4.2 Arch with Angled Members. The area is modeled as a 9” thick concrete deck with equal spans of 13.125’. After all of the members were drawn in accordance with section 4.2, the group set the live and dead load cases to be analyzed in SAP2000. All analysis and design information for the arch bridge without angled members can be found, upon request, in the SAP2000 report.

4.1.4. Hand Check of Calculations

4.1.4.1. Supporting Cables (Angle Members)

The first step in making sure that the bridge is modeled correctly is by verifying that the load on the deck is transferred correctly to the arch members. This is accomplished by verifying the loads supported by the angles that support the deck.

Detailed in the appendix in 8.1.1, the group performed hand calculations verifying both the live and dead loads supported by the angle members. Using the tributary of a single concrete deck piece (10’ x 13.125’), the group found that the difference in the analysis by SAP2000 from what hand calculations showed are a difference of +5.6% for the live load, and -2.5% for the dead load.

4.1.4.2. Arch

Since the group determined to go with the three-hinged arch, hand calculations were easily carried out with their being six unknown forces and six equations to solve for these forces. The steps used in calculating the forces are shown in 8.1.2 where the loading condition used is that of the dead load of the 9” concrete slab. After performing the hand calculations, the group compared these values to those obtained through the SAP2000 analysis. These values were off approximately -8.3% for all of the loads in the x-direction while the values were off +8.9% for the loads in the y-direction.

4.1.5. Conclusion

Based upon the comparison of values from those obtained through the analysis in SAP2000 to those derived from the hand calculations, the group has determined that this model is an accurate representation of the proposed pedestrian bridge (without the angled arch members). With all of the values within an acceptable range compared to any hand calculations (largest difference of 8.9%), the proposed model supported loads as the group determined it should.

The group has determined that the differences calculated for the arch itself are larger than that of the angled members due to the shape of the arch. Although the arch is drawn as a parabolic arch, the shape is not completely parabolic for reasons further discussed in section 4.2. Instead of being a completely smooth parabola from the initial point to its end, the arch is broken down into 16 equally sized portions. Because of this, although the arch is close to being parabolic, there are some slight differences along the shape of the arch which allows for shear and moment forces to be introduced into the arch. It is the effects of these forces that cause the variance in the SAP2000 analysis versus the hand calculations.

4.2. Arch with Angled Members

4.2.1. Modeling of Bridge

The process of modeling the bridge that must span over Coliseum Boulevard is completed in accordance to the four steps detailed in 1.6.1. Upon creating a new model, the group chose to design the bridge by utilizing a user defined grid system. By previously calculating the required span, length, and height for the bridge in order to meet the height requirements specified (17.55’ from top of pavement to bottom of lowest bridge member), the group determined that the bridge would span 210’, and have a maximum height of 44’. This height was chosen because it falls within the normal rise-to-span ratios of 1:4.5 to 1:6 that are commonly used for the design of arch bridges (Chen and Duan). Hence, the grid was set up as follows: 211 X-units at 1’ spacing (210’), 2 Y-units at 10’ spacing (10’ width), and 45 Z-units at 1’ spacing (44’).

4.2.1.1. Frame Members

Since the group is familiar with the SAP2000 program, immediately after the grid is defined, the group began to define the materials and members used in the model. This allows for the design to go smoothly since all member shapes and sizes are defined prior to drawing any of the structure’s members.

4.2.1.1.1. Arch Members

By using an arched structure, the group knew in advance that the main forces carried in the structure would be compression forces. With this being the case, the group chose HSS members for the main arch supports for their known performance in supporting large compressive forces. Opening the accompanying database included in SAP2000, the group was able to import various HSS sizes (diameter and thickness) into the model.

Once the sizes were brought into the model, the next step was to define an auto select list named “HSS”. Defining an auto select list allows the user to draw the members in the grid with the initial size being the median size of all of the selected members. The advantage of defining an auto select list comes when the design process in SAP2000 takes place: now, during the design process, the software will optimize the member size eliminating the need for a “trial and error” approach in designing the structure.

Easily accomplished in SAP2000, the arch members can be drawn in the model through the application of two point-and-clicks with the mouse, and a few user inputs. Going to the draw frame member option, the user is prompted for what type of member is to be drawn: straight frame, curved frame, cable, or tendon. In addition to the member type, the user determines the section type, in this case the “HSS” auto-shape, and whether the member experiences any moment releases, selecting either “Continuous” if it is to be modeled as one solid member, or “Pinned”, if there is to be pinned connections at transition points. For the arch members, the “Curved Frame”, “HSS”, and “Continuous” are selected in this menu.

The next step was to actually draw in the member. Clicking on the initial reference point (0,0,0), and then dragging the mouse to (210,0,0), another dialog box appears prompting the user for some information in determining the shape of the curved frame member. In the box for curve type, the “Parabolic Arch – 3rd Point Coordinates” is selected in order to draw a parabolic arch (the reasoning behind this is detailed in Section 3.6.1.). Selecting the 3rd point coordinate as (0,105,44) allows for the arch to be designed in accordance with the calculations previously determined to yield the correct distance and height requirements for the location of the bridge. Figure 11 shows the arch members drawn in SAP2000 on the XZ gridlines.

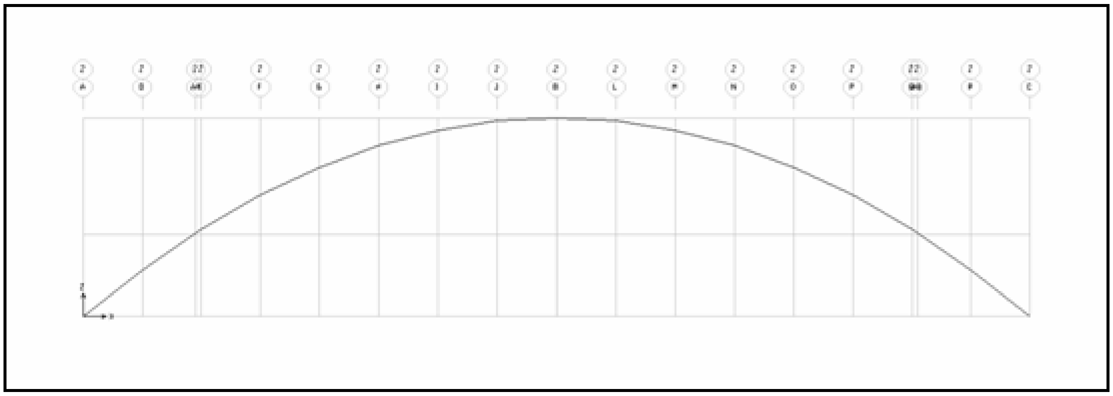

Figure 11. Arch members drawn in the user defined grid.

With the points of the arch determined, the next step was to determine how the software would mesh the members together. Previously doing trial designs for the bridge, the group knew that keeping the arch as a single object leads to inaccurate analysis results in SAP2000. Instead of keeping as a single object, the arch members are modeled as multiple equal length objects. Not only does this allow for accurate analysis in SAP2000, it also allows for ease in construction due to ability to manufacture similar members, and not having to construct numerous unique pieces. For the design of the parabolic arch, the group decided to use 16 similar sized members to form the main arches, and deciding to use each of these connections as the joints where the cables would transfer the bridge deck loads to the arch. In addition, the group defined an internal pin connection at the apex of the arch so that the structure could be analyzed as a three-hinged arch.

With this first arch member in place, all the group had to do to draw in the other member was to replicate the entire shape. The second arch was replicated linearly at a distance of 42.36 ft in the Y-direction. If the arch was to remain in the XZ plane (as the model used in the verification process was) the group would be able to move on to the next step; however, the final design is to be composed of arched members that angle into the center of the walkway to give a more aesthetically pleasing look. Angling the members is similar to replicating the arch along the Y-axis only this time the group replicated the arch 23º into the center along the line that makes up the base of the structure. Figures 12 and 13 show what the arch members look like when they have been angled into the center which gives the entire structure a more aesthetically appealing look.

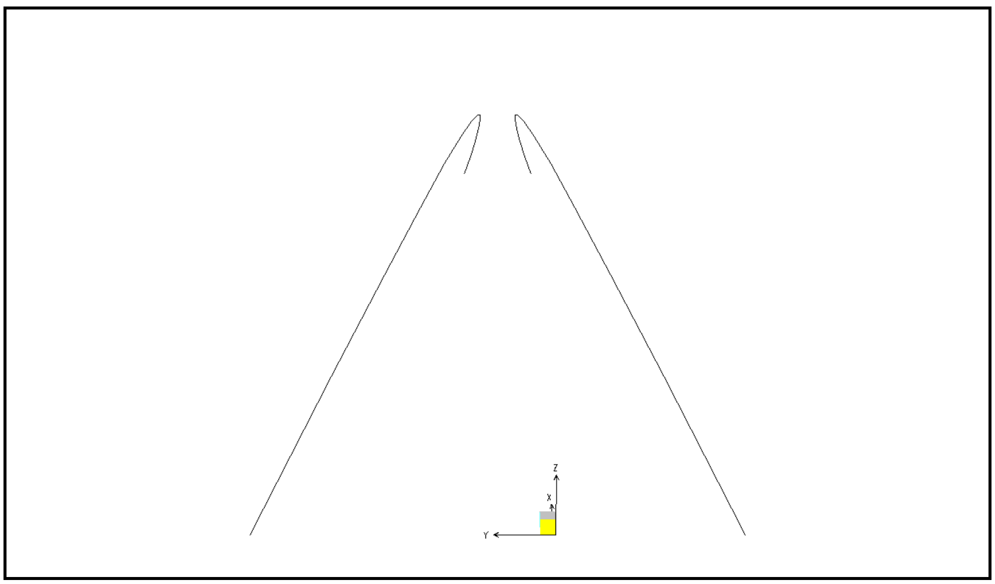

Figure 12. Arch members at an angle of 23º from perpendicular.

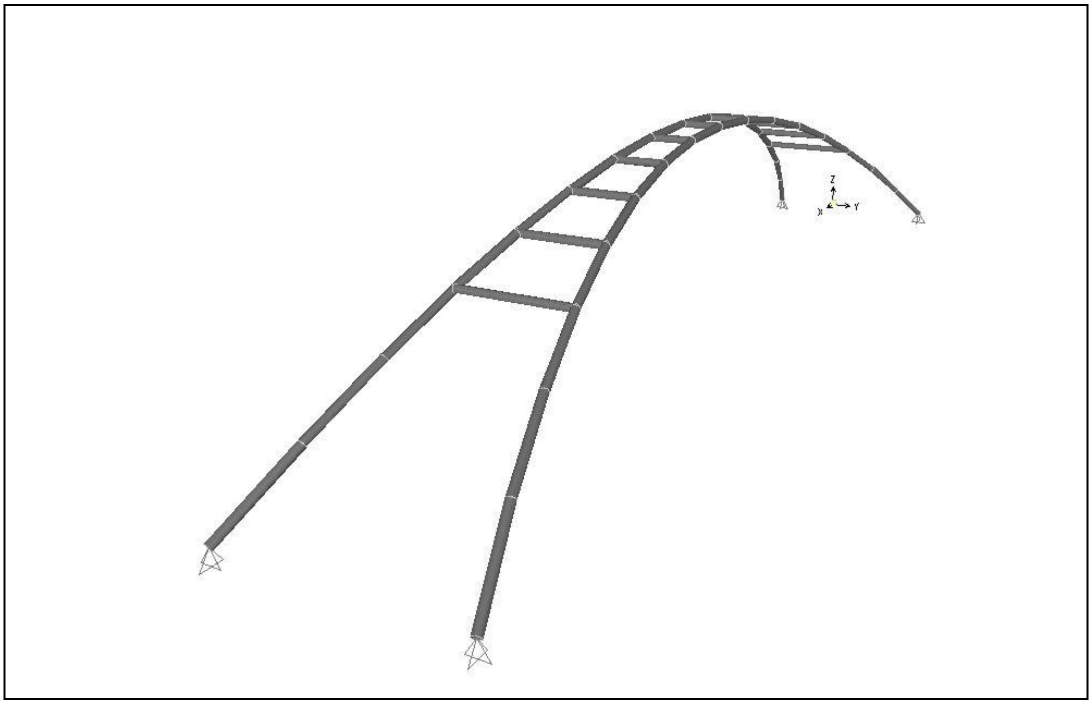

Figure 13. Structural frame for pedestrian bridge.

4.2.1.1.2. Cables

In the model, what will be the cable members in the final design are modeled as L shaped angle pieces in SAP2000. Substituting a straight line object like an angle for a cable was a tip that the group found in the CSI Analysis Reference Manual. The group determined to follow this tip after numerous failed attempts at accurately modeling the cables in SAP2000. Knowing that cables can only support tensile stresses, all the group had to do was assign a frame compression limit of zero to all of the angled members. The only catch is that to analyze these members without the ability to carry compression forces is that the software must execute a nonlinear analysis for the compression limit to be taken into effect. For the DEAD and LIVE load cases, these limits do not need to be set since the only forces that are applied to the structure will be gravitational forces; however, for all of the dynamic loading cases (all three wind load cases as well as the moving vehicle load cases) the compression limits must be set to force the cables to carry only tensile forces.

Much like the arch members, the frame objects used are selected from an auto select list, only this time they are defined as "ANGLE". Drawing the angled members in was made amazingly easier since prior to drawing in any members, the group spent a great deal of time in defining a grid system that makes for drawing the model quickly. Since the grid is in place, all the group has to do is draw the angles from the arch down to where the bridge deck will be. Also during this step, the lateral supports in between the arch members were drawn in, but instead of using angles for these members, the members are defined to be HSS since they will be carrying both compression and tensile forces depending on the loading conditions.

4.2.1.2. Areas

As stated in section 1.4.2, the other material used in the design of the bridge is concrete. During the structural analysis, the self weight of the bridge is what contributes to the applied dead load on the structure, so before drawing the concrete deck in the group had to determine an approximate thickness for the bridge deck. By using the tables found in Design of Concrete Structures the group was able to determine an approximate thickness of 6". The calculations used to determine this thickness are detailed further in section 4.5 Slab Design.

All of the deck sections were drawn in with the "Quick Area" tool in SAP2000. Drawing the areas in the XY plane, all of the deck sections are the same with dimensions of 10' wide and 13.125' long. Just as was explained for the beams and the arch members, this allows for ease of construction both at the plant as well as in the field. Figure 14 shows the bridge with the cables removed to allow for the deck to be easily seen.

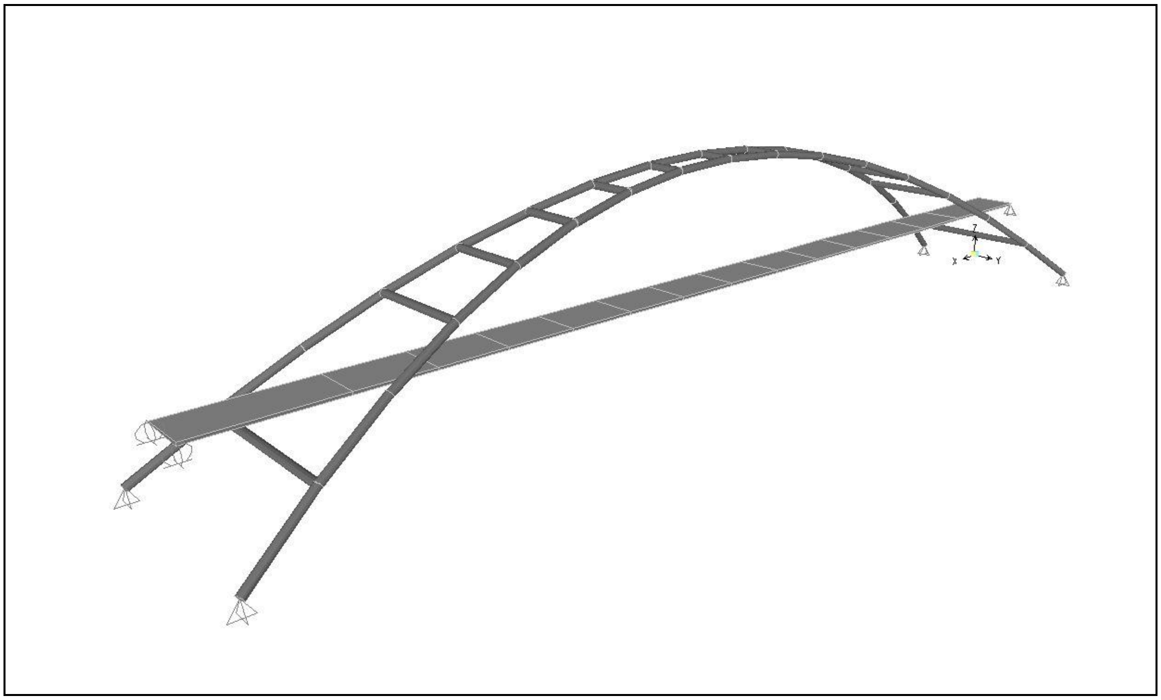

Figure 14. Bridge with deck drawn in (cables removed for clarity).

Once all of the area sections have been drawn in the model, the next step is to define them. As will be shown in section 4.5 Slab Design, the concrete slab is 6” thick with a compressive strength of 4 ksi. The concrete slab can be accurately defined in the “Areas Section: Shell Section Data”. In this menu, the group defined the slab to be 6” thick, constructed with f'_c of 4000 psi, as well as defining the reinforcing steel thickness and cover distances. For a more accurate analysis of the slab, the group defined the slab as a layered shell element which takes into account the composite nature of the concrete slab. In defining the slab as a layered shell, the group had to determine potential covers for top and bottom layers of reinforcing steel, along with the material used for the steel.

After defining what the slab would be constructed of, the next step was to adjust the stiffness modification factors of the deck. For this, the group lowered the factor for Membrane Modifiers f11 and f12 to zero (from one where the others remained). The finalized model with the concrete deck is shown in Figure 15.

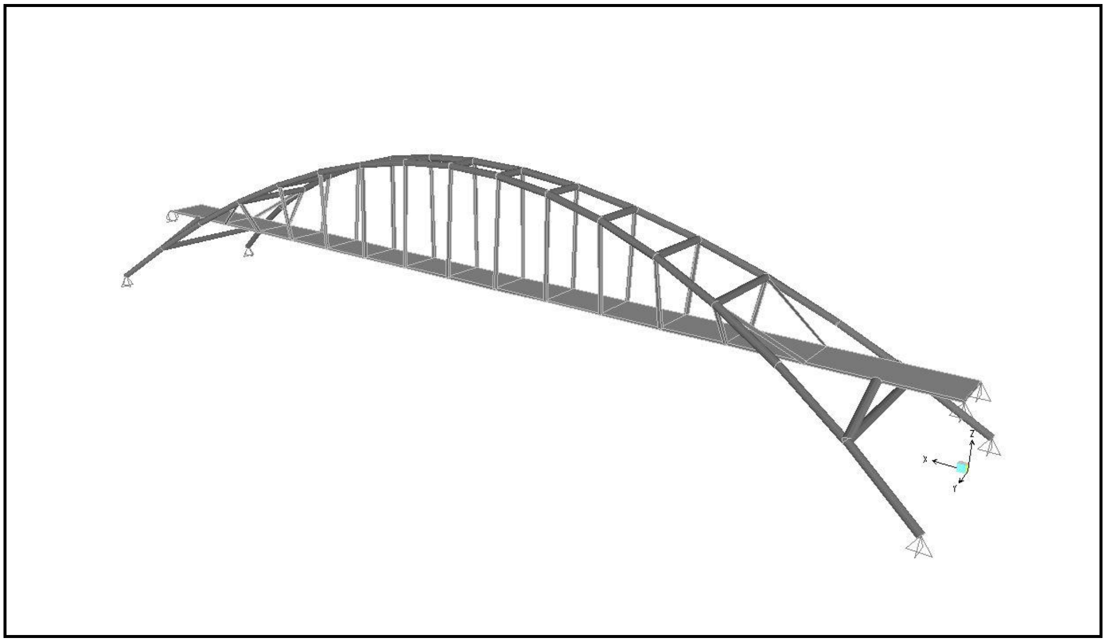

Figure 15. Complete Model of Pedestrian Bridge (Extruded view).

Now that all of the members are drawn in the model, the next step is to define how the objects are meshed with each other. By selecting all of the members (frame and area) at once, the meshing function is completed efficiently. The first area that is meshed is all of the frame objects which are meshed with joints as well as at intersections with other frames and area objects. The same is done for the area objects; areas are meshed with intersection with other frame objects and with point objects on the area's edges.

4.2.1.3. Restraints and Releases

As shown in Figure 15, the complete bride structure has all exterior restraints positioned at the joint locations where the bridge will come in contact with exterior supports. The restraints at all four points of the arch are modeled as pinned-connections, effectively eliminating any moment forces in the connection as well as maintaining the desired three-hinged arch for analysis purposes. Exterior supports for the concrete slab consist of a pinned-connection at one end with the other end being modeled as a roller-connection which allows for temperature expansion and contraction in the concrete deck.

In addition to the restraints used for modeling the exterior supports, various conditions and restraints are used for modeling of the frame members of the structure. The first condition that needed to be altered was the internal moment release at the apex of the arches. Approaching both sides of the apex, the ends of the final members are released from any moment forces. This release then allows for the software to analyze the joint as a pinned-connection.

Another restraint used for all of the cable (angle) members, the horizontal supporting HSS members, as well as for the beams is they are all released from any moment forces developing in the members. By releasing these members from both the major and minor moments at each of their ends changes these members to be analyzed as pin-pin connections at all joint locations.

The final modification used in the model was the release from any compression forces from forming in the cable members, as discussed earlier in section 4.2.1.1.2 Cables. Performing the action of both releasing the cable members from developing any compression forces in them (by setting the compression limit to “0”), and performing a nonlinear analysis on the bridge under certain loading conditions results in the angle members being analyzed as if they were drawn in the model as actual cables.

4.2.2. Loads

4.2.2.1. Dead Load

Dead loads are those loads that are permanently applied to the structure. For the pedestrian bridge that is being designed, there are three sources for the dead load: the weight of the concrete deck, the weight of any railing/supports on the side of the walkway, and the self weight of the structure. The group decided to use normal weight concrete for the decking which has an average weight of 150 pcf. Since the deck will be made from pre-cast concrete, once the bridge is built on the site some kind of overlay will need to be added in order to allow for the bridge to have a smooth, continuous surface. For this, the contractor may decide to coat the top of the concrete with an overlay, so an additional load of 10 psf has been added to take this overlay into account. In addition to the load from the deck, there was also a 90 plf load applied on either side of the walkway that takes into account any railing/fencing that will be built on the bridge. The railing/fencing load was transferred to the structure by a user defined load of 90 plf on the edge beams that support the concrete deck. Since the edge beams were designed through hand calculations performed by the group, a load was also applied by the group to the edge of the deck for the self weight of this beam. With the edge beam being later calculated to be a 10" x 16" rectangular beam, the group had to add 300 plf to either side of the deck to account for this weight. Finally, the self weight of the structure itself, including all HSS, Angles, and Beams, is calculated in SAP2000.

