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# Analysis and Design of Arch-Type Pedestrian Bridge for Static and Dynamic Loads

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## ABSTRACT

A major obstacle for pedestrians south of the Indiana-Purdue University Fort Wayne (IPFW) campus is Coliseum Boulevard: a main arterial for the city of Fort Wayne, Indiana, which has an average daily traffic (ADT) of 50,000 vehicles. With this high ADT value, crossing by foot can not only be challenging, but can be dangerous. Thus, a pedestrian bridge over Coliseum Boulevard was proposed to 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 that already exist on the IPFW campus, the new bridge should be designed so that it too can be transformed into a landmark for the IPFW campus as the other two bridges have become. This paper presents the conceptual design of the pedestrian bridge considering four potential bridge concepts as well as the modeling, analysis and design details for the selected arch type pedestrian bridge. The selected concept for the pedestrian bridge was analyzed and designed using SAP2000 for static dead and live loads as well as for the wind loads according to AASHTO specifications and INDOT requirements. The same ideas could be drawn to Abdalli New Area in Jordan where innovative designs brings pleasure and comfort to residence and shoppers in the area.

## 1. Project overview

### 1.1 Importance and location

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 inception of the Crossroads partnership, the number of participating students has steadily grown to the level that 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 32 km (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, both campuses have been integrated into the design of the River Greenway Trail system.

Both of these projects face a common foe, Coliseum Boulevard (Indiana 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 between IPFW and Ivy Tech, as well as to connect the River Greenway Trail to Shoaff Park to the northwest of the IPFW campus. Figure 1 shows the proposed location of the bridge as determined by the IPFW Physical Plant in August of 2008.

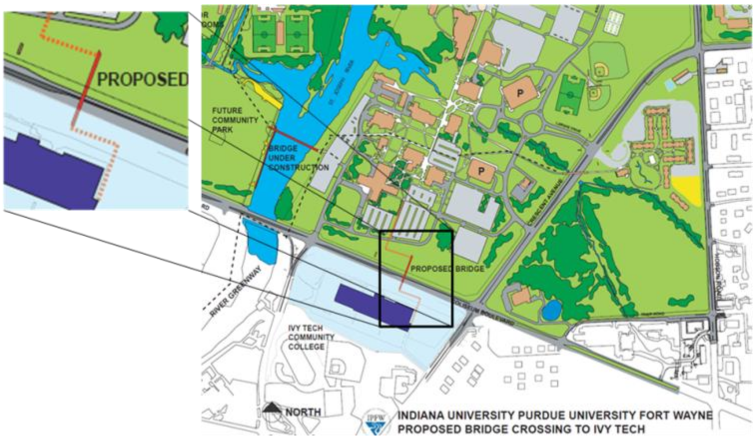


Fig1. Proposed Location of Pedestrian Bridge

The selected location is in an open area where there are currently no structures which 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 allowable slope (5%) 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. In addition to using the natural slope, the selected location minimizes the impact on vehicular traveler's view of the brick IPFW sign which is viewable to both eastbound and westbound traffic. 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 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 grade level is reached using the American's with Disabilities Act (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 a stairway that may be constructed next to the bridge which can give the pedestrians a direct exit from the bridge to the Ivy Tech campus. The selected location also allows for pedestrians to access the bridge from the

River green way Trail. By doing this, allows the River green way Trail to finally be able to connect to the parks and trails to the north of Coliseum Boulevard.

**1.2 Requirements and limitations**

The bridge must span entirely over Coliseum Boulevard due to the minimal width of median in the roadway and need to cohere to the ADA which sets a maximum slope of 5% for the walkway. The right of way is 24.4 m (80 ft) from each direction of the centerline of Coliseum Boulevard and the width of bridge is to be 3 m (10 ft). The bridge shall be designed for a minimum life span of 50 years. A clearance height of at least 5.35 m (17.6 ft) from the top of the existing pavement is required. The minimum pedestrian live load is 4.3 kPa (85 psf) and the design wind speed is 14.8 km/h (90 mph) for a 3 second wind gust. The bridge must be designed according to American Association of State Highway and Transportation Officials (AASHTO) as well as the Indiana Department of Transportation (INDOT) requirements.

In addition to meeting the above requirements and specifications, there are also numerous design variables that must be considered for the bridge. For aesthetic considerations, the selected bridge type should have an innovative design to fit with the other two pedestrian bridges on the IPFW campus: the Willis Family Bridge (relies upon two cables suspended from triangular-shaped supports to carry the bridge deck), and the Venderly Family Bridge (cable-stayed bridge consisting of two main towers with anchored cables to support the bridge deck). 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. With the main classroom building for Ivy Tech being close to the road, the design could include an additional structure that would connect the bridge directly to the building. This would allow for ease of use for the students as they would be directly in the Ivy Tech building once they cross Coliseum Boulevard. Another design variable is whether a covered or uncovered path will be used.

With the tough current economic times, 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. Although the preliminary design 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 the fact that the ADT is 50,000 vehicles on Coliseum, which dictates the need to minimize the adverse effects of closing the road down for long periods of time. In addition, the length of the intended steel members of the bridge must be limited to a maximum length of 30.5 m (100 ft) long and 4.3 m (14 ft) tall, which is the maximum allowable length to be transported on a trailer.