4.2.2.2. Live Load

The live loads applied to the bridge are variable loads applied to the bridge that are in addition to the dead loads on the structure. There are three live loads applied to the bridge: that of pedestrians, wind loading (Section 4.2.2.4), and a moving service vehicle load (Section 4.2.2.3). As described in the Reference 8 revised LRFD code, the specified live load for a pedestrian bridge can be taken as 90 psf. Previously, the LRFD design specified that use of a 85 psf; however, with the changing factors that the LRFD has used over the years, it has been found that a 90 psf live load multiplied by the factor of 1.75 (the current factor for a live load on a pedestrian bridge) is sufficient for pedestrian bridges. By using this load, the LRFD revised code states that, "Consideration of dynamic load allowance is not required with this loading [90 psf live load]" (LRFD Guides Specifications for

Pedestrian Bridges). For the design of the pedestrian bridge crossing over Coliseum Boulevard, the group has decided to use a live load of 85 psf with a check on the dynamic response of the structure being performed later.

4.2.2.3. Service Vehicle Load

In addition to the uniform live and dead loads applied to the bridge, the bridge must also be designed to carry the loading of a moving service vehicle. A designated service vehicle is needed in the design of the bridge in case there is an emergency vehicle needs to cross over the structure, or if a maintenance vehicle needs to access the walkway (i.e. removal of snow on the concrete deck). As detailed in *AASHTO LRFD Bridge Design Specifications*, with the walkway on the bridge being only 10 ft wide, the code recommends using an H5 design service vehicle, as shown in Figure 16. Further, the AASHTO code states that the service vehicle load is not applied in combination to the pedestrian live load.

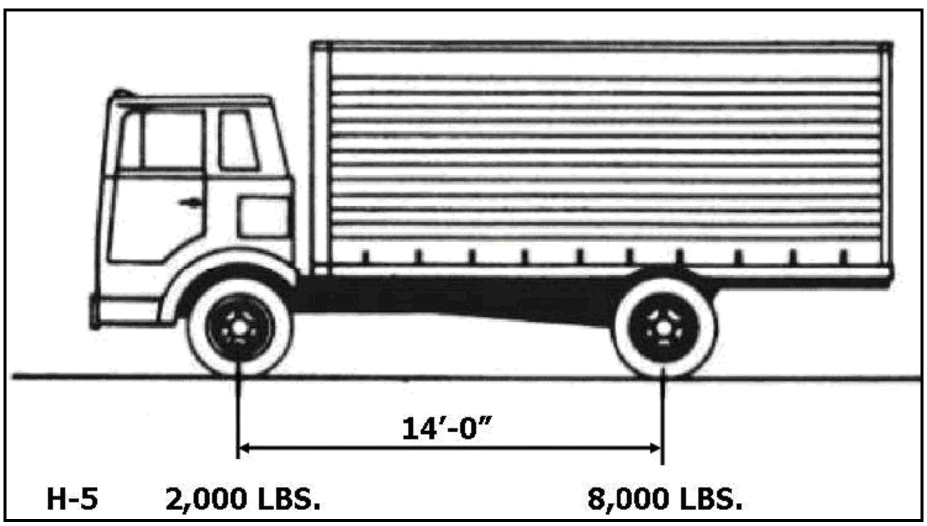

Figure 16. H5 Service Vehicle (www.dot.state.fl.us)

In order to apply the service vehicle load, the group first had to define lanes on the bridge deck that the vehicle would travel on. The group decided to define two lanes on the bridge each of which were centered at 2.5 ft from the exterior edge of the deck. These lanes are the paths on which the design vehicle will travel.

After defining the lanes on the bridge, the next step was to model the design vehicle in SAP2000. The group created a new service vehicle in the software since the H-5 vehicle was not a standard vehicle in the software. Defining the vehicle as shown in Figure 16 allows for the loads to be correctly applied to the bridge deck, and hence transferred correctly to the structure.

4.2.2.4. Wind Loading

For any structure, the force applied to it by the wind is a major concern in the design of the structure. Unlike the loads previously discussed, the wind loading is applied perpendicular to the structure, and not in the direction of the force of gravity. To determine the force from the wind, the group first had to find out what the maximum wind speed that the bridge should be designed for. Figure 17 below, shows the eastern 2/3 of the United States and the design values for wind speeds in these locations. The wind speeds shown are for a 3-second gust, and are based on the ASCE 7-02 standard.

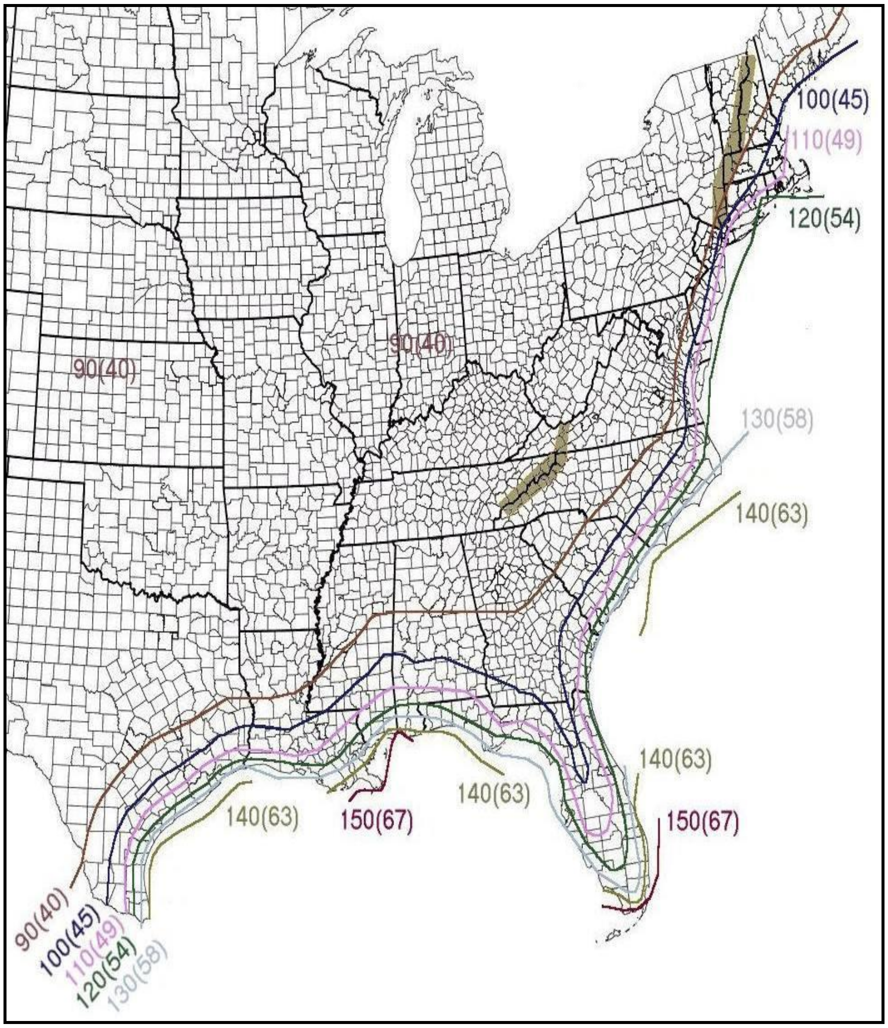

Figure 17. Design wind speeds (ASCE 7-02 Standard) (www.standarddesign.com).

Based on the map in Figure 17, the group determined that the design wind speed for the structure would be 90 mph. Applying the wind load to the bridge was easily performed in SAP2000 through the user defined loading patterns. In this menu, the group defined three different wind load conditions: WIND, WIND2, and WIND3. While entering the three conditions, the group defined each in SAP2000 as wind loads, and based the conditions on the ASCE 7-05 standards.

After entering the three load patterns, the group had to modify the wind load through the “Modify Lateral Load” tab. In this dialog box, the group could model the bridge based on characteristics of the structure. The first step was to determine what surfaces of the structure would be exposed, and in this the group entered the frame and area objects of the bridge would be exposed while the structure itself would be open. This accurately analyzes the bridge as it is to be designed over Coliseum Boulevard.

The next step in this box was to define the direction at which the wind is hitting the structure. This is what the difference between the three wind patterns is, with the angles for WIND, WIND2, and WIND3 being: 0º, 90 º, and 45 º, respectively. The final step to complete in this box is to determine the wind coefficients for the structure. As described above, the design wind speed is 90 mph based on the wind speed map. The exposure type of the bridge was defined to be "B", since the bridge would be located in an urban environment (Hibbeler).

As described in section 4.2.1.1.2 Cables, in order for the compression limit of 0 to be taken into effect, the wind loading has to be calculated using a nonlinear analysis. This step was also simple to perform in SAP2000 with the group simply having to go in the "Define Load Case" box, and then defining each of the wind load patterns to be performed at a nonlinear analysis. By doing this, the group could model the bridge cables accurately using angle members in place of actual cables.

4.2.3. Summary of Loads Applied to Structure

Table 2 shows a detailed summary of the loads applied to the structure. It is through these loads, and various combinations of them, that the final design of the structure was determined.

Table 2. Summary of loads applied to structure.

Loading Pattern	**Weight**
Dead Load	Self Weight of Structure
	Concrete = 150 pcf
	Overlay/Surface = 10 psf
	Railing/Fencing = 90 plf
Live Load	85 psf (pedestrian/snow)
Moving Load	H5 Service Vehicle = 10,000 lb
Wind Loading Conditions	
Auto Lateral Load Pattern	ASCE 7-05
Wind Speed	90 mph, 3 second gust
Exposure	B
Importance Factor	1.0
Topographical Factor, K_{zt}	1.0
Gust Factor	0.85
Directionality Factor, K_d	0.85
Solid/Gross Area Ratio	0.2

4.3. Structural Analysis

Once the bridge's geometry and the loads that it will incur are modeled in SAP2000, the next step is to perform the structural analysis on the bridge. When moving to this step, the user has the option of analyzing all, or only one, of the loading cases. The group ran all of the loading cases at one time which takes the software under 10 s to do (depends on the computer the user is working on). Following is a summary of the information the group received after SAP2000 analyzed the modeled loading conditions from section 4.2.3.

4.3.1. Deformed Shapes

As stated earlier in the software review of SAP2000, one of the easiest ways to verify if the structure is modeled correctly is to compare the deformed shape given in SAP2000 with what the user anticipates the deformed shape to be. If the deformed shape of the structure is abnormal, then the user knows that something is modeled incorrectly; however, if the deform shaped looks accurate the chances are high that the structure was modeled correctly.

An example of a deformed shape where the user realizes something is wrong happened to the group while first trying to accurately model the parabolic arch in SAP2000. After setting up the model, the group felt that everything was entered correctly. The group determined there was an error after performing the structural analysis through SAP2000 and then viewing the deformed shape of the bridge. In this window, the group saw what looked like one of the arches caving into the other arch. Additionally, the concrete deck and other members of the structure remained in place without experiencing any deformations. This was a sure sign that the structure was modeled incorrectly which made the group go back to fixing the modeling of the bridge.

The subsequent subsections display various deformations according to the different load cases that the bridge endures.

4.3.1.1. Dead and Live Loads

The deformed shapes for both the dead and live load cases are similar with the only difference being the magnitude of the deflection in each case. Figure 18 shows an XZ-plane view of the bridges deformation after being analyzed with the dead and live loads. Note the two lines running down the length of the bridge which are the user designed lanes for the service vehicles to traverse. It should be noted that all of the deformed shapes shown are not actual deformations, but are instead magnified to give the engineer an exaggerated view of how the bridge is expected to deform.

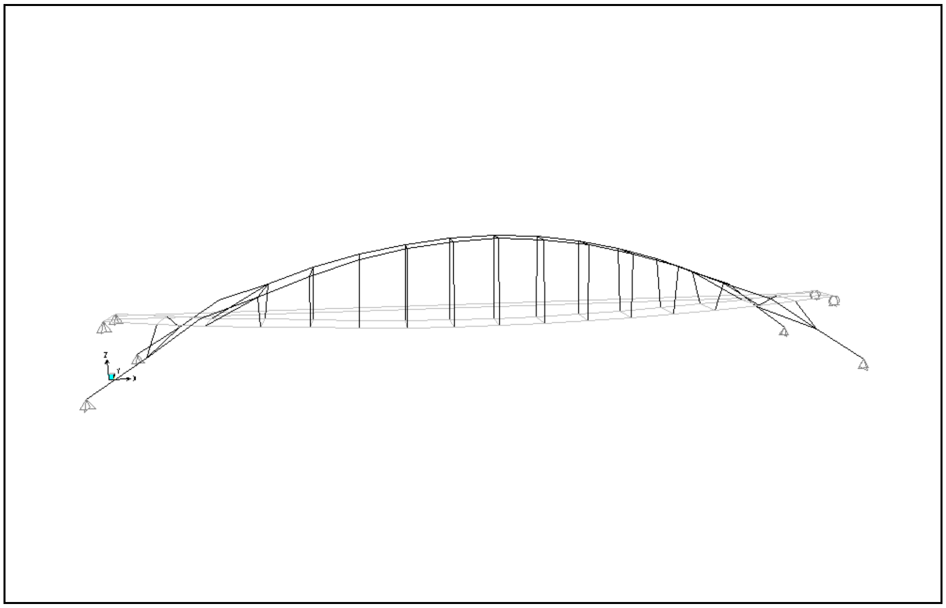

Figure 18. XZ-plane view of deformed shape under dead and live loads.

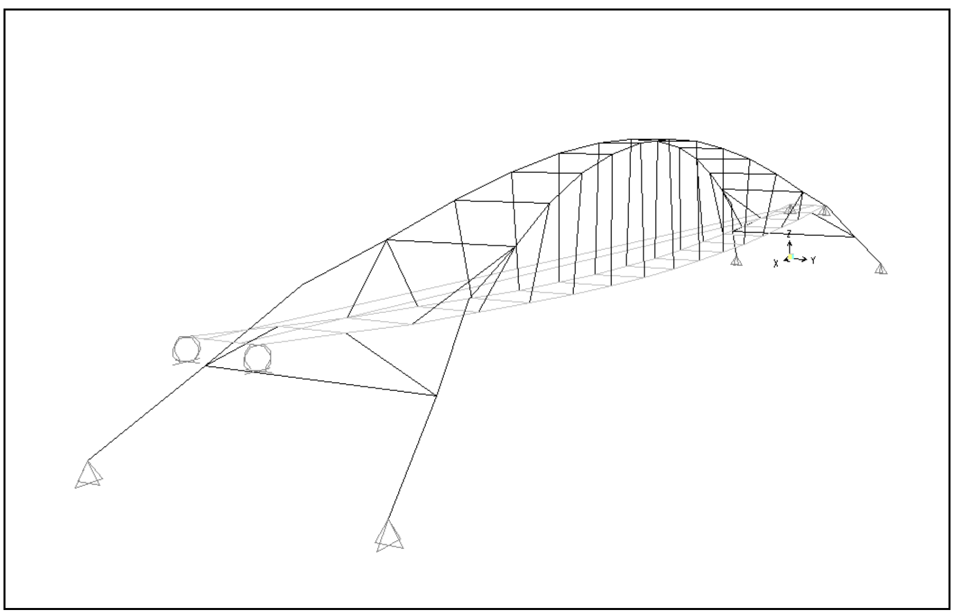

Figure 19. 3d view of the deformed bridge under dead and live loads.

4.3.1.2. Service Vehicle Load

There were two differences between the H5 and H5-2 load cases: first, the service vehicle begins its movement along the bridge at opposite ends, and second, the bridge stiffness is modified on the H5-2 load case. For the H5-2 load case, the initial stiffness of the bridge prior to the service vehicle moving across the bridge was taken to be the stiffness of the bridge at the end of the load case WIND. By applying the load pattern this way, it helps model the bridge as if a moving vehicle load is on the bridge during high wind conditions. It will be shown later, that this load case controls the steel design of the structure based upon the fatigue loading introduced to the structure during this load case.

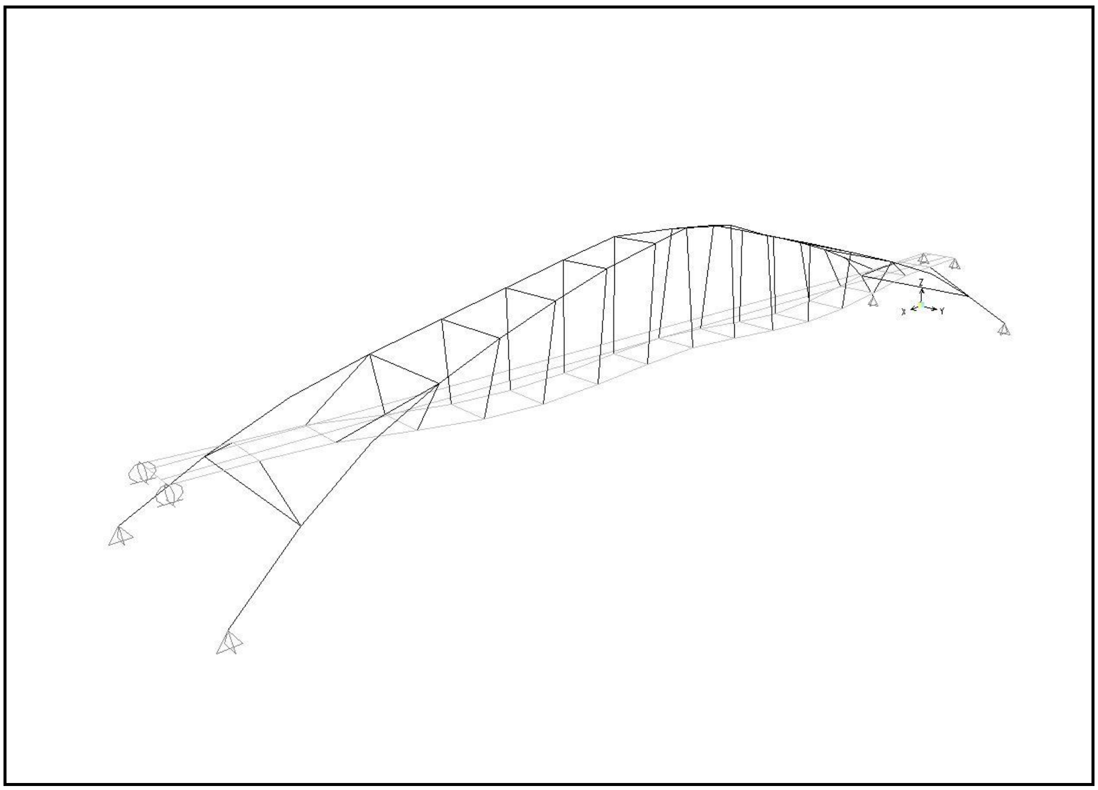

Figure 20. Deformed shape for both the H5 and H5-2 load cases.

4.3.1.3. Wind Loading

The various wind loading conditions WIND, WIND2, and WIND3 all have different deformations associated with them since each case represents a different angle at which the wind acts on the bridge. Angles of loading for WIND, WIND2, and WIND3 are 0º, 90 º, and 45 º, respectively. Following are some screen captures from SAP2000 for the different wind loading cases.

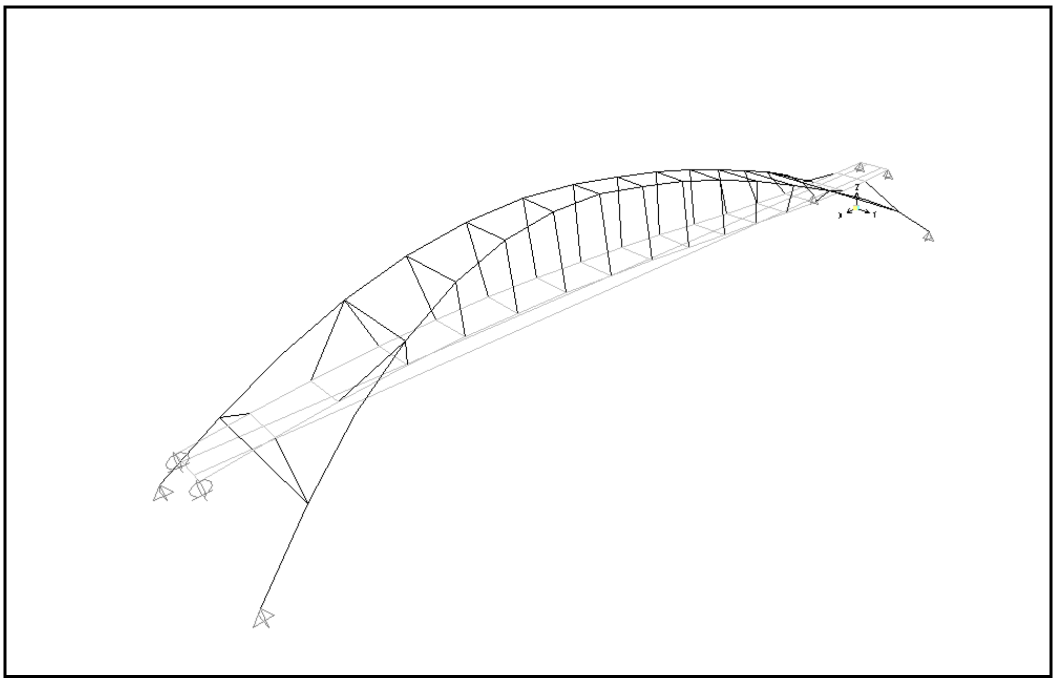

Figure 21. Deformed view for WIND.

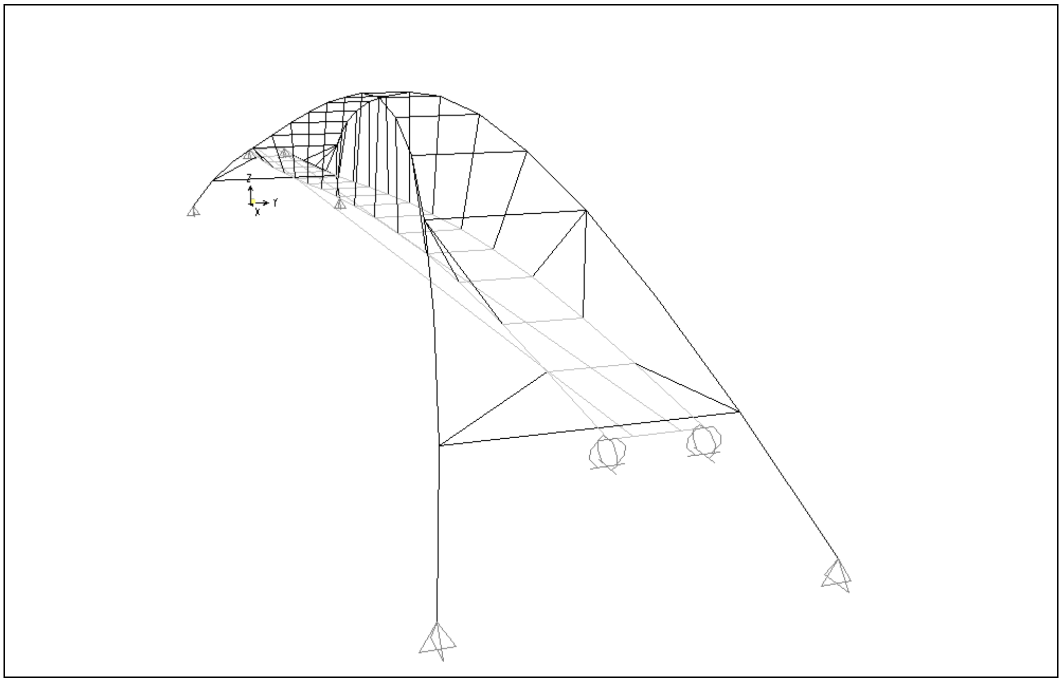

Figure 22. Deformed view for WIND looking down length of bridge.

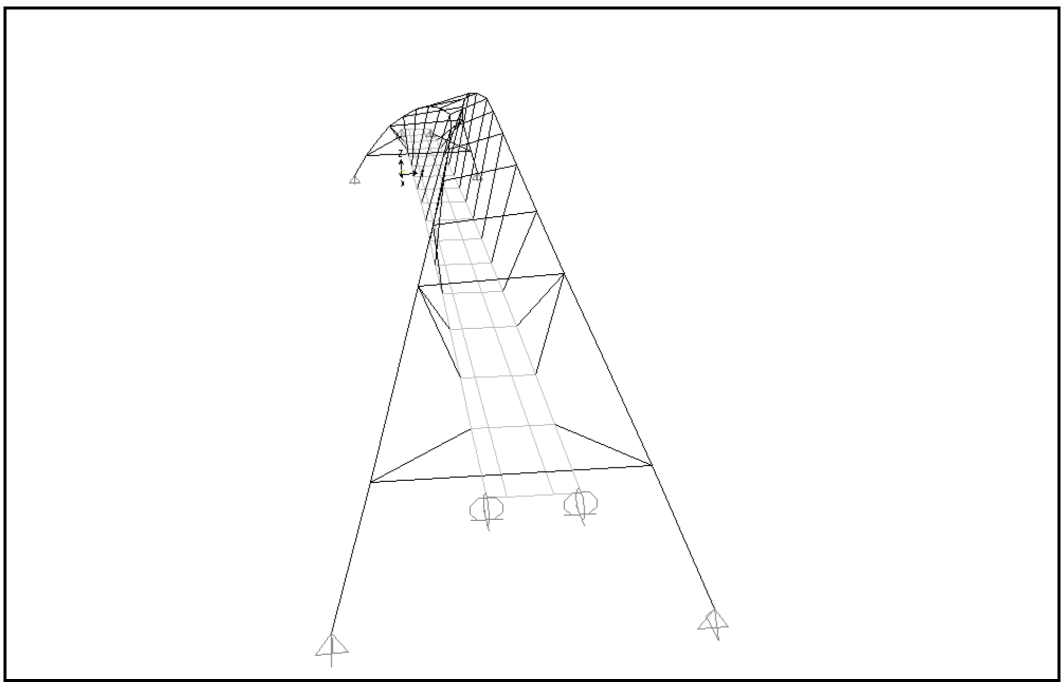

Figure 23. Deformed view for WIND2 looking down length of bridge.

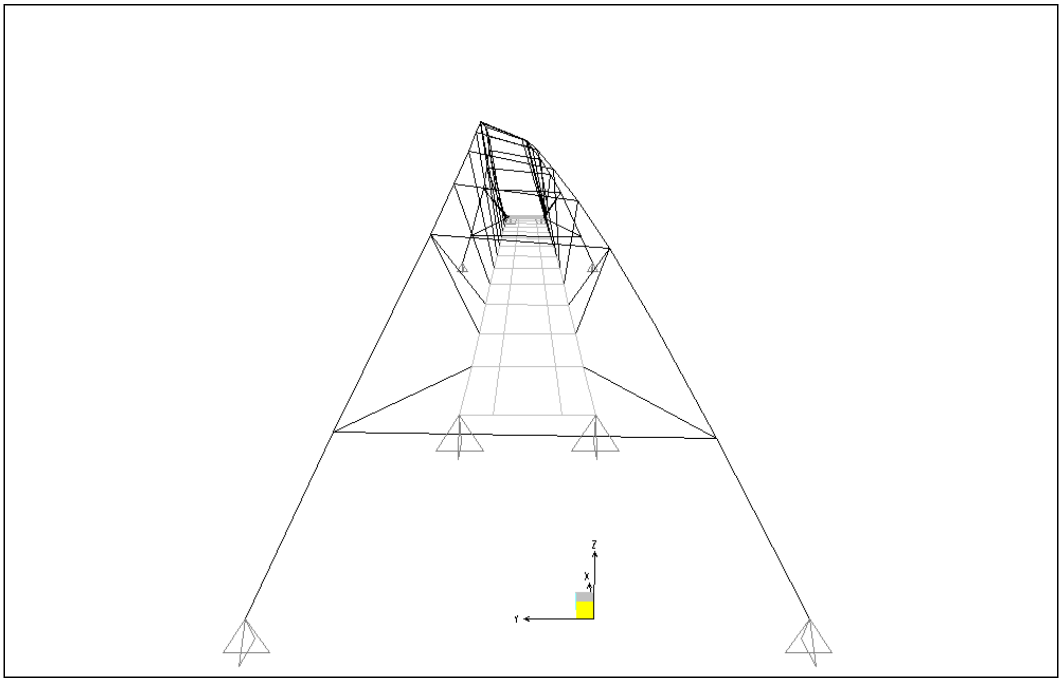

Figure 24. Deformed view for WIND3 looking down the walkway of the structure.

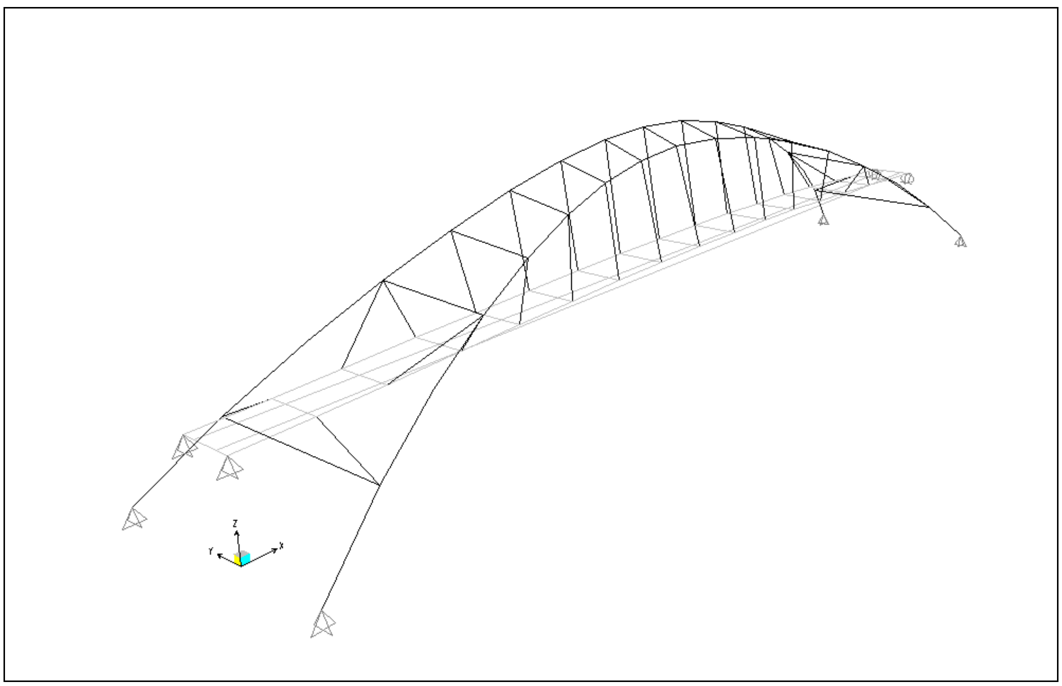

Figure 25. Side view of the deformed shape from WIND3.

4.3.2. Joint Loading

After viewing the deformed shapes for the various loading conditions, the next option was to view the loads at the exterior joints. The following screen shots from SAP2000 display the forces at the pinned connections supporting the arch. Each of the figures is labeled according to the corresponding loading condition.

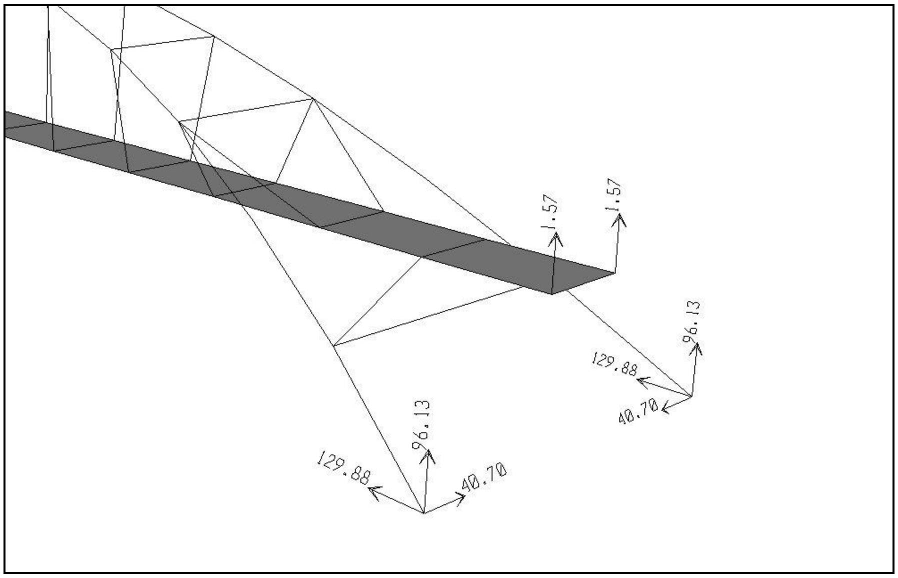

Figure 26. Joint reactions for DEAD load case (symmetric at each end).

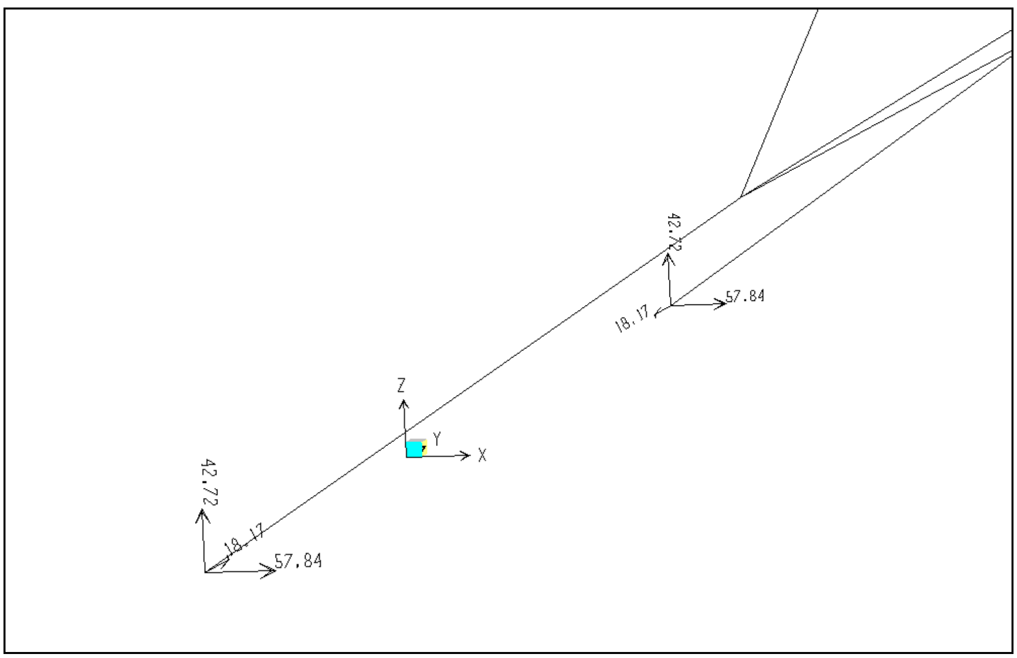

Figure 27. Joint reactions corresponding to LIVE load case (symmetric at each end).

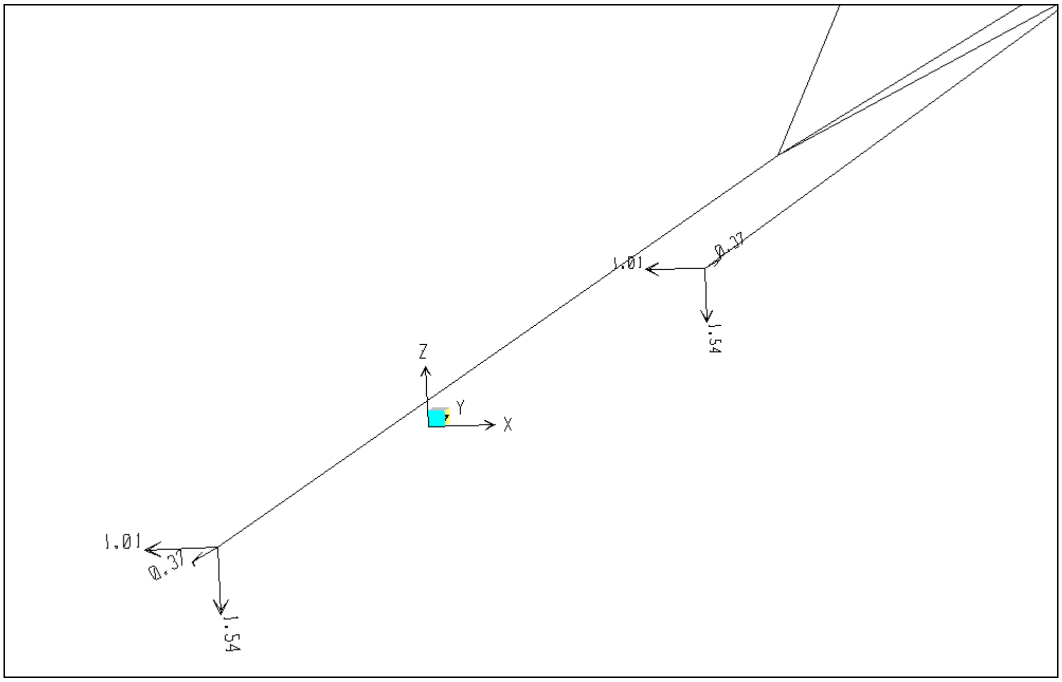

Figure 28. Joint reactions at starting end for WIND loading case.

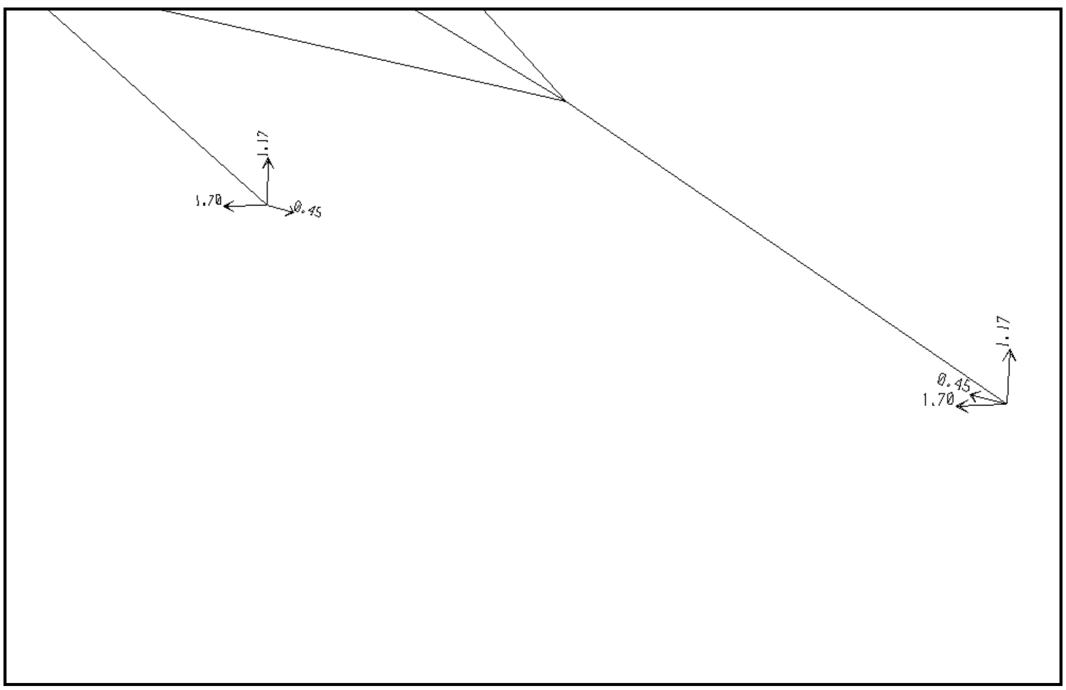

Figure 29. Joint reactions at the end of the arch for WIND loading case.

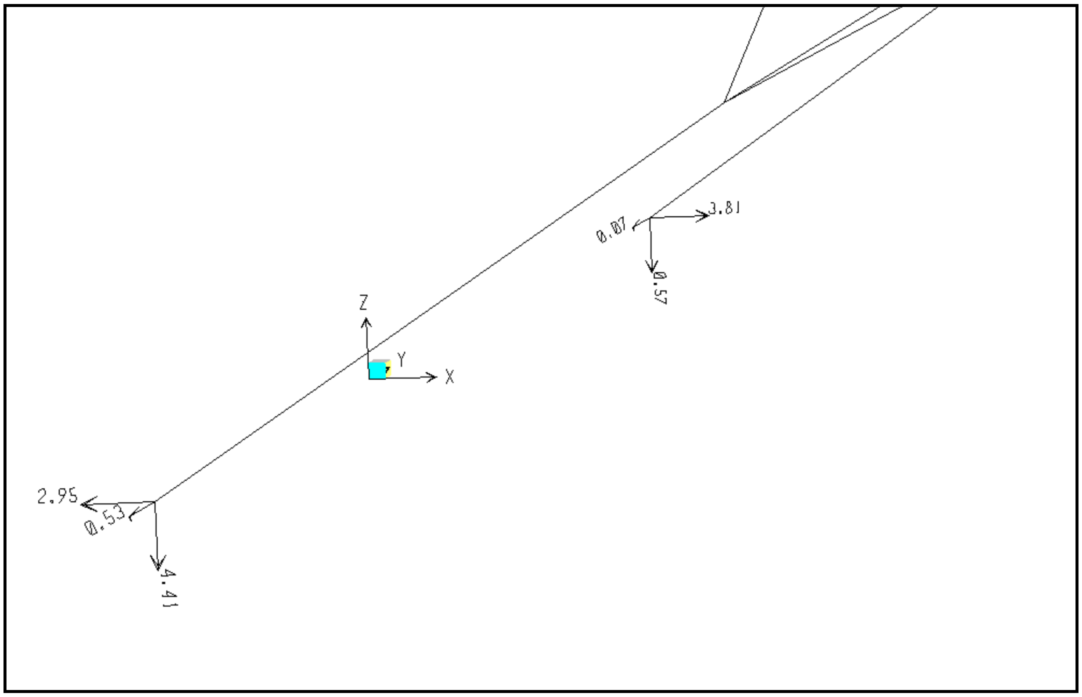

Figure 30. Joint reactions at the start of bridge span for WIND2 load case.

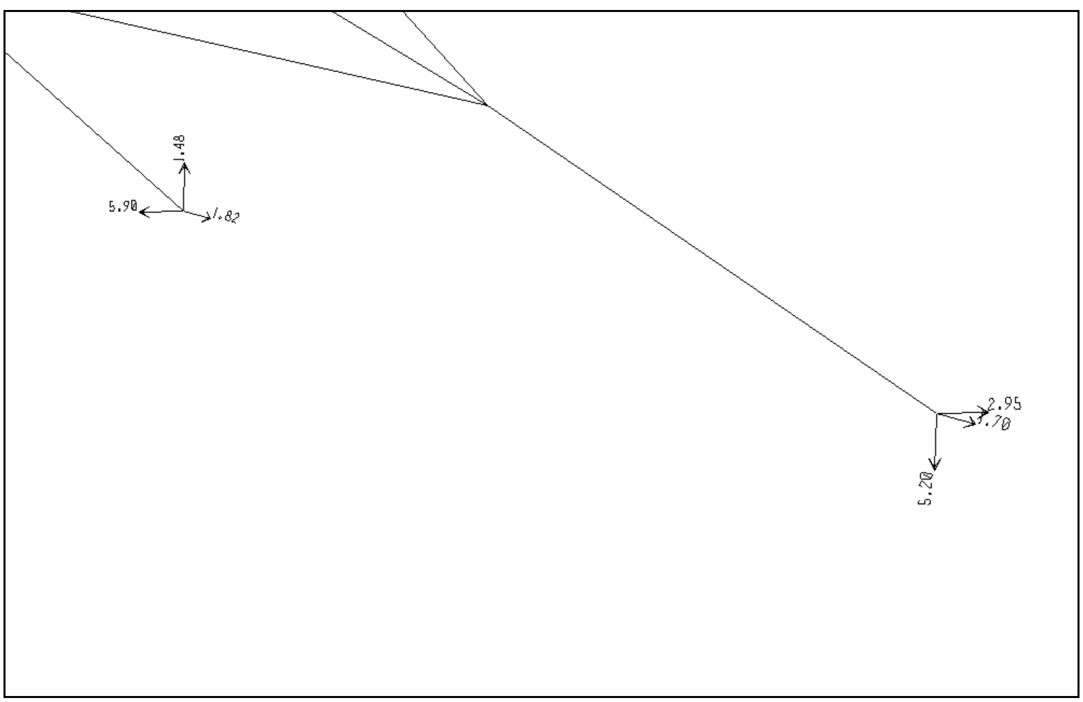

Figure 31. Joint reactions at end of span for WIND2 load case.

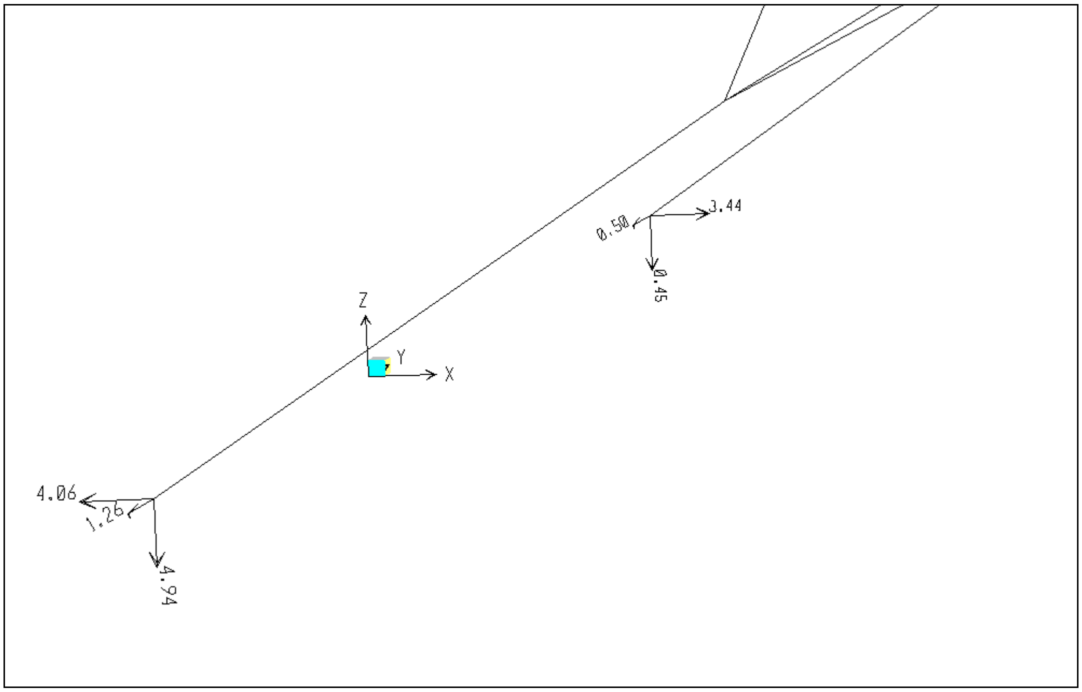

Figure 32. Joint reactions at the start of the span for WIND3 load case.

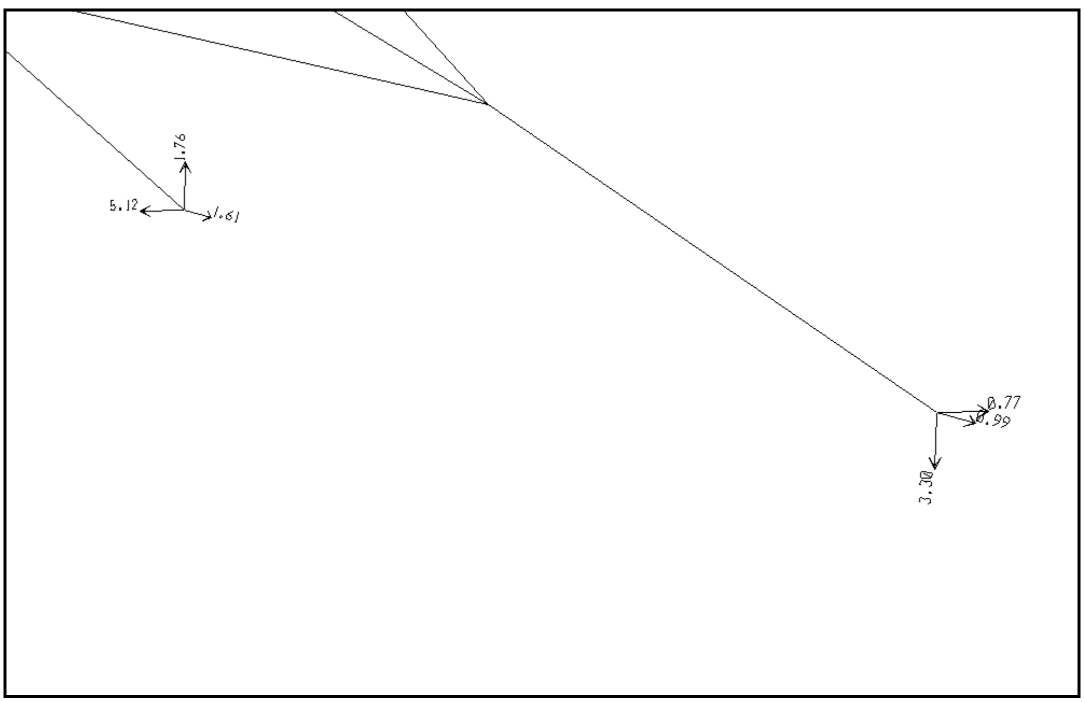

Figure 33. Joint reactions at the end of the span for WIND3 load case.

4.3.3. Frame/Cable Loads

Another great feature of SAP2000 is once the analysis is completed, the user can view the forces that each member of the structure is experiencing. In the following figures, compression forces are in red whereas tension forces are in yellow. A quick check of the cable members shows that they all do carry tension loads only, so the group was able to easily identify that the compression limit of zero was executed correctly. As viewed in the close up pictures of the structure, as per the view is set up, the forces are shown in accordance to their magnitude and have the maximum force labeled on the diagram.

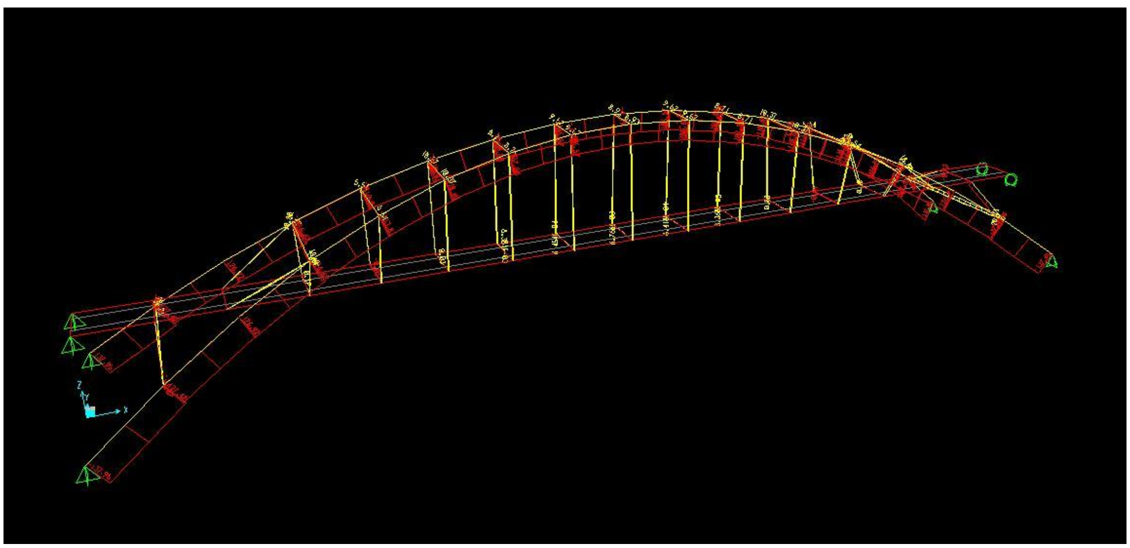

Figure 34. Overview of frame forces from DEAD load case.

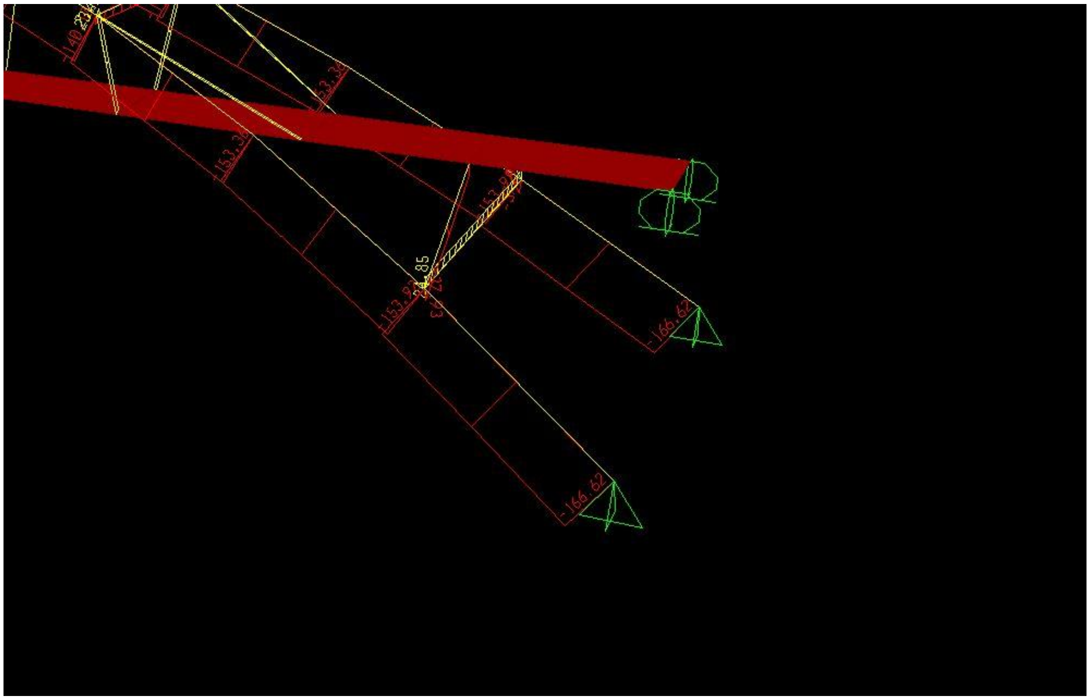

Figure 35. Frame axial force for DEAD load case at the base.

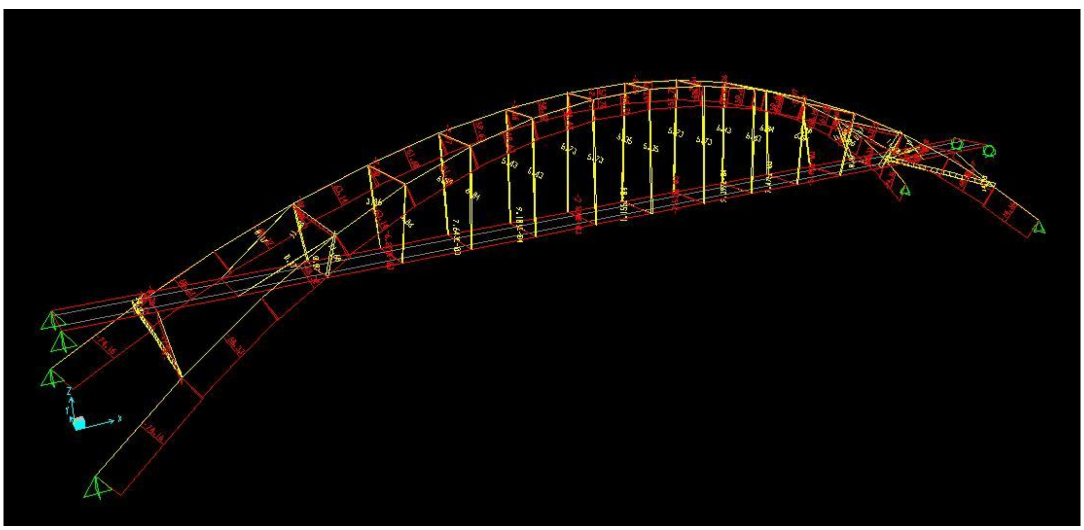

Figure 36. Overview of frame forces for LIVE load case.

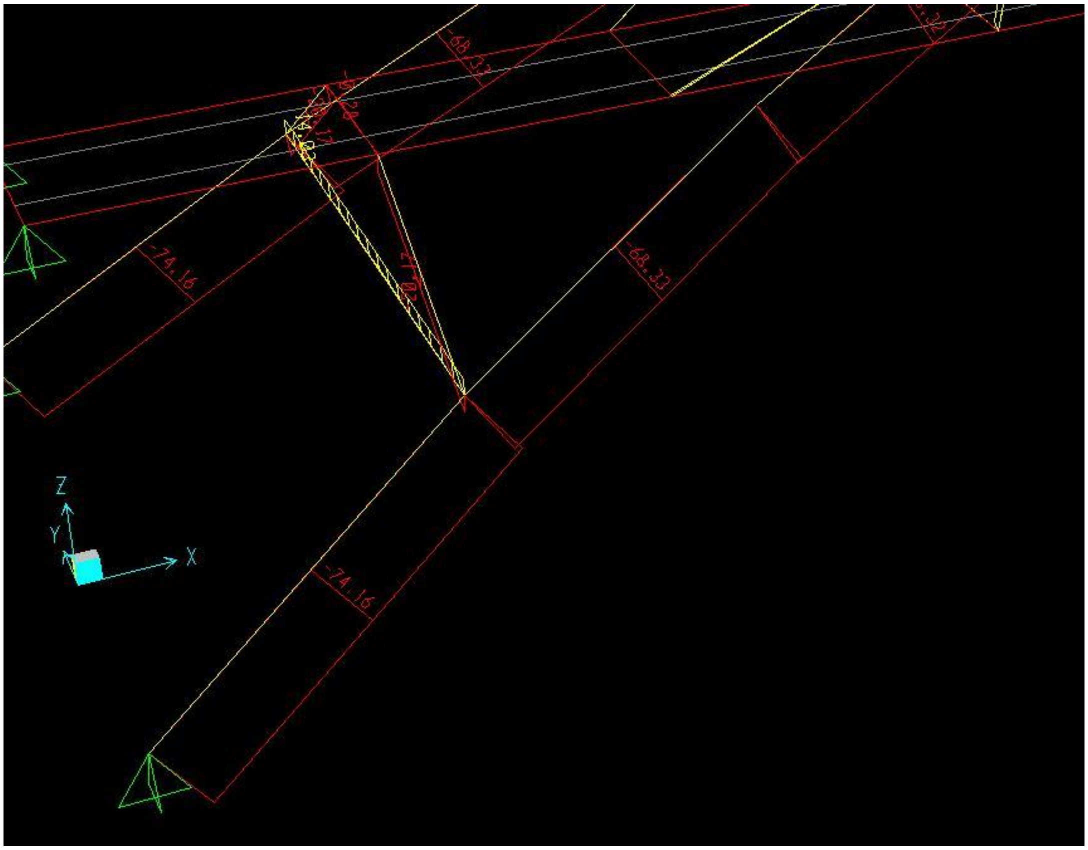

Figure 37. Close up view of frame forces for LIVE load case.

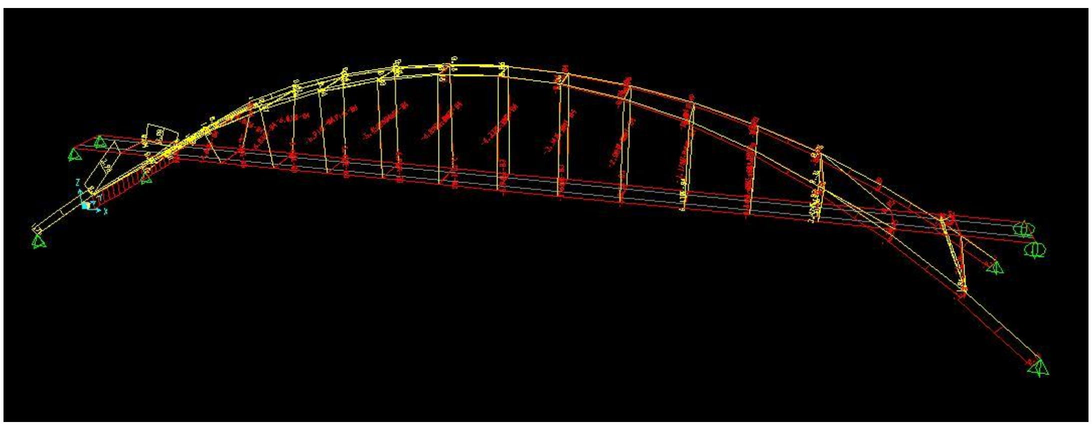

Figure 38. Overview of frame forces from WIND load case.

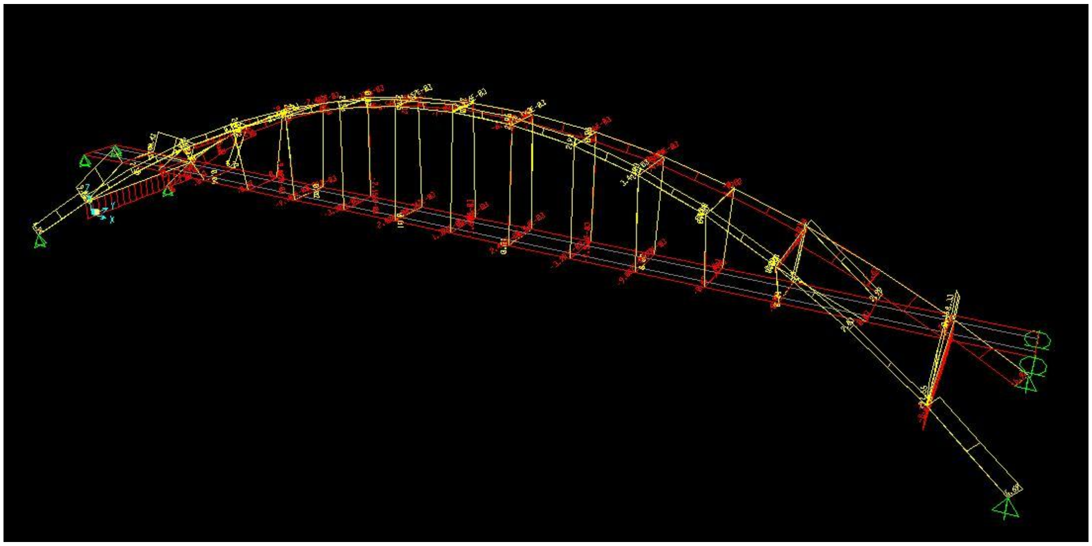

Figure 39. Overview of frame forces from WIND2 load case.

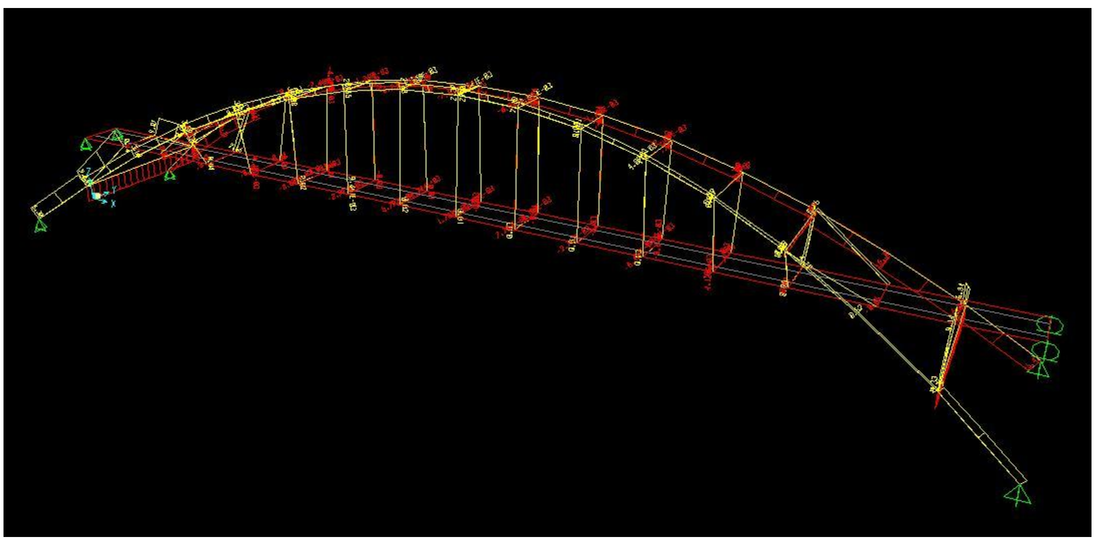

Figure 40. Overview of frame forces from WIND3 load case.

4.3.4. Shell Stresses

Another display option is actually viewing the intensity distributions for the area in the model which in this case is the concrete deck. Figure 40 shows the uniform loads in different intensities by using various colors for the different strengths of the force. In this figure, the maximum force is in the color blue whereas the minimum force is shown in a light green. For each loading case, there are numerous options for displaying the area forces which includes displaying shell forces, shell stresses, and resultant forces for each component of the shell along with the different layers in the shell. Figure 40 was used just for an idea of the shell force display since many figures could be incorporated into the report.

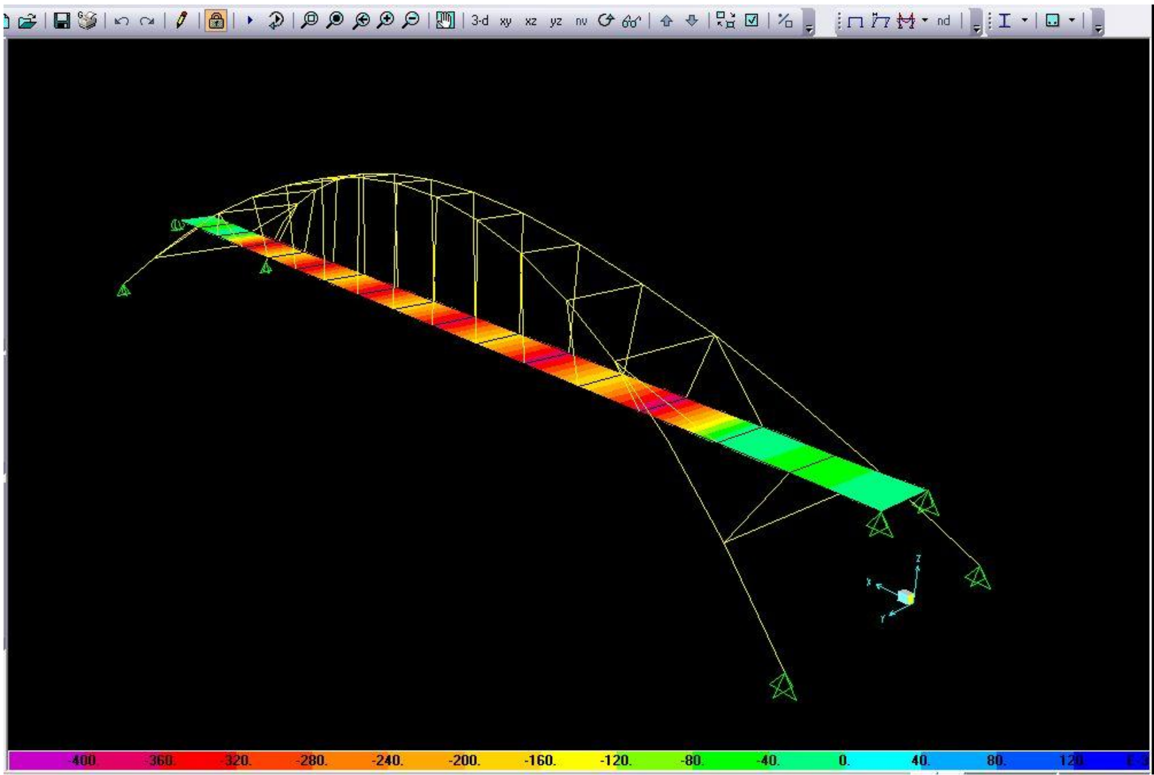

Figure 41. Maximum shell stress in concrete deck (scale is in kip).

4.3.5. Influence Lines

Yet another powerful tool with SAP2000 is its ability to easily display influence lines for joints and structural members. An influence line, "represents the variation of either the reaction, shear, moment, or deflection at a specific joint in a member as a concentrated force moves on the member" (Hibbeler). A quick glance at this line serves as an aide to where on the structure the member is most affected by the moving load. The influence line is constructed by calculating the mechanical behavior of the structure when a unit load traverses the structure. The following subsections show influence lines for various members of the bridge, with the member and mechanical behavior shown in the figure's caption.

4.3.5.1. Joint

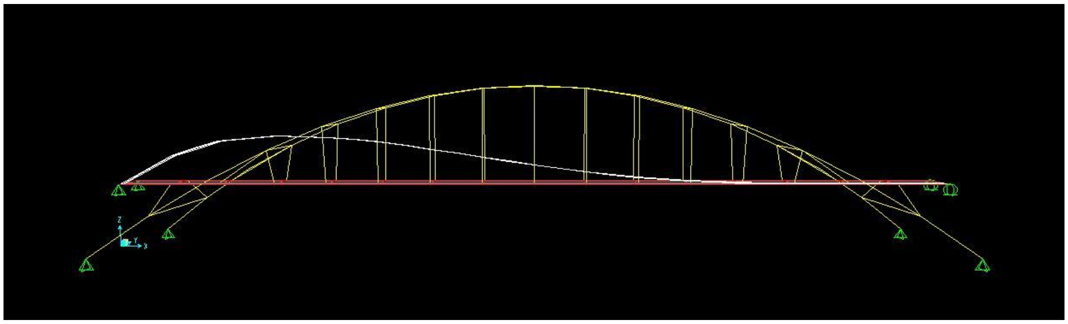

Figure 42. Influence line for the joint reaction at the start of the span.

4.3.5.2. Cable

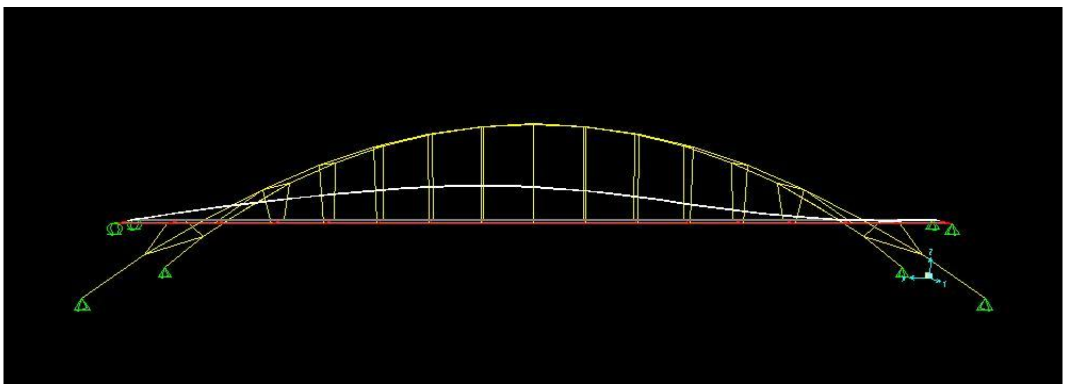

Figure 43. Influence line for the axial force in the cable member at the mid-span.

4.3.5.3. Arch

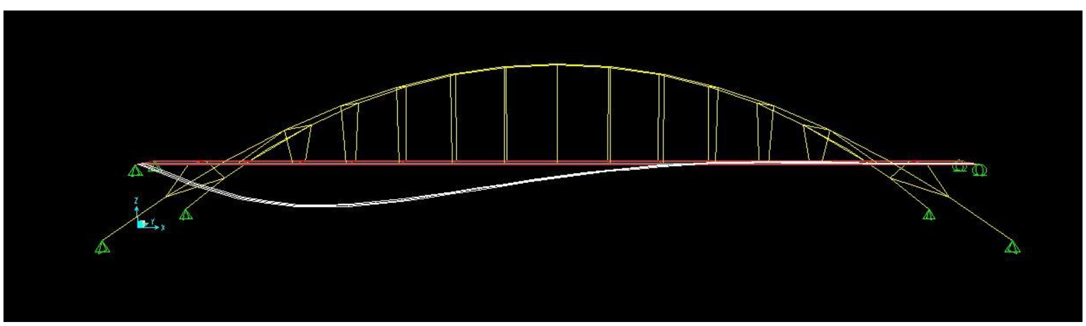

Figure 44. Influence line for axial force for 2nd arch member in from start of span.

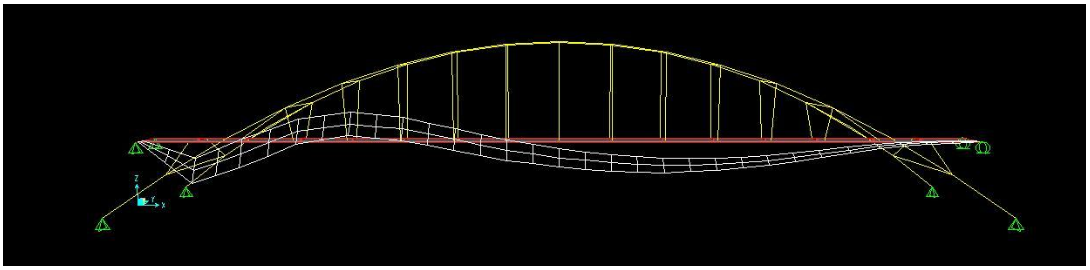

Figure 45. Influence line for moment force for 2nd arch member in from start of span.

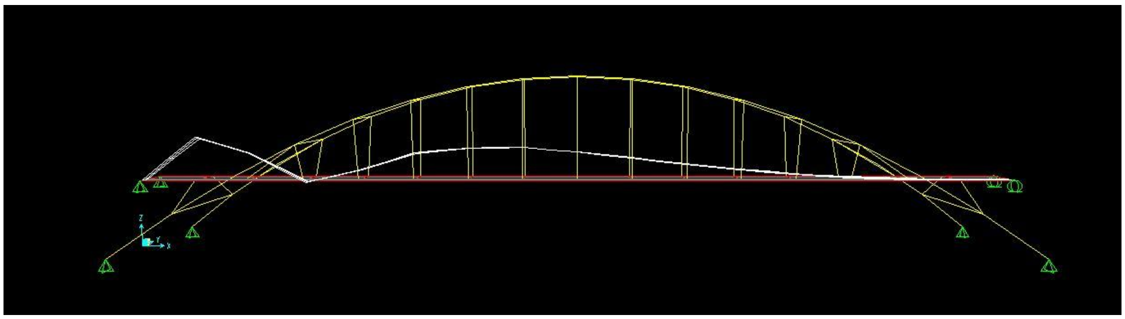

Figure 46. Influence line for shear force for 2nd arch member in from start of span.

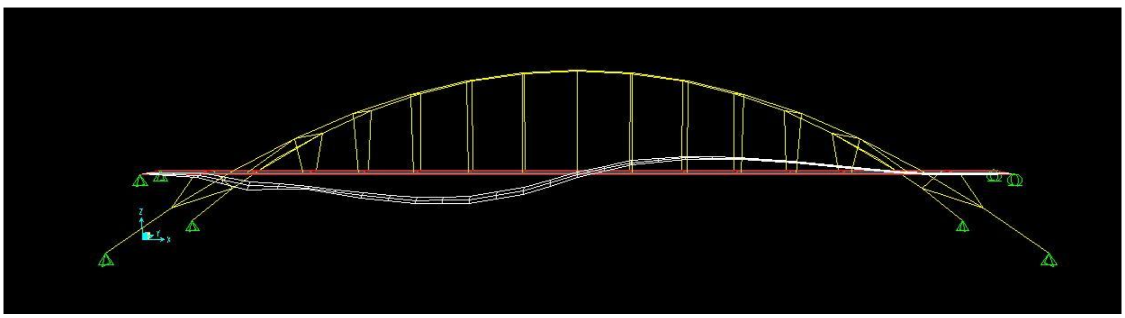

Figure 47. Influence line for torsion force for 2nd arch member in from start of span.

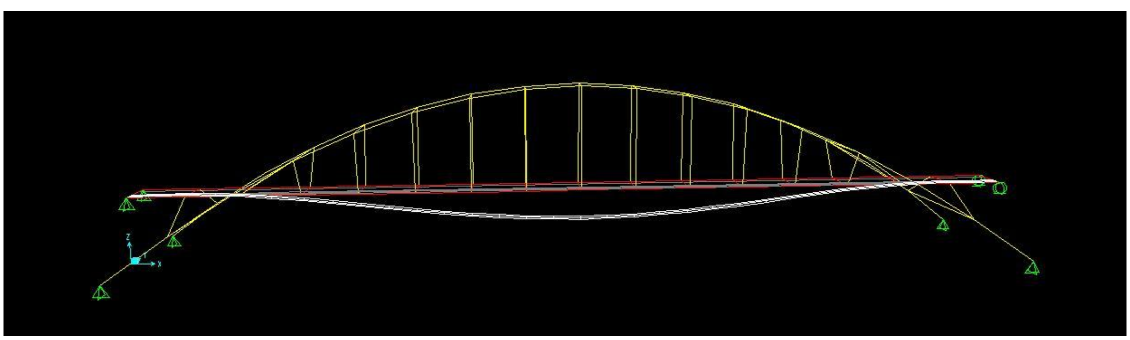

Figure 48. Influence line for axial force for frame member at apex of the arch.

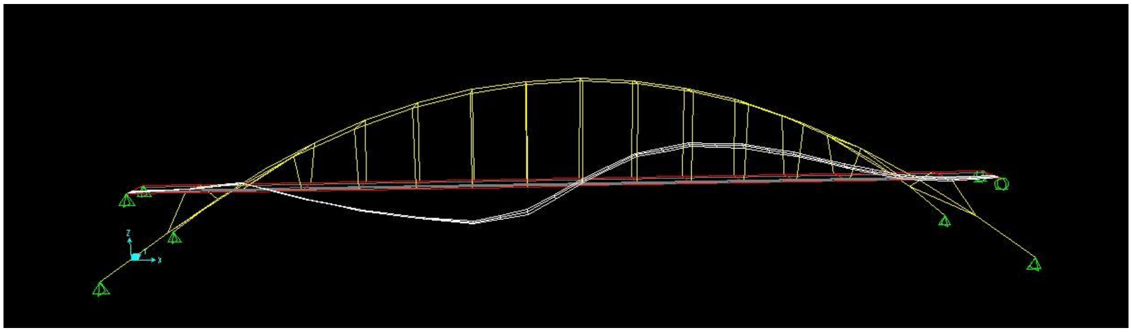

Figure 49. Influence line for moment force for frame member at apex of the arch.

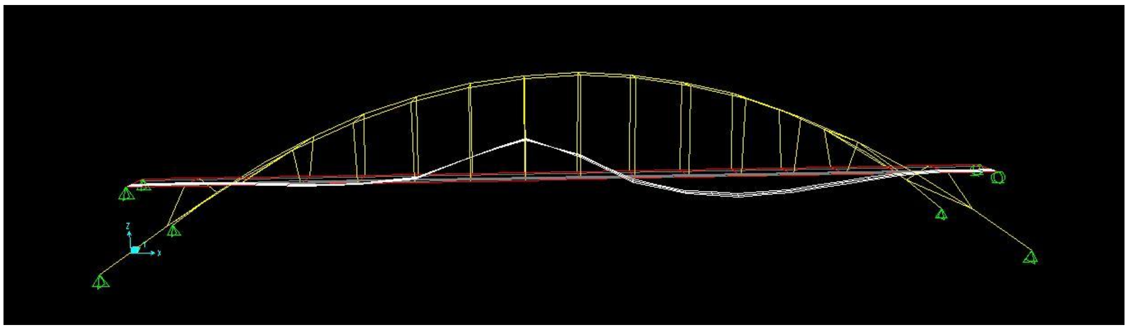

Figure 50. Influence line for shear force for frame member at apex of the arch.

4.4. Structural Design

4.4.1. Design Load Combinations

The group used the default load design combinations for bridges that are saved in the SAP2000 program to perform the steel design of the pedestrian bridge. These combinations adhere to the AASHTO-LRFD design combinations used by DOT's around the country. As can be seen by the different names of the load case combinations, some of the combinations are used for strength design while others are used to determine the design per serviceability issues, and some others are designed for fatigue of the structure. Table 3 names the combinations as well as details the load combinations used for each case.

Of the 150 different load case combinations, the one predominately used in the steel design structure is DSTL2 which multiplies the dead load by a factor of 1.2 and the live load by a factor of 1.75; however, in some instances, the controlling load case combination is due to fatigue loading. As can be seen in the deformed shape videos, and drawings, there can be large deformations found in the arch from the portion where the deck rests on top of the arch to where the first lateral support is located. When performing in advanced dynamic analysis of the structure, it was found that fatigue loading would control the steel design of the arch for the H5 service vehicle load. This was due to the fact that the group analyzed the H5 moving load by using the stiffness found at the end of the nonlinear wind loading. Performing this analysis leads the size of the arch to be increased from an HSS 16 x 0.375 to HSS 18 x 0.375. Through these load increases, the structure can safely be designed in accordance with the LRFD specifications.

Table 3. Design load combinations used for the steel design of the bridge.

Load Case Combination	Scale Factor
DSTL1	1.4D
DSTL2	1.2D + 1.75L
DSTL3	1.2D + 1.6L
DSTL4	1.0D
DSTL5	1.0D + 1.0L
STR-I1	1.25D
STR-I2	0.9D
STR-I3	1.25D + 1.75MODAL
STR-I4	1.25D + 1.75H5
STR-I5	1.25D + 1.75H5-2
STR-I6	0.9D + 1.75MODAL
STR-I7	0.9D + 1.75H5
STR-I8	0.9D + 1.75H5-2
STR-II1	1.25D
STR-II2	0.9D
STR-II3	1.25D + 1.35MODAL
STR-II4	1.25D + 1.35H5
STR-II5	1.25D + 1.35H5-2
STR-II6	0.9D + 1.35MODAL
STR-II7	0.9D + 1.35H5
STR-II8	0.9D + 1.35H5-2
STR-III1	1.25D + 1.4WIND
STR-III2	1.25D + 1.4WIND2
STR-III3	1.25D - 1.4WIND
STR-III4	1.25D - 1.4WIND2
STR-III5	0.9D + 1.4WIND
STR-III6	0.9D + 1.4WIND2
STR-III7	0.9D - 1.4WIND
STR-III8	0.9D - 1.4WIND2

Load Case Combination	Scale Factor
STR-III9	0.9D + 1.4WIND3
STR-III10	1.25D + 1.4WIND
STR-III11	1.25D + 1.4WIND2
STR-III12	1.25D + 1.4WIND3
STR-III13	1.25D - 1.4WIND
STR-III14	1.25D - 1.4WIND2
STR-III15	1.25D - 1.4WIND3
STR-III16	0.9D + 1.4WIND
STR-III17	0.9D + 1.4WIND2
STR-III18	0.9D + 1.4WIND3
STR-III19	0.9D - 1.4WIND
STR-III20	0.9D - 1.4WIND2
STR-III21	0.9D - 1.4WIND3
STR-IV1	1.5D
STR-IV2	0.9D
STR-IV3	1.25D
STR-IV4	0.9D
STR-V1	1.25D + 0.4WIND
STR-V2	1.25D + 0.4WIND2
STR-V3	1.25D - 0.4WIND
STR-V4	1.25D - 0.4WIND2
STR-V5	0.9D + 0.4WIND
STR-V6	0.9D + 0.4WIND2
STR-V7	0.9D - 0.4WIND
STR-V8	0.9D - 0.4WIND2
STR-V9	1.25D + 0.4WIND + 1.35MODAL
STR-V10	1.25D + 0.4WIND + 1.35H5
STR-V11	1.25D + 0.4WIND + 1.35H5-2
STR-V12	1.25D +0.4WIND2 + 1.35MODAL
STR-V13	1.25D + 0.4WIND2 + 1.35H5
STR-V14	1.25D + 0.4WIND2 + 1.35H5-2
STR-V15	1.25D + 0.4WIND3 + 1.35MODAL
STR-V16	1.25D + 0.4WIND3 + 1.35H5
STR-V17	1.25D + 0.4WIND3 + 1.35H5-2
STR-V18	1.25D - 0.4WIND + 1.35MODAL
STR-V19	1.25D - 0.4WIND + 1.35H5
STR-V20	1.25D - 0.4WIND + 1.35H5-2
STR-V21`	1.25D - 0.4WIND2 + 1.35MODAL
STR-V22	1.25D - 0.4WIND2 + 1.35H5
STR-V23	1.25D - 0.4WIND2 + 1.35H5-2
STR-V24	1.25D - 0.4WIND3 + 1.35MODAL
STR-V25	1.25D - 0.4WIND3 + 1.35H5
STR-V26	1.25D - 0.4WIND3 + 1.35H5-2
STR-V27	0.9D + 0.4WIND + 1.35MODAL
STR-V28	0.9D + 0.4WIND + 1.35H5
STR-V29	0.9D + 0.4WIND + 1.35H5-2
STR-V30	0.9D + 0.4WIND2 + 1.35MODAL

Load Case Combination	Scale Factor
STR-V31	0.9D + 0.4WIND2 + 1.35H5
STR-V32	0.9D + 0.4WIND2 + 1.35H5-2
STR-V33	0.9D + 0.4WIND3 + 1.35MODAL
STR-V34	0.9D + 0.4WIND3 + 1.35H5
STR-V35	0.9D + 0.4WIND3 + 1.35H5-2
STR-V36	09.D - 0.4WIND + 1.35MODAL
STR-V37	09.D - 0.4WIND + 1.35H5
STR-V38	0.9D - 0.4WIND + 1.35H5-2
STR-V39	09.D - 0.4WIND2 + 1.35MODAL
STR-V40	0.9D - 0.4WIND2 + 1.35H5
STR-V41	0.9D - 0.4WIND2 + 1.35H5-2
STR-V42	09.D - 0.4WIND3 + 1.35MODAL
STR-V43	09.D - 0.4WIND3 + 1.35H5
STR-V44	0.9D - 0.4WIND3 + 1.35H5-2
EE-I1	1.25D
EE-I2	0.9D
EE-I3	1.25D + 1.0MODAL
EE-14	1.25D + 1.0H5
EE-I5	1.25D + 1.0H5-2
EE-I6	0.9D + 1.0MODAL
EE-I7	09D + 1.0H5
EE-I8	0.9D + 1.0H5-2
EE-II1	1.25D
EE-II3	1.25D + 1.0MODAL
EE-II4	1.25 + 1.0H5
EE-II5	1.25D + 1.0H5-2
EE-II6	0.9D + 1.0MODAL
EE-II7	0.9D + 1.0H5
EE-II8	0.9 + 1.0H5-2
SER-I1	1.0D + 0.3WIND
SER-I2	1.0D + 0.3WIND2
SER-I3	1.0D - 0.3WIND
SER-I4	1.0D - 0.3WIND2
SER-I5	1.0D + 0.4WIND3
SER-I6	1.0D + 0.3WIND + 1.0MODAL
SER-I7	1.0D + 0.3WIND + 1.0H5
SER-I8	1.0D + 0.3WIND + 1.0H5-2
SER-I9	1.0D + 0.3WIND2 + 1.0MODAL
SER-I10	1.0D + 0.3WIND2 + 1.0H5
SER-I11	1.0D + 0.3WIND2 + 1.0H5-2
SER-I12	1.0D + 0.3WIND3 + 1.0MODAL
SER-I13	1.0D + 0.3WIND3 + 1.0H5
SER-I14	1.0D + 0.3WIND3 + 1.0H5-2
SER-I15	1.0D - 0.3WIND + 1.0MODAL
SER-I16	1.0D - 0.3WIND + 1.0H5
SER-I17	1.0D - 0.3WIND + 1.0H5-2

Load Case Combination	Scale Factor
SER-I18	1.0D - 0.3WIND2 + 1.0MODAL
SER-I19	1.0D - 0.3WIND2 + 1.0H5
SER-I20	1.0D - 0.3WIND2 + 1.0H5-2
SER-I21	1.0D - 0.3WIND3 + 1.0MODAL
SER-I22	1.0D - 0.3WIND3 + 1.0H5
SER-I23	1.0D - 0.3WIND3 + 1.0H5-2
SER-II1	1.0D
SER-II2	1.0D + 1.3MODAL
SER-II3	1.0D + 1.3H5
SER-II4	1.0D + 1.35H5-2
SER-III2	1.0D + 0.8MODAL
SER-III3	1.0D + 0.8H5
SER-III4	1.0D + 0.8H5-2
SER-IV1	1.0D + 0.7WIND
SER-IV2	1.0D + 0.7WIND2
SER-IV3	1.0D - 0.7WIND
SER-IV4	1.0D - 0.7WIND2
SER-IV5	1.0D + 0.7WIND3
SER-IV6	1.0D - 0.7WIND3
SER-IV7	1.0D + 0.7WIND
SER-IV8	1.0D + 0.7WIND2
SER-IV9	1.0D + 0.7WIND3
SER-IV10	1.0D - 0.7WIND
SER-IV11	1.0D - 0.7WIND2
SER-IV12	1.0D - 0.7WIND3
FAT1	0.75MODAL
FAT2	0.75H5
FAT3	0.75H5-2

4.4.2. Design Members

4.4.2.1. Arch Members

Using the steel design function in SAP2000, the group adequately sized the steel arch members for the pedestrian bridge. Since the arch members were initially designed as an automatically selected HSS member, when the design commences the program optimizes the design members in accordance to the load case combinations shown in Table 3. As shown in Figure 51, the maximum axial force in the arch members was calculated to be 329.93 kip (compression). In order to support this load, the software selected an HSS 18 x 0.375 to be used as the main structural shapes for arch members.

As stated previously, when using only the DSTL2 loads, the arch was designed for an HSS 16 x 0.375, but by performing an advanced dynamic analysis of the moving vehicle load, SAP2000 designed the arch members to be the larger 18" diameter HSS. Figure 52 shows this design.

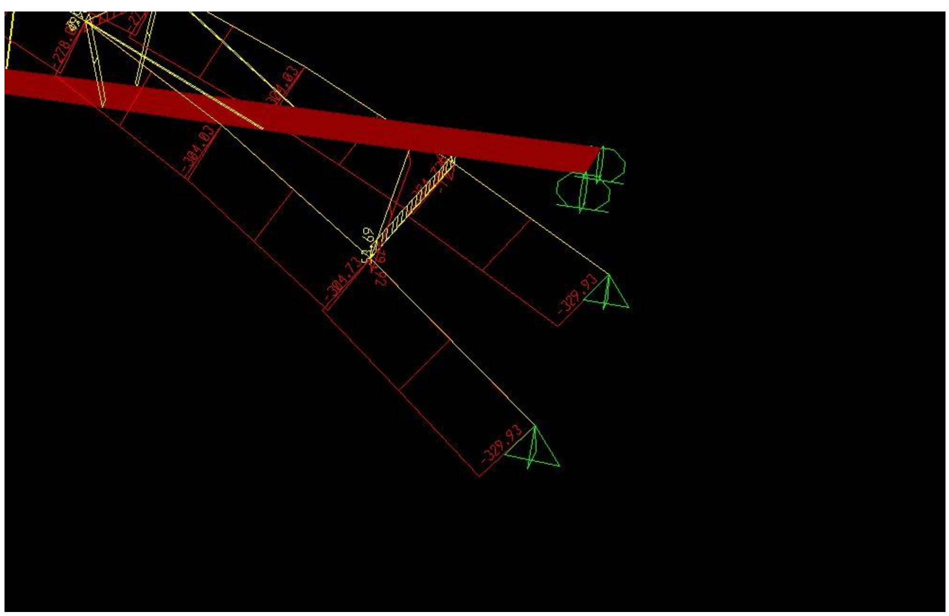

Figure 51. Maximum axial force from DSTL2.

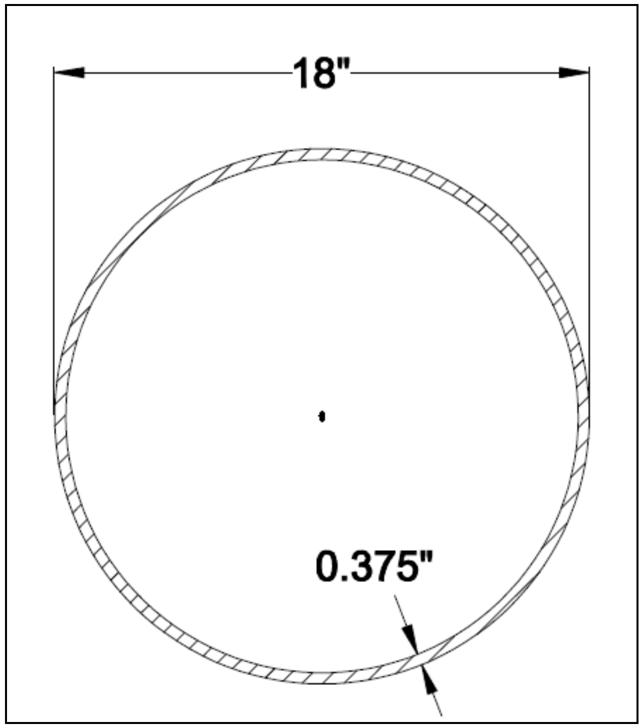

Figure 52. Typical section of arch member.

4.4.2.2. Cross Members on Arch

When designing the structure, the group allowed SAP2000 to optimize all of the members used in the arch. During this steel design, the program designed the cross members that are used for the lateral support to be designed at the same dimensions as what the arch members are, HSS 18 x 0.375. Although this size of structural shape is way more than adequate for the loads that are in the cross supports, the group determined that it was aesthetically pleasing to use the same size for these members as what is found in the main arch supports.

4.4.2.3. Cables

Since the cable members were modeled as angle members in SAP2000, the group needed to perform hand calculations to determine the size of cables needed to support the bridge deck. The maximum force applied to the cables was determined in SAP2000 and used for the calculations that are detailed in the Appendix. Based upon a maximum tensile force of 26.719 kips, which is from the Load Case DSTL2, the required diameter of the cable members is 1 in.

4.4.3. Slab Design

As described in the *Bridge Engineering Handbook*, loads applied to the slab can be distributed to effective slab widths which can then be analyzed as a simply supported beam. By doing this, the group was able to perform a set of hand calculations to determine all relevant design information for the concrete deck. These hand calculations are shown in the Appendix.

Using the calculations from the Appendix shows that the deck should be constructed of a 6” deep concrete slab. The reinforcement steel required for the longitudinal direction is No. 3 rebar, placed 7” o.c. while the reinforcement steel that is needed to control the shrinkage and temperature was calculated to be No. 3 rebar, placed 10” o.c.

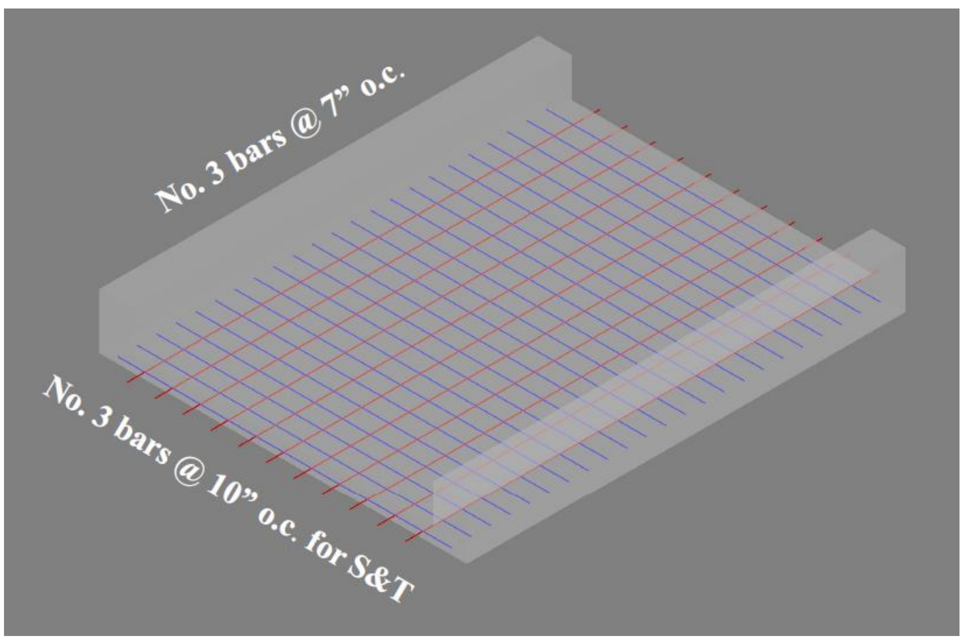

Figure 53. Typical rebar spacing.

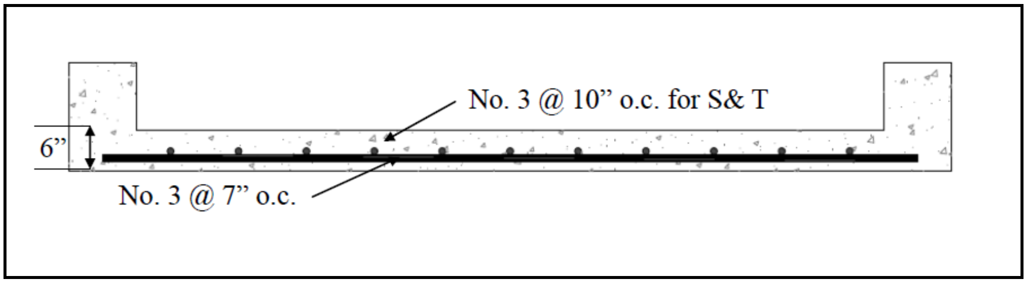

Figure 54. Typical slab cross section.

4.4.4. Concrete Edge Beams

Concrete edge beams will effectively carry the loading transferred from the concrete deck to the cables hanging from the arch members. This beam will be designed to support 1.6 k/ft (from factored live and dead loads) which transfers the loading from the decks to the beams, a 0.09 k/ft for railing, and the self weight of the concrete beam.

Calculations from the Appendix show that the beam should be constructed of a 10" wide and 16" deep rectangular concrete beam. The beam shall be reinforced with 3 No. 4 steel reinforcement on the bottom and 2 No. 4 bars on the top to allow for anchorage for the cables tying into the beams. As far as the shear reinforcement that is required, No. 3 stirrups will be used. Figure

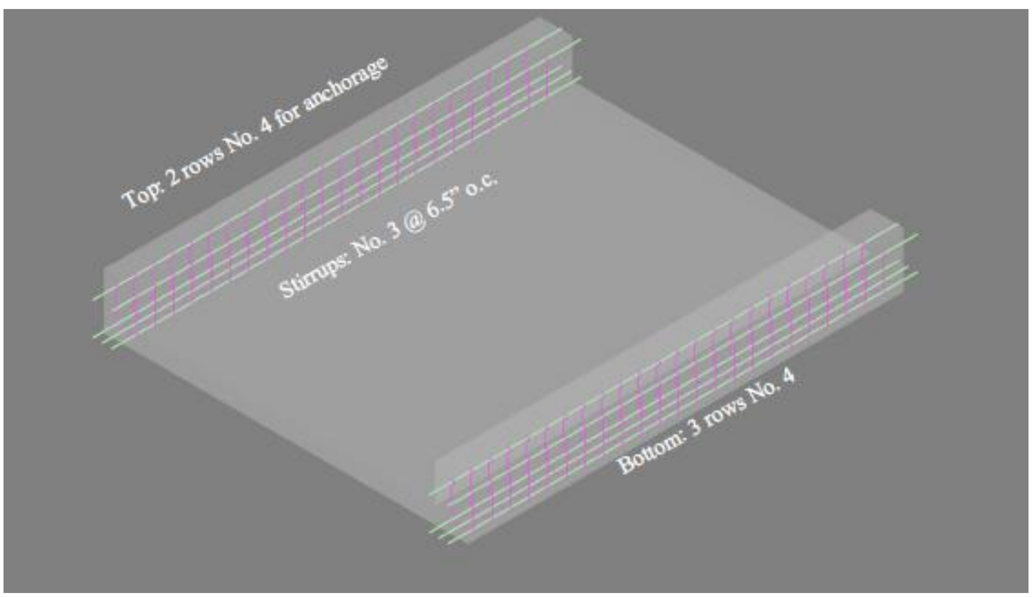

Figure 55. Edge beam design.

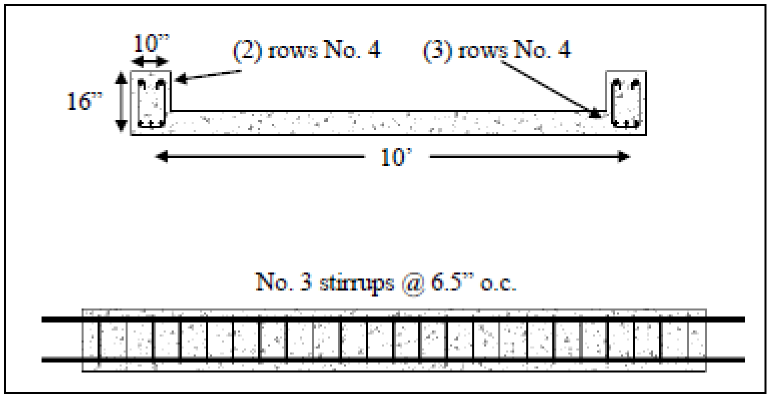

Figure 56. Typical cross sections for edge beam.

4.4.5. Footing Design

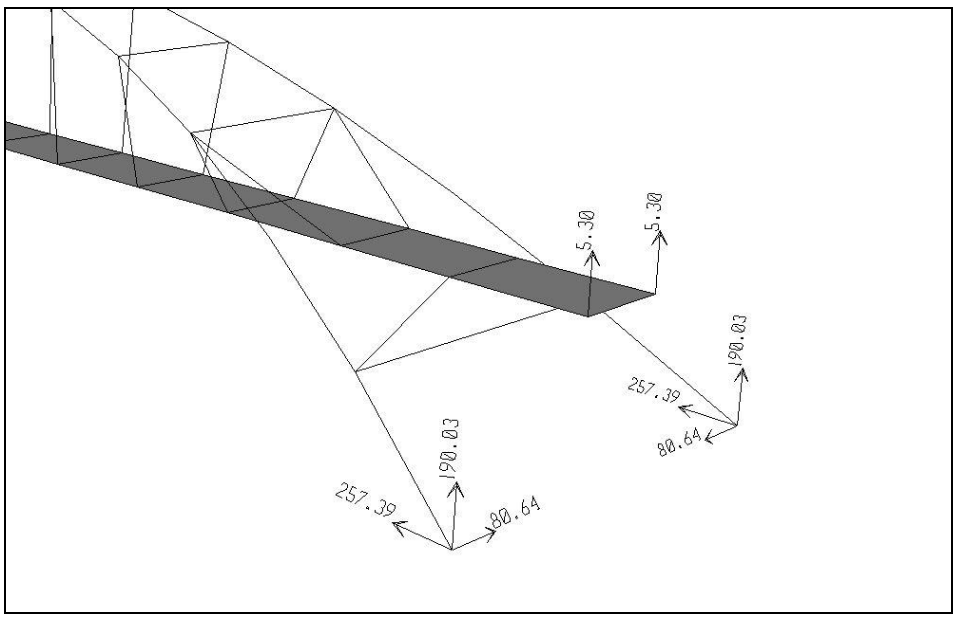

Figure 57. Base reactions for DSTL2 load case.

The supports for each of the four arches are to be designed from concrete using a design compressive strength of 4000 psi and yield strength Figure 57. of the reinforcing steel of 60,000 psi, and the base reaction forces found in . Using the support reactions shown in Figure 57, the steps taken for the design of the concrete footers is shown in the Appendix. When designing only for the reaction coming directly into the footer (since footer will be placed at same angle as arch tying into it) the design for each of the footers would be 8' x 11' x 2'. This design is adequate for the arch bridge since the bridge has a large horizontal force at the foundation. Thus, the design in the Appendix takes into account the thrust force in the horizontal direction, and assumes that the footer itself will be able to support this force. The subsequent design for the footer is calculated to be a 10’ x 12’ x 11’ with 6’ of the footer being below grade.

4.4.6. Vibrations

As described in section 6 of the LRFD Guide Specifications for the Design of Pedestrian Bridges, “Vibration of the structure shall not cause discomfort or concern to users of a pedestrian bridge”. The code later prescribes a limit to the fundamental frequency of the first vertical mode to be greater than 3.0 Hertz (Hz), in the absence of any applied live loads. If the fundamental frequency does not satisfy this limit than a more in depth look at the dynamic performance of the bridge must be undertaken.

To alleviate any problems that these oscillations may cause, the response of the bridge can be held in check simply by adding more mass to the bridge, if it is needed. Since the frequency of the bridge is controlled by Newton’s equation:

$$\sum F = ma$$

By rearranging it can easily be shown that,

$$a = \frac{\sum F}{m}$$

Thus, increasing the mass will decrease the acceleration of the structure.

The LRFD Guide has a simple formula to determine the fundamental frequency of a pedestrian bridge. The formula is:

$$f \geq 2.86 \ln\left(\frac{180}{W}\right)$$

Where;

f = fundamental frequency (Hz)

W = total weight of the supported structure (kips)

As stated above, if this frequency is greater than 3.0 Hz, then no further investigation is required.

With the only weight that is calculated in the frequency equation being that of the supported structure, the group had to determine the approximate weight of the concrete deck. As long as the deck's weight is large enough, the frequency of the bridge can be estimated to be large enough that the structure will not vibrate under its first mode. Since the final deck was designed to be regular concrete that is 6" thick, the weight of the deck was calculated by:

$$\mathrm{W} = \frac{131.25\,ft^2}{1}\,x\,\frac{0.150k * 0.5}{ft^2}\,x210\,ft = 2067\text{ k}$$

Using this weight in the frequency equation gives:

$$f \geq 2.86\ln\left(\frac{180}{2067}\right)$$

$$f = -6.98\text{ Hz}$$

When dealing with a frequency, the sign convention is similar to everything else where the sign of the value describes the direction of the vibration. Thus, a natural frequency of 6.98 Hz exceeds the minimum value of 3.0 Hz, so no further vibration analysis is required for the structure.

4.4.7. Deflection

When designing a structure, one must first analyze the structure based on strength conditions. If the structure is capable of carrying the loads safely, the next step is to verify that the structure meets serviceability requirements. In SAP2000, deflection limits are taken into consideration when the software performs the steel design of the given structure. When printing the report directly from SAP2000, joint deflections are given in table format according to each of the individual load combinations.

The appendix shows a portion of one such table that shows the maximum joint deflection. As is shown in this table, the maximum deflection was found to be just under ¼” (0.240 in). Using the maximum allowable deflection per Ref. 8 for a pedestrian bridge being L/500, it is easily determined that this pedestrian bridge meets the deflection requirements. (This was calculated from the maximum member length of the concrete being 13.125’ leading to a maximum allowable deflection of 0.315 in.) The small values found for the deflection of the bridge confirm the earlier statement that the deformed shapes that the software produces are exaggerated to help the user better visualize what has happening with the structure.

4.5. Final Design

4.5.1. SAP2000 Report

Another feature of the SAP2000 is software is its ability to prepare advanced technical reports for the structure that the engineer is designing. In the appendix, there are a few samples of the types of tables that the software prepares. It should be noted that the complete SAP2000 report was not included with this project since the final report was in excess of 500 pages.

In the report, the user can find information pertaining to the coordinates of each joint, property of the materials that are used for the design of the structure, the actual displacements of each joint, etc. The report serves as another way that the engineer can verify the analysis of the structure as well as allowing them to have all of the design information in convenient table form. This allows for easy reference of the mechanical behavior of the designed structure.

4.5.2. Final Design Drawings

For the final design of the structure, the group imported the final model of the bridge from SAP2000 into AutoCAD where the structure was rendered as is shown in Figure 58. This design shows the structure designed with the properly sized members calculated previously.

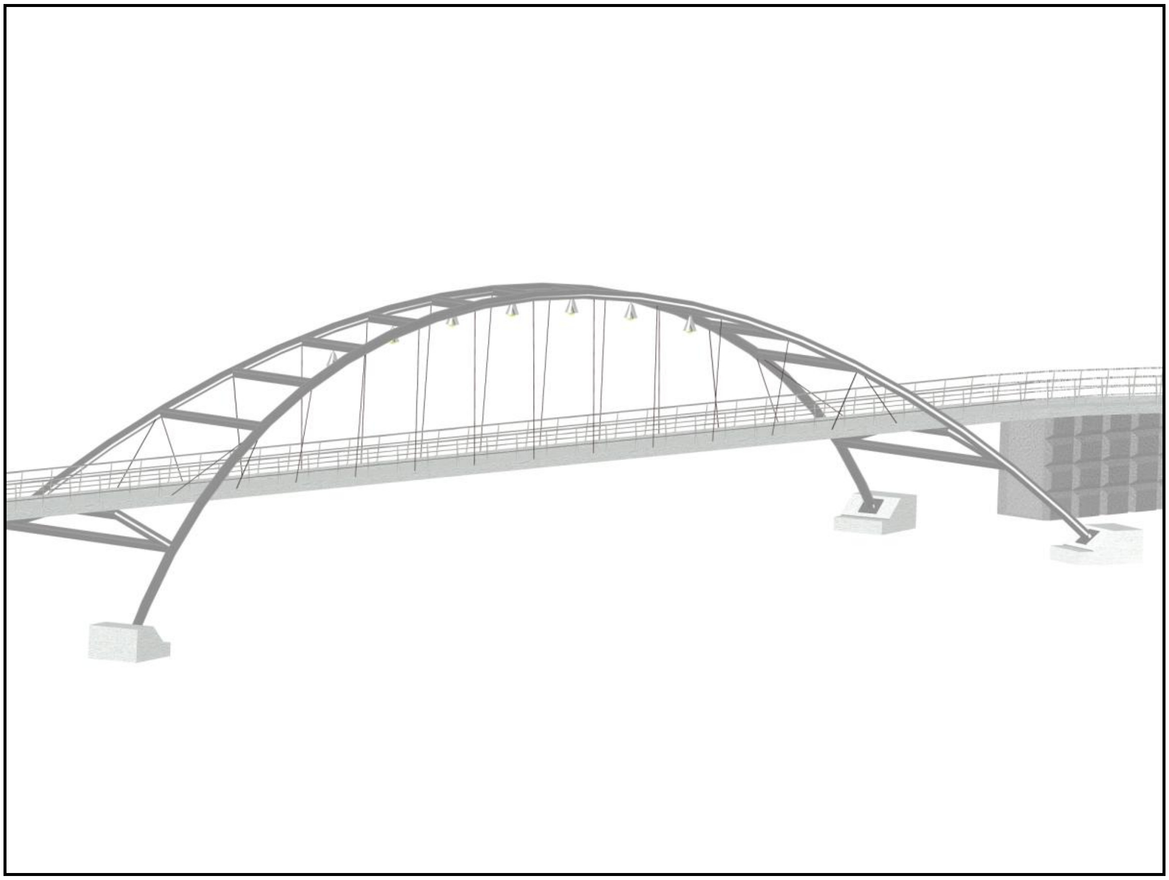

Figure 58. 3-d rendering of the final design.

In addition to doing the renderings in AutoCAD, the group also dimensioned the bridge in this software. Figures 59-61 display the plan views for the front, side, and top of the bridge, respectively.

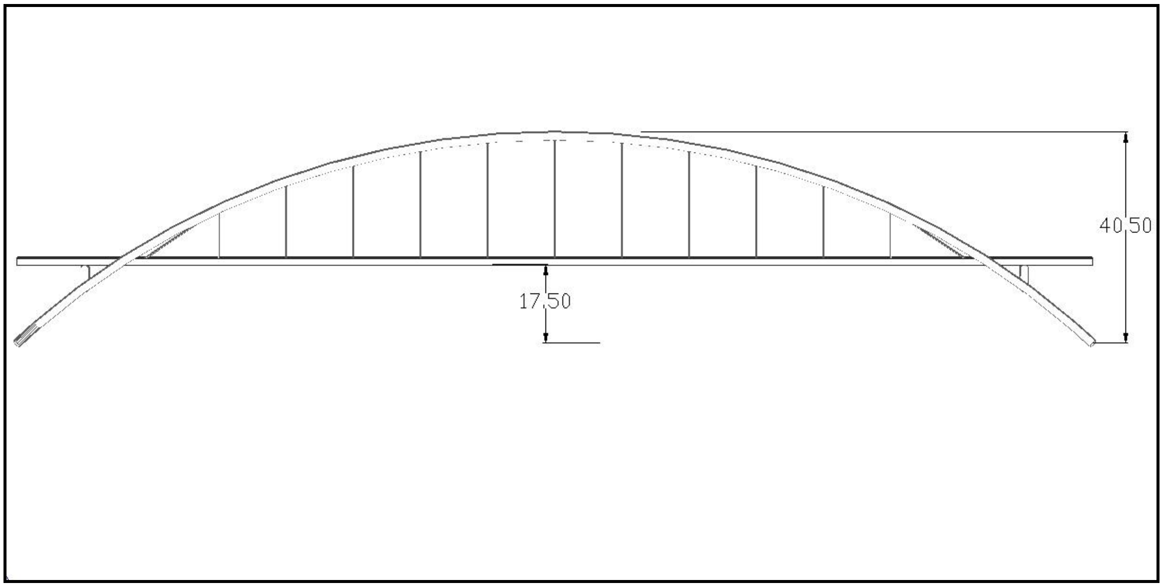

Figure 59. Front dimensional view of the pedestrian bridge.

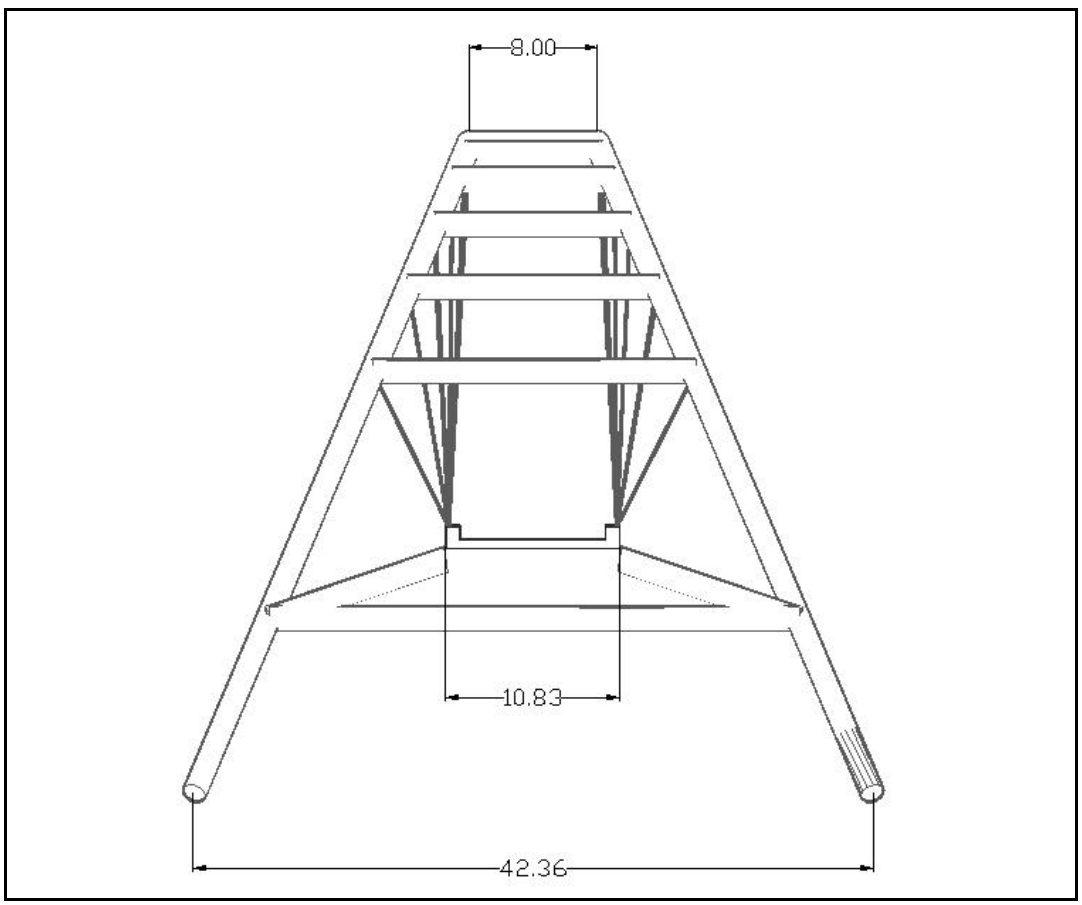

Figure 60. Side dimensional view of the bridge.

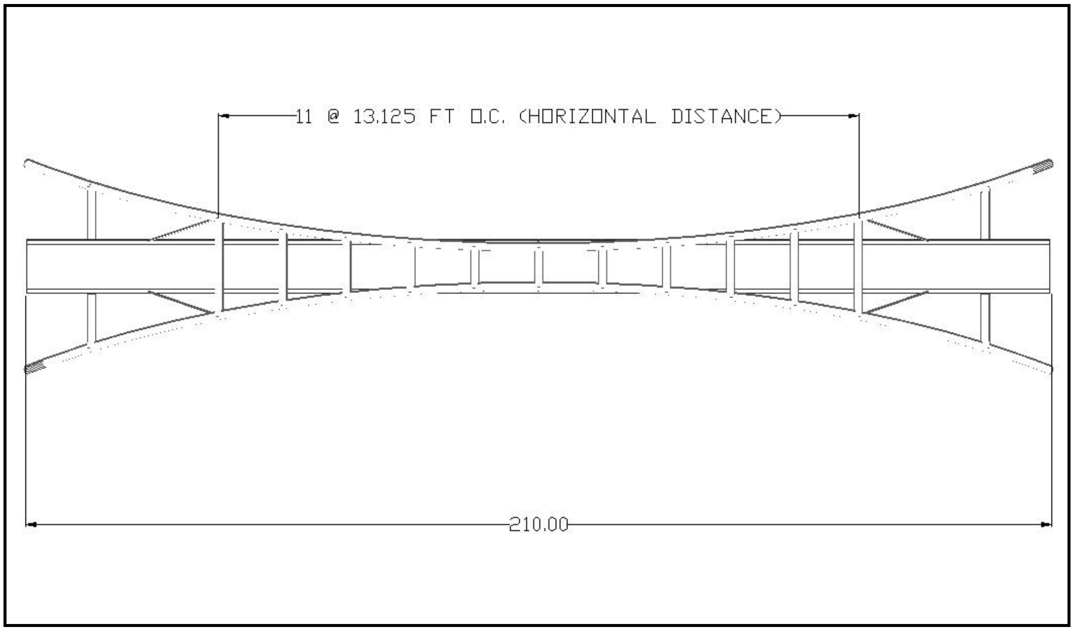

Figure 61. Top dimensional view of the bridge.

Figure 62 shows how the cables will tie into the concrete edge beams. For each connection, the cables will be wrapped around a steel eyelet that is embedded into the concrete edge beam. The excess steel cable will be cut and crimped as shown.

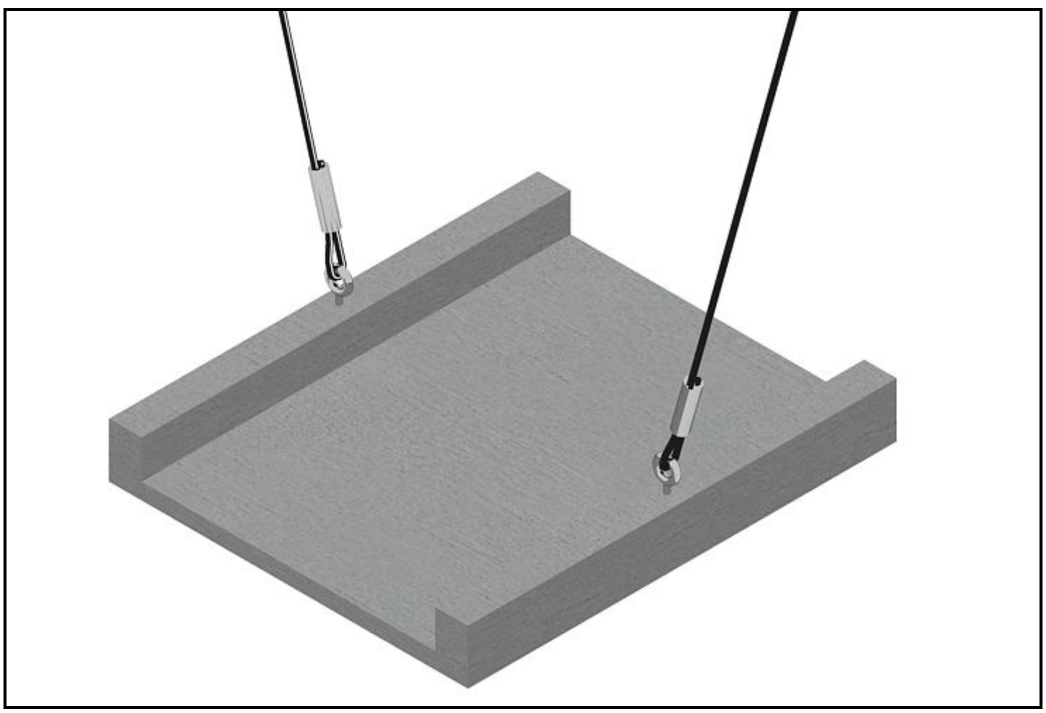

Figure 62. Rendering of cable connecting to edge beam.

4.6. Alternate Design Considerations

The final design could be altered in order to allow for alterations that may be requested by the structure's owner. Three possible design alternates could include: enclosing the walkway, making the structure "smart", and/or applying a wrap to represent the mascot of IPFW.

4.6.1. Enclosing the Walkway

The final design could easily be altered to allow for the bridge deck to be completely enclosed instead of open as the final design portrays. If requested by the owner, the design could entail a covering over the sidewalk, much like the design of the Willis Family Bridge. Minor changes in the dead load as well as how the wind affects the structure would be the only concerns in the structural analysis of the covered bridge. By covering the bridge, there would be a tremendous increase in surface area that the wind force would affect, so the design would need to be altered to accommodate the increases in horizontal forces that would come from the larger wind force.

4.6.2. Smart Bridge

Another minor design alteration could lead to the bridge becoming a "smart" structure. Implementing a structural health monitoring system (SHM) to the bridge would allow for the owners to actively monitor the state in which the bridge finds itself. By relaying information about the bridge's mechanical behavior through a system of sensors attached to the bridge, the university would be able to maintain the bridge when maintenance is needed. SHM technology has recently gained the support of the engineering community in helping manage the issue with the nation's crumbling infrastructure. The system would allow for the owners to know what condition the bridge is in at any given time, and not have to rely on visual inspections to determine the health of the bridge.

There would be few changes that would need to be made in order to apply SHM sensors to the bridge structure. Only if conduits were run in the concrete deck would any considerations need to be taken on the structural end of the design. With this bridge being designed by members of the university, all forces as well as any potential problem areas on the bridge are easily known; allowing for sensors to be placed in the exact locations that are needed the most. By adding an SHM system to this new bridge, not only would the IPFW campus be at the forefront of this innovative technology, but it could also provide a valuable learning tool to the campus's engineering students.

4.6.3. Mastodon Tusks

Another option that the group considered would be completely for IPFW, and do little to help Ivy Tech. What the group thought was to somehow apply a wrap to the arch members that would take the form of mastodon tusks coming up from opposite sides of the foundation. By doing this, the bridge could become a cornerstone of the IPFW campus, and provide more exposure to the various IPFW sports programs. Figure 63 shows how the group envisioned wrapping the arch members to form two-pairs of mastodon tusks.

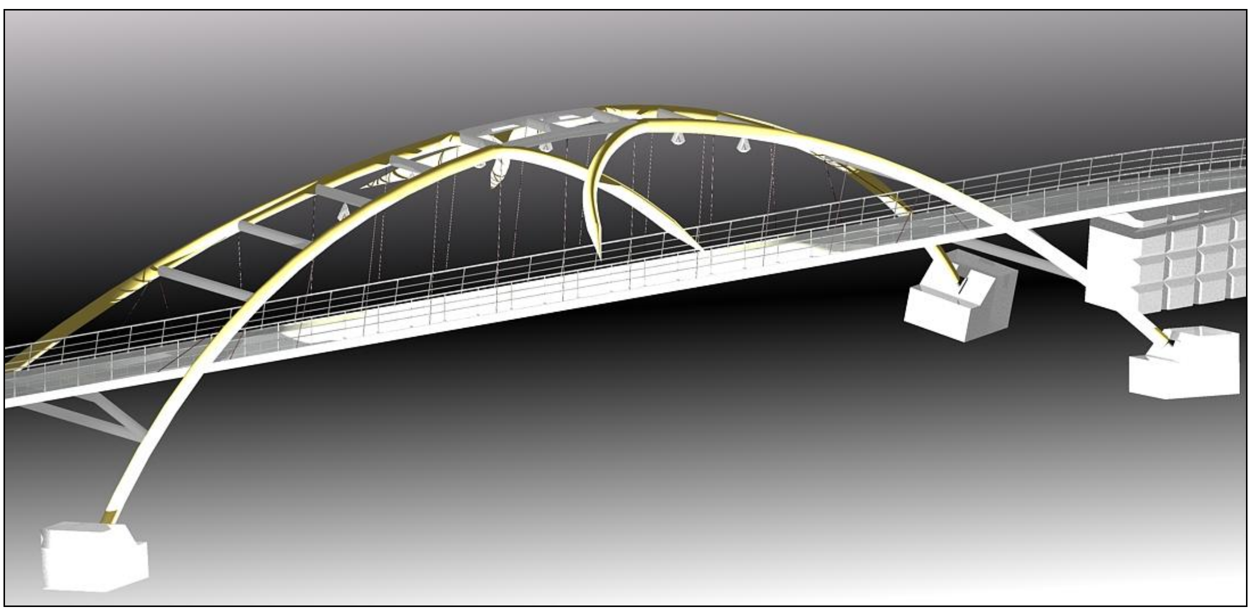

Figure 63. Arch members enclosed to form Mastodon Tusks.

4.6.4. Construction Materials

Instead of building the walkway of the bridge completely out of concrete, the design could be modified so that the deck is constructed out of a combination of both concrete and steel. In order to decrease the weight of the bridge, as well as the concrete that would be needed for the bridge, the deck could be constructed out of a thinner concrete deck underlain by corrugated steel. The steel would then serve as structural support for the concrete with the major difference between this design and the proposed design being that the corrugated steel would also need to be supported by a steel beams. Although more pieces would be required for this design, this approach would lessen the weight of the walkway. Since the natural frequency of the bridge (7.0 Hz) is much greater than the minimum frequency required before a dynamic analysis needs to be performed on the bridge, the weight can be decreased before any unwanted oscillations may appear.

5. Section V: Cost Analysis/Estimation

5.1. Construction Techniques

As described in detail in 45.13 of the *Bridge Engineering Handbook*, there are some difficulties contractors are faced with when constructing a steel arch bridge. When it comes to constructing a steel arch structure, matching up the curved arch pieces in order to make the correct continuous radius is difficult to say the least. It has been found that workers on the construction site have had troubles making field-measured geometric and stress conditions agree with those that are calculated theoretically by the bridge designers.

There are two general practices used in steel arch bridge design: the field adjustment procedure and the shop control procedure. In the field adjustment procedure, it is required for the workers on the site to carry out a program of steelwork surveys and measurements as the erection of the steel arches progressives. It is then the steelworkers' requirement to make any field adjustments needed to maintain the arch dimensions within the previously defined overall tolerances of the arch.

The second procedure, the shop control procedure, puts all of the trust in the initial site survey and uses these measurements as the basis for the dimensions used in the construction of all the parts of the bridge. With this approach, the field workers are assumed to not have to make any field adjustments during the construction of the bridge. For the proposed pedestrian bridge over Coliseum Boulevard, the group has determined to use the shop control procedure due to the relatively short span used in the design of the bridge.

In addition to the design procedures, there are also two general methods of arch bridge construction: the tie back and the false frame work methods. In the tie back method, piers on either side of the span of the bridge are used to support the main ribs used in the arch structure. The cables are directly connected to the arch pieces as well as the pier to support any loads carried by the members.

For the false frame work method, a set of supports are constructed underneath the bridge to carry the arches as they protrude from either side of the main bridge span. Since this pedestrian bridge is crossing over a major arterial road, the group has decided the best construction method to be used would be the tie back method which allows for a minimal impact on travelers on Coliseum Boulevard.

5.2. Cost Estimation

For an accurate cost breakdown of the bridge, one can not only look at the price for the materials, but must add in all factors involved when building a new structure such as this. The figures shown in Table 4 were taken from the TE Application that the IPFW physical plant submitted in August 2008. At the time, the project was not approved; however, much of the pricing information should still be valid slightly over a year later.

Table 4. General cost breakdown for pedestrian bridge.

Activity	**Estimated Cost**
Project Development and Environmental Studies	$30,000
Engineering and Final Plans Preparation Work	$330,000
Construction	$3,600,000
Construction Engineering and Inspection Activities	$540,000

Table 5 shows a more detailed cost breakdown for the construction of the pedestrian bridge. As stated above, there are more factors involved in building the bridge other than the cost of materials and labor needed for the structure. These figures were also compiled from the TE Application filed in 2008.

Table 5. Detailed cost breakdown for construction of pedestrian bridge.

Activity	Estimated Cost
Archeological Study	$30,000
Engineering and Final Plan Prep	$330,000
Mobilization and Demobilization	$50,000
Site Clearing and Traffic Control	$100,000
Sitework and Excavation	$300,000
Structural Piling	$200,000
Reinforcing Steel	$300,000
Concrete Work	$600,000
Structural Steel	$1,500,000
Utilities	$200,000
Electrical Work and Lighting	$250,000
Restoration	$100,000
Construction Engineering/Inspection	$540,000
Estimated Total Cost	**$4,500,000**

6. Conclusion

With pedestrian travel over Coliseum Boulevard being as dangerous as it is, the group feels that the best possible way in ensuring safe travel over this roadway is by constructing a new pedestrian bridge. In addition to helping pedestrians safely cross over Coliseum Boulevard, the structure should be of an innovative design of the same caliber as the other two pedestrian bridges located on the IPFW campus.

Based upon the extensive research put forth by this senior design group, the most suitable type of bridge to meet the needs of this structure is of an arch style design. With a overall span of 210’ and a height off of the footer of 40.5’, the structure is not only safely able to carry all of the forces that it would be exposed to, but it will also be of the same level of design as is to be expected by administrators at IPFW.

Utilizing steel and concrete for the major design members, erection of the structure would proceed quickly due to the ability of most of the main components being prefabricated off of the job site. Utilizing this design method would greatly minimize the effects that the construction of the bridge would have to travelers who use Coliseum Boulevard on a daily basis.

7. References

[1] American Association of State Highway and Transportation Officials. AASHTO LRFD Movable Highway Bridge Design Specifications: 2008 Interim Revisions. Washington, DC: American Association of State Highway and Transportation Officials, 2008.

[2] American Institute of Steel Construction. Steel Construction Manual. 13^{th} Edition. Chicago: AISC, 2007.

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[6] "Crossroads". Ivy Tech Community College – Fort Wayne. http://www.ivytech.edu/fortwayne/crossroads/ 28May 2009

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[10] Parkman, Kathy. "River Greenway". City of Fort Wayne Parks & Recreation Website. http://www.fortwayneparks.org/index.php?option=com_content&view=article&id=151&Itemid=34. 28 may 2009

[11] Pedestrian Bridges. Florida Department of Transportation (FDOT). http://www.dot.state.fl.us/structures/StructuresManual/CurrentRelease/DesignGuidelines/SDG10PedestrianBridges.htm. 7 Aug 2009.

[12] "Provision of facilities for pedestrians and bicycles". 2008 Indiana Department of Transportation, Transportation Enhancement (TE) Application.

8. Appendix

8.1. Hand Calculations

8.1.1. Angle Member Hand Calculations

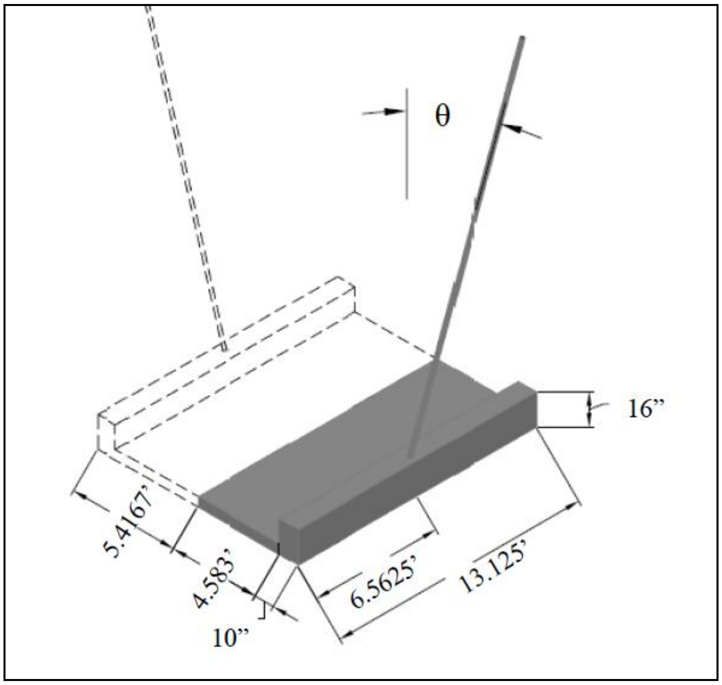


Figure 64. Free body diagram of typ. slab section.

Using 100 psf live load; slab thickness of 9” and slab length of 13.125 ft
Live Load
With 500 plf

LL = 6.56 k
[SAP2000 = 6.21 k; difference 5.6%]

Dead Load
49.21875 ft^3

DL = 7.38k
[SAP2000 = 7.57 k; difference 2.5%]

8.1.2. Arch Hand Calculations

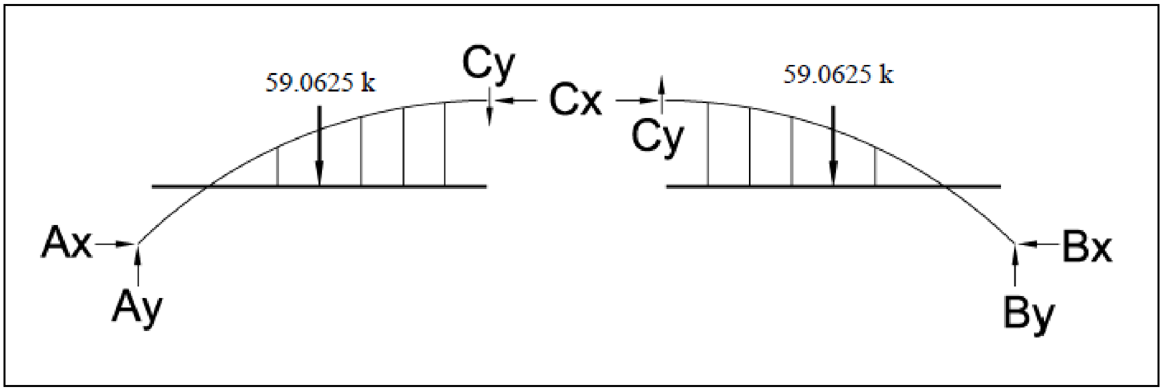


Figure 65. Free body diagram of parbolic arch.

Calculated with 9" concrete slab for the deck

$$CCW(+)\sum M_A = 0;$$
$$0 = 44C_x - 105C_y - (51.6797)(59.0625)$$
$$0 = 44C_x - 105C_y - 3052.33$$

$$CW(+)\sum M_B = 0;$$
$$0 = 44C_x + 105C_y - (51.6797)(59.0625)$$
$$0 = 44C_x + 105C_y - 3052.33$$

M_A into M_B

$$0 = 88C_x - 6104.66$$
$$88C_x = 6104.66$$

$A_x = B_x = C_x = 69.37$ k

[SAP2000 = 75.67 k; difference of 8.3%]

CHECK C_y

$$0 = (44)(-69.4) + 105C_y - 3052.33$$

$C_y = 58.15$ k

$A_y = B_y = 51.68$ k

[SAP2000 = 56.73 k; difference of 8.9%]

8.1.3. Concrete Slab Design

<u>Assumptions:</u>

f_y = 60 ksi

f'_c = 4 ksi

Minimum cover, d = 6" – 1.0 = 5.0"

Unit Weight of Concrete = 150 lb/ft^3

Minimum slab thickness (from Table 13.1 Ref. "concrete design")

Simply supported slab $h_{min} = l/20$

$$h_{\min} = \frac{l}{20} = \frac{10x12}{20} = 6.0''$$

Use $\mathbf{h_{min}}$ = 6"

Since one-way slab, load per 1' width

<u>Design Load Calculation:</u>

<u>Dead Load:</u>

Self-Weight of Slab = $\frac{6}{12} x 150 lb/ft^3 = 75$ lb/ft^2

Superimposed Dead Load = 20 lb/ft^2

Total Dead Load = 95 lb/ft^2 x 1 ft = 95 lb/ft

<u>Live Load:</u>

Total Live Load = 90 lb/ft^2 x 1 ft = 90 lb/ft

<u>Design Load Combination:</u>

1.2D+1.6L

Wu = 1.2 (95 lb/ft) + 1.6 (90 lb/ft) = 258 lb/ft

Each slab, simply supported:

$$M_{\max} = \frac{wl^2}{8} = \frac{258 * 10^2}{8} = 3.225k - ft$$

$$V_{\max} = \frac{wl}{2} = \frac{0.258 * 10}{2} = 1.29k$$

From Table A.9 Ref. "Concrete Design"
$\rho = 0.003$ and ϕ Mn = 3.9 k - ft
$A_s = \rho bd$
$A_s = 0.003x12x5 = 0.18$ in^2/ft
$A_{s,min} = 0.0018bh$
$A_{s,min} = 0.0018x12x6 = 0.1296$ in^2/ft
$A_s > A_{s,min}$

From Table A.3 Ref. "Concrete Design"

Bar No. 3 at 7.5" spacing, A_s= 0.18 in^2/ft
Bar No. 3 at 7.0" spacing, A_s= 0.19 in^2/ft

Choose Bar No. 3 at 7.0" spacing for ease of construction

Spacing Requirement:
$3" \leq s \leq \min\{3h,12\}$
$3" \leq 7" \leq 12"$

Use Bars No. 3 at 7" spacing

Shrinkage and Temperature Reinforcement:

$A_{s,min} = 0.1296$ in^2/ft
From Table A.3 Ref. "Concrete Design"
Bar No. 3 at 10" spacing, A_s= 0.13 in^2/ft
$A_s > A_{s,min}$

Spacing Requirement:
$3" \leq s \leq \min\{5h,18\}$
$3" \leq 10" \leq 18"$

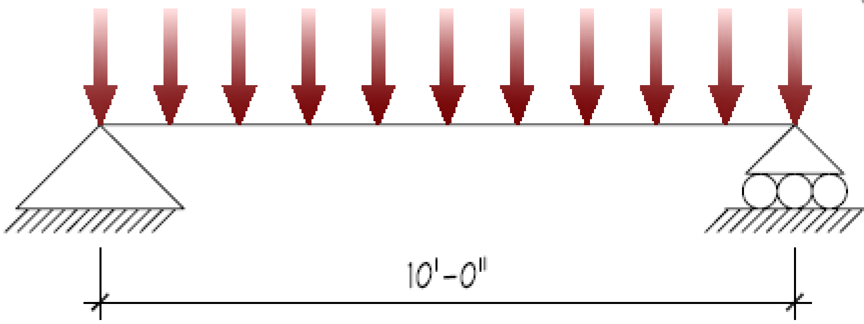
Use Bars No. 3 at 10" spacing for shrinkage and temperature

Shear Design:

$V_{max} = 1.29$k
$$V_c = 2\sqrt{f_c'}b_w d = 2\sqrt{4000}x12x5.0 = 7590 \text{ lb}$$
$V_c = 7.59$ k
No stirrups required if:
$V_{max} < \phi V_c$
$\phi V_c = 0.75 * 7.59 = 5.69$ k

Therefore no stirrups are required

8.1.4. Edge beams to support the concrete deck



$$\text{Reaction due to Dead Load} = \frac{wl}{2} = \frac{95*10}{2} = 475k$$

$$\text{Reaction due to Live Load} = \frac{wl}{2} = \frac{90*10}{2} = 450k$$

<u>Assumptions:</u>

f_y = 60 ksi

f'_c = 4 ksi

Dead Load due to Railing = 90 lb/ft

Unit Weight of Concrete = 150 lb/ft^3

Try 10”x16” (dimension of beam)

<u>Design Load Calculation:</u>

<u>Dead Load:</u>

$$\text{Self-Weight of Beam} = \frac{10}{12} x \frac{16}{12} x 150 lb/ft^3 = 167 \text{ lb/ft}$$

Railing = 90 lb/ft

Total Dead Load = 167 + 475 + 90 = 732 lb/ft

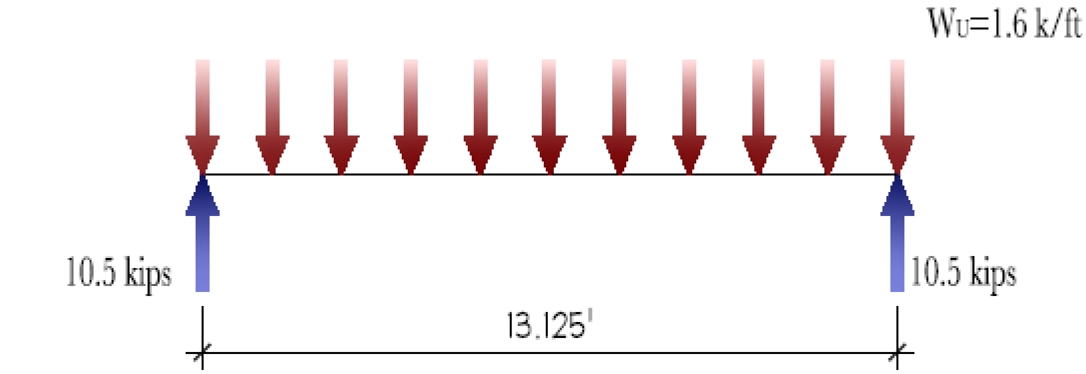
<u>Live Load:</u>

Total Live Load = 450 lb/ft

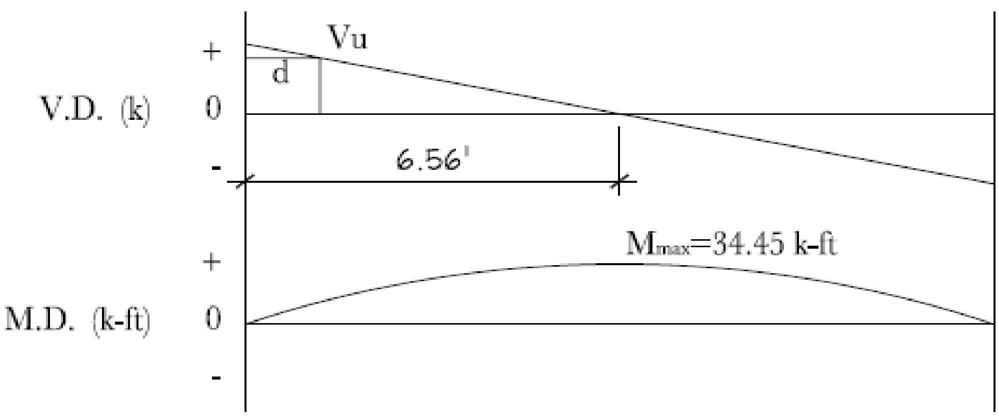
Design Load Combination:

1.2D+1.6L

Wu = 1.2 (732 lb/ft) + 1.6 (450 lb/ft) = 1.6 k/ft



$$\text{Reactions} = \frac{wl}{2} = \frac{1.6*13.125}{2} = 10.5k$$



$$M_{\max} = \frac{wl^2}{8} = \frac{1.6*13.125^2}{8} = 34.45k - ft$$

$$Mu = \frac{bd^2R}{12000} = R = \frac{12000Mu}{bd^2} = \frac{12000*34.45}{10*13.5^2} = 227\,psi$$

From Table A.5a Ref. “Concrete Design”

$\rho = 0.0039 > \rho_{min} = 0.0033$

$A_s = 0.0039 \times 10 \times 13.5 = 0.5262 \text{ in}^2$

From Table A.3 Ref. “Concrete Design”

Use 3 bars No. 4 (A_s = 0.60 in^2)

Use 2 bars No. 4 on top of beam for anchorage

Shear Reinforcement:

$$\phi \mathrm{V_c} = 0.75x2\sqrt{f_c^{'}}b_w d = 0.75x2\sqrt{4000}x10x13.5 = 12.8 \text{ k}$$

From similar triangles method, V_{max} = 8.7 k

$$\frac{\phi \mathrm{Vc}}{2} = \frac{12.8}{2} = 6.4k$$

Since $V_{max} > \frac{\phi \mathrm{Vc}}{2}$ minimum amount of stirrups is needed

Recommended minimum beam width to accommodate different stirrup sizes:

Stirrup Size	Minimum beam width
# 3	10”
# 4	12”
# 5	14”

Use No. 3 stirrups (A_v = 0.22in^2)

Minimum spacing is needed:

$$s_1 = \min\{\frac{AvFy}{0.75\sqrt{f'c}b} \; or \; \frac{AvFy}{50b}\}$$

$$s_1 = \min\{\frac{0.22*60000}{0.75\sqrt{4000}*10} = 27.83" \; or \; \frac{0.22*60000}{50*10} = 26.4"\}$$

s_1=26.4in

According to ACI (section 11.5.5.1) the maximum allowable spacing when

$$\frac{\phi \mathrm{Vc}}{2} < V_{max} < \phi \mathrm{Vc} \; :$$

s_{max}= min {s_1, d/2, 24in}

s_{max} = min {26.4”, 6.75”, 24”} = 6.75”

This ensures each 45° crack is intercepted by at least one stirrup

Use 6.5” spacing for ease of construction

8.1.5. Footing Design

Using the reaction forces from DSTL2 load case (Figure 53):

$F_x = 257.39$ k

$F_y = 80.64$ k

$F_z = 190.03$ k

$$R_{xy} = \sqrt{257.39^2 + 80.64^2} = 269.73k$$

$$R = \sqrt{269.73^2 + 190.03^2} = 330k$$

Assuming $f'_c = 4$ ksi & Allowable soil bearing capacity, $q_a = 4.5$ k/ft^2

Effective bearing capacity:
Assuming a maximum of 4' of concrete,
$q_e = 4500 - (150 \times 4) = 3900$ k/ft^2

$$A_{req} = \frac{330k}{3.9k/ft^2} = 84.62 \text{ ft}^2$$

Use a 8' x 11' rectangle, A = 88 ft^2

$$q_u = \frac{330k}{8\,ft} = 3.78 \text{ k/ft}^2$$

Design for punching shear:
Perimeter:
$b_o = 4(24 + 20) = 176$ in

$V_{u1} = 3.78 \text{ k/ft}^2(88 - (44/12)^2)$
$V_{u1} = 281.82$ k

Available shear strength:

$$V_c = 4\sqrt{f'_c}\,b_o d$$

Assuming d = 20"

$$V_c = 4\sqrt{4000}(176)\left(\frac{20}{1000}\right) = 890.5 \text{ k}$$

ø = 0.75
ø $V_c = (0.75)(890.5) = 667.87$ k

V_{a2} = (3.78)(3.67)(8') = 110.99 k = 111 k

$$V_c = 2\sqrt{4000}(8)(12)\left(\frac{20}{1000}\right) = 242.86 \text{ k}$$

ø V_c = (0.75)(242.86) = 182 k

Reinforcing steel design (f_y = 60 ksi):
Across critical sections of the footer:

$$M_u = 3.78k/ft^2 * 8ft * \left(\frac{4.5ft^2}{2}\right)12in/ft = 3674 \text{ k-in}$$

$$A_s = \frac{3674k-in}{0.9(60(20-1))} = 3.58 \text{ in}^2$$

$$A_{s,Min} = \frac{3\sqrt{4000}}{60000}x96inx20in = 6.07 \text{ in}^2$$

But no less than,

$$A_{s,Min} = \frac{200}{60000}x96inx20in = 6.4 \text{ in}^2$$

Use A_s = 6.4 in^2

Using #7 rebar (A_b = 0.60 in^2):

(11) #7 rebar @ 8.5 in spacing for the 11 ft length

For the 8 ft length:

$$M_u = 3.78k/ft^2 * 11ft * \left(\frac{3ft^2}{2}\right)12in/ft = 2245 \text{ k-in}$$

$$A_s = \frac{2245k-in}{0.9(60(20-1))} = 2.188 \text{ in}^2$$

$$A_{s,Min} = \frac{3\sqrt{4000}}{60000}x132inx20in = 8.35 \text{ in}^2$$

But no less than,

$$A_{s,Min} = \frac{200}{60000}x132inx20in = 8.8 \text{ in}^2$$

Use A_s = 8.8 in^2

Using #7 rebar (A_b = 0.60 in^2):
(15) #7 rebar @ 8.5 in spacing for the 8 ft length

Height of footer:
ACI recommends a minimum of 3” cover when concrete is in contact with the ground,

Diameter of #7 rebar = 0.875 in

3” + 0.4375” = 3.4375 in
With d = 20 in

Use h = 24 in.

The above detailed design is for the soil to be able to support the footings in the vertical direction; however, with such a large thrust force (269.73 k), additional design considerations must be made in order to resist this force. Either the soil can support this, or concrete can. The group decided to go with concrete supporting it and calculated this by:

$$\Sigma M = 269.73h - 190.03 * 3 = 0$$

h = 2.11 ft

Thus, the height at which the force from the arch members comes into the footing shall be 2.11 ft above the center of gravity of the footing. With this, can calculate the weight of the footer needed:

$$\Sigma F_y = -190.03 + Weight = 0$$

Weight of the support = 190.03 k

190.03 = 0.150 lb/ft^3 * 12’ * 10’ * h

H = 10.56’

Use a height of 11’

Note: the dimensions of the footer (12’ x 10’) were modified in order to shorten the above height.

The final footer shall be designed as:

10’ x 12’ x 11’

With 6’ of the footer being below grade.

8.1.6. Tension Cable Design

Assumptions:

P_u= 26.719k

Steel A36 (F_u= 58 ksi)

$$A_D = \frac{P_u}{\phi 0.75 F_u} = \frac{26.719}{0.75*0.75*58} = 0.82\,\text{in}^2$$

$$A = \frac{\pi}{4} d^2$$

$$d = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4*0.82}{\pi}} = 1.02\,\text{in}$$

8.2. Sample SAP2000 Data

1.1. Joint coordinates

Table 1: Joint Coordinates

Table 1: Joint Coordinates

Joint	CoordSys	CoordType	GlobalX	GlobalY	GlobalZ
			in	in	in
1	GLOBAL	Cartesian	0.000	0.000	15.680
2	GLOBAL	Cartesian	0.000	10.000	15.680
3	GLOBAL	Cartesian	210.000	0.000	15.680
4	GLOBAL	Cartesian	210.000	10.000	15.680
5	GLOBAL	Cartesian	13.125	10.000	15.680
6	GLOBAL	Cartesian	13.125	0.000	15.680
8	GLOBAL	Cartesian	196.875	10.000	15.680
9	GLOBAL	Cartesian	196.875	0.000	15.680
18	GLOBAL	Cartesian	0.000	-16.180	0.000
34	GLOBAL	Cartesian	210.000	-16.180	0.000
35	GLOBAL	Cartesian	0.000	26.180	0.000
74	GLOBAL	Cartesian	39.375	0.000	15.680
75	GLOBAL	Cartesian	39.375	10.000	15.680
76	GLOBAL	Cartesian	52.500	0.000	15.680
77	GLOBAL	Cartesian	52.500	10.000	15.680
78	GLOBAL	Cartesian	65.625	0.000	15.680
79	GLOBAL	Cartesian	65.625	10.000	15.680

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Figure 66. Screen shot displaying joint coordinate table from SAP2000 report.

2. Material properties

This section provides material property information for materials used in the model.

Table 8: Material Properties 02 - Basic Mechanical Properties

Table 8: Material Properties 02 - Basic Mechanical Properties						
Material	UnitWeight	UnitMass	E1	G12	U12	A1
	Kip/in3	Kip-s2/in4	Kip/in2	Kip/in2		1/F
4000Psi	1.5000E-01	4.6621E-03	519120	216300	0.200000	5.5000E-06
A615Gr60	4.9000E-01	1.5230E-02	4.2E+06			6.5000E-06
A992Fy50	4.9000E-01	1.5230E-02	4.2E+06	1.6E+06	0.300000	6.5000E-06
MAT	4.9000E-01	1.5230E-02	4.2E+06	1.6E+06	0.300000	6.5000E-06

Table 9: Material Properties 03a - Steel Data

Table 9: Material Properties 03a - Steel Data			
Material	Fy	Fu	FinalSlope
	Kip/in2	Kip/in2	
A992Fy50	7200.000	9360.000	-0.100000

Table 10: Material Properties 03b - Concrete Data

Table 10: Material Properties 03b - Concrete Data		
Material	Fc	FinalSlope
	Kip/in2	
4000Psi	576.000	-0.100000

Figure 67. SAP2000 report table of material properties.

PARABOLICARCHBENTVERIFIEDWITH6INSLAB85PSFLLnonlinearwithservicevehicle11252009.SDB SAP2000 v12.0.
8. Joint results 07 December 200

Table 22: Joint Displacements

Joint	OutputCase	StepType	U1 in	U2 in	U3 in	R1 Radians	R2 Radians	R3 Radians
91	H5	Max U2	0.000	0.000	0.000	0.000	0.000	0.000
91	H5	Min U2	0.000	0.000	0.000	0.000	0.000	0.000
91	H5	Max U3	0.000	0.000	0.000	0.000	0.000	0.000
91	H5	Min U3	0.000	0.000	0.000	0.000	0.000	0.000
91	H5	Max R1	0.000	0.000	0.000	0.000	0.000	0.000
91	H5	Min R1	0.000	0.000	0.000	0.000	0.000	0.000
91	H5	Max R2	0.000	0.000	0.000	0.000	0.000	0.000
91	H5	Min R2	0.000	0.000	0.000	0.000	0.000	0.000
91	H5	Max R3	0.000	0.000	0.000	0.000	0.000	0.000
91	H5	Min R3	0.000	0.000	0.000	0.000	0.000	0.000
91	H5-2	Max U1	9.090E-04	-4.781E-03	0.094	-7.090E-03	-2.750E-04	-4.300E-05
91	H5-2	Min U1	-1.743E-03	0.015	6.005E-03	6.662E-03	4.200E-04	1.760E-04
91	H5-2	Max U2	-1.743E-03	0.015	6.005E-03	6.662E-03	4.200E-04	1.760E-04
91	H5-2	Min U2	3.700E-05	-0.015	-0.061	-6.720E-03	1.001E-03	-1.790E-04
91	H5-2	Max U3	4.340E-04	3.850E-03	0.175	6.676E-03	1.310E-03	5.400E-05
91	H5-2	Min U3	6.300E-05	-9.876E-03	-0.240	-0.013	8.060E-04	-1.240E-04
91	H5-2	Max R1	-9.390E-04	9.652E-03	-0.109	0.013	-1.300E-04	9.600E-05
91	H5-2	Min R1	1.530E-04	-9.637E-03	-0.239	-0.013	-5.920E-04	-1.240E-04
91	H5-2	Max R2	-1.090E-04	-0.010	-0.194	-0.011	3.206E-03	-1.260E-04
91	H5-2	Min R2	5.600E-04	-8.219E-03	-0.128	-0.011	-3.622E-03	-1.090E-04
91	H5-2	Max R3	-1.743E-03	0.015	6.005E-03	6.662E-03	4.200E-04	1.760E-04
91	H5-2	Min R3	3.700E-05	-0.015	-0.061	-6.720E-03	1.001E-03	-1.790E-04
92	DEAD		-1.440E-04	2.685E-06	-0.048	-6.600E-05	-2.651E-03	-1.200E-05
92	LIVE		-3.000E-05	1.796E-06	-0.022	-3.700E-05	-1.286E-03	-6.515E-06
92	H5	Max U1	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Min U1	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Max U2	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Min U2	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Max U3	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Min U3	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Max R1	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Min R1	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Max R2	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Min R2	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Max R3	0.000	0.000	0.000	0.000	0.000	0.000
92	H5	Min R3	0.000	0.000	0.000	0.000	0.000	0.000
92	H5-2	Max U1	1.026E-03	6.292E-03	0.069	6.333E-03	1.249E-03	7.900E-05
92	H5-2	Min U1	-1.860E-03	-0.018	4.571E-03	-7.413E-03	-2.010E-04	-1.850E-04
92	H5-2	Max U2	-2.000E-05	0.018	-0.070	7.564E-03	4.320E-04	1.820E-04
92	H5-2	Min U2	-1.860E-03	-0.018	4.571E-03	-7.413E-03	-2.010E-04	-1.850E-04
92	H5-2	Max U3	4.810E-04	-5.037E-03	0.148	-6.070E-03	3.202E-03	-4.000E-05

Figure 68. SAP2000 report: joint displacements.

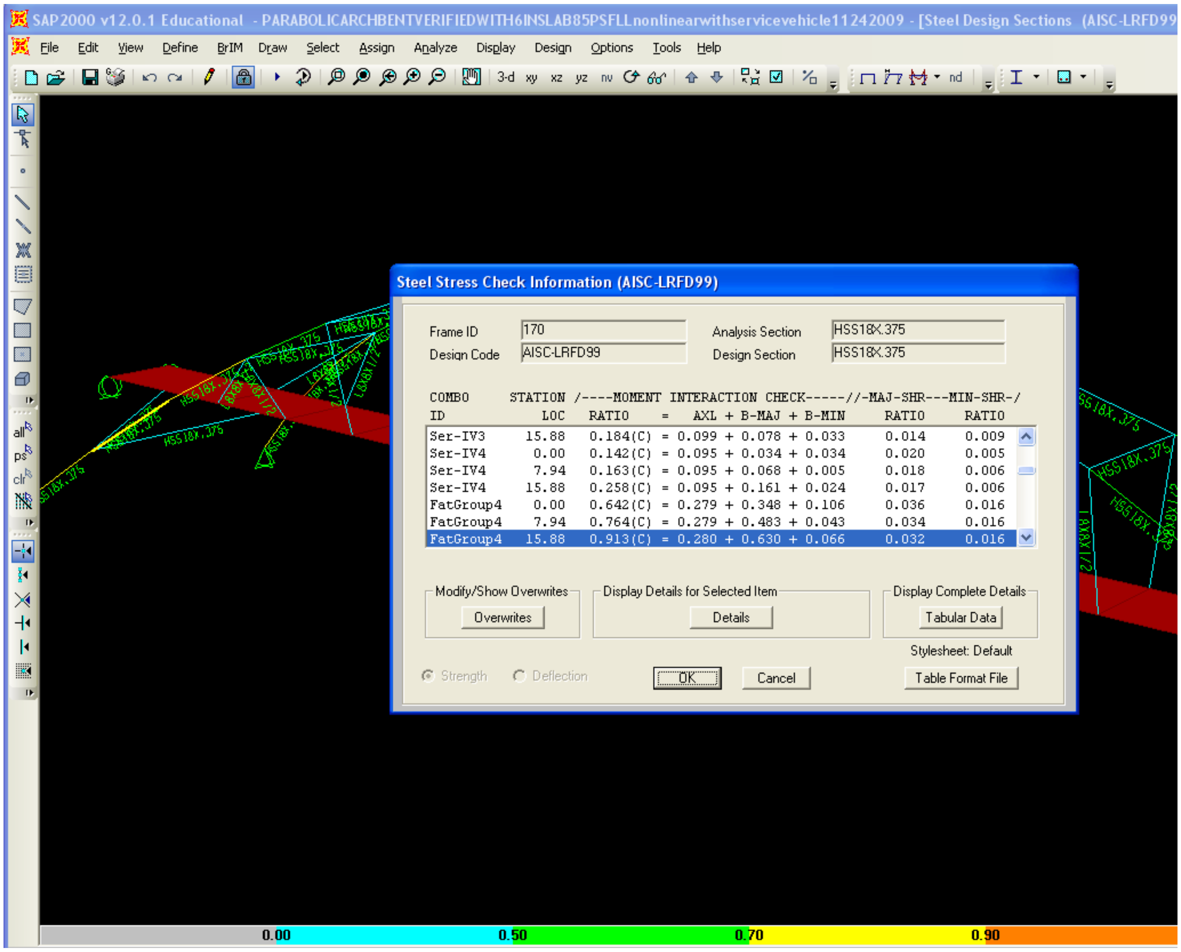

Figure 69. SAP2000 screen shot for max design force in HSS member.

SAP2000 Steel Design

Project________________
Job Number________________
Engineer________________

```
AISC-LRFD99 STEEL SECTION CHECK
Combo  : FatGroup4
Units  : Kip, ft, F



Frame  : 170              Design Sect: HSS18X.375
X Mid  : 190.313          Design Type: Brace
Y Mid  : 20.405           Frame Type : Ordinary Moment Frame
Z Mid  : 13.606           Sect Class : Non-Compact
Length : 15.879           Major Axis : 0.000 degrees counterclockwise from local 3
Loc    : 15.879           RLLF       : 1.000

Area   : 0.135            SMajor : 0.048         rMajor : 0.520          AVMajor: 0.121
IMajor : 0.036            SMinor : 0.048         rMinor : 0.520          AVMinor: 0.121
IMinor : 0.036            ZMajor : 0.063         E      : 4176000.000
Ixy    : 0.000            ZMinor : 0.063         Fy     : 7200.000

STRESS CHECK FORCES & MOMENTS
    Location          Pu         Mu33        Mu22         Vu2         Vu3          Tu
    15.879       -215.251     278.201      29.198       0.000       0.000     -33.299

PMM DEMAND/CAPACITY RATIO
    Governing         Total           P      MMajor      MMinor       Ratio      Status
    Equation          Ratio       Ratio       Ratio       Ratio       Limit       Check
    (H1-1a)           0.913   =   0.280   +   0.630   +   0.066       0.950          OK

AXIAL FORCE DESIGN
                         Pu     phi*Pnc     phi*Pnt
                      Force    Capacity    Capacity
    Axial          -215.251     770.062     873.000

MOMENT DESIGN
                         Mu      phi*Mn       Cm       B1       B2        K        L       Cb
                     Moment    Capacity   Factor   Factor   Factor   Factor   Factor   Factor
    Major Moment    278.201     392.748    0.821    1.000    1.000    1.000    1.000    1.226
    Minor Moment     29.198     392.748    0.600    1.000    1.000    1.000    1.000

SHEAR DESIGN
                         Vu      phi*Vn      Stress      Status          Tu
                      Force    Capacity       Ratio       Check     Torsion
    Major Shear       8.400     261.900       0.032          OK       0.000
    Minor Shear       4.118     261.900       0.016          OK       0.000
```

Figure 70. SAP2000 steel section check (critical member).