The above thinking is not confined to USA experience, others worldwide use such thinking, for example in Jordan pedestrian bridges of various shapes and structures have been used to connect sides of highways and under certain occasions to connect buildings laying at two sides of roads. Cost of them is not prohibitive, but with wise management it could also be significantly reduced.

## 2. Analysis and Design Software

SAP2000 was used to model, analyze and design the pedestrian bridge due to the software's flexibility that allows for linear, nonlinear, static and dynamic analysis and design of two and three dimensional structures as complicated as the "Bird's Nest" Stadium from the Games of the XXIX Olympiad. Any structural design completed in SAP2000 may be broken into four steps: modeling, analysis, display analysis, and design. The first step in modeling with SAP2000 is to define the model type and establish grid system to lay down the various members with their actual dimensions. Prior to placing members in the model, the materials types and properties must be defined. The software includes many predefined steel sections. In addition, the user can create any shape as well as assign any materials properties to it. 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/restraints to precisely model the joints, and applying any releases to the members. The last step in the modeling process is to apply the loads to the structure. SAP2000 allows for live, dead (which includes the structures self weight), moving, earthquake, and wind loads that can be analyzed both separately and concurrently according to major design codes such as AASHTO LRFD.

If the user has taken the time to meticulously set up an accurate model, analysis of the structure becomes streamlined. With the loading conditions already applied to the model, all the user must do is to determine which load cases they would like to run, and then 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 during the analysis process. In some cases, numerous iterations may be needed in order for an acceptable convergence value to be established. Following the completed analysis of the structure, the user is then able to view the mechanical behavior of the structure. For every different display option that can be selected, the user is given the option to view the results per the selected loading condition.

Once the analysis is completed, 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. Much like the deformed shape display, this view allows the user to determine if the structure is acting accordingly to the design load cases acting on it.

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 design option and code. 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, the option "Verify Analysis vs. Design Section" will 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, the analysis and design are easily repeated until the analysis and design members converge.

## 3. Conceptual Design

Four concepts were considered for the pedestrian bridge: cable-stayed, truss, suspension, and arch (Fig. 2). The advantages and disadvantages of each concept were considered in order to select the best superstructure type for the project location.

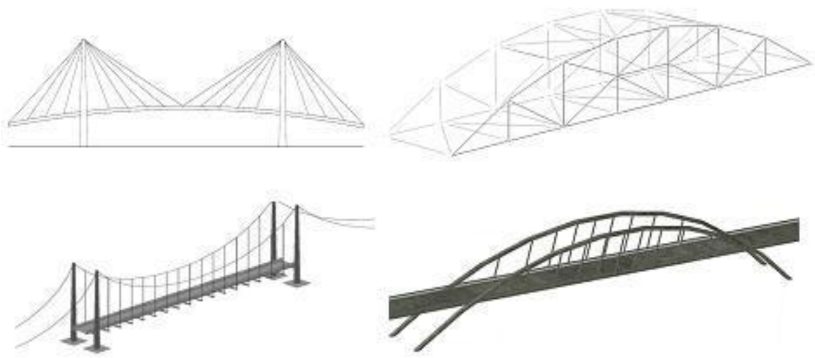

Fig.2. Pedestrian bridge options, clockwise from top left: stayed cable, truss bridge, arch bridge, and suspension bridge

The advantages of cable-stayed bridge include its aesthetically pleasing view, and ability for long spans. The disadvantages include: the need for adequate spacing on either side of columns to reduce eccentric loading, covering takes away from the appeal of the design, more cost effective for long spans (not for this short of a span), difficult to construct, and already one on the IPFW campus (crossing the St. Joseph River).

The advantages of the truss bridge include its low cost, ease of construction, and ability to cover it while maintaining original appearance. For these reasons such bridges exist in third world countries like Jordan. The disadvantages are: it is not aesthetically pleasing and will not be compatible with the innovative design of the other two bridges on the IPFW campus.

The advantages of the suspension bridge include its aesthetically pleasing view, comparable design with the other bridges on campus, and the ability for long spans. The disadvantages include the need to have adequate distance for anchorage points on either side of the main supporting columns (space is limited on Ivy Tech side of the bridge), difficult and unattractive to cover, expensive to construct, and would need to close Coliseum Boulevard for an extended period of time during construction.

The advantages of the arch bridge include its aesthetically pleasing view, new concept to IPFW campus, can easily be covered, construction can be formed to minimize the impact on the traffic on Coliseum Boulevard (many of the pieces can be prefabricated), cost effective given the bridge requirements. The disadvantage includes the large horizontal forces applied to the foundations from the arch which require special footing design.

The selected concept was the arch bridge due to its significant advantages when compared to the alternatives at this location. An arch is an excellent choice in supporting long span structures due to their ability to reduce bending moments and shear forces 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 49 m (160 ft)” (Chen and Duan, 1999). By limiting the bending and shear stresses induced on the arch structure, member sizes are reduced. With compression forces being the main forces the arch is supporting, 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. Depending on its given application, various types of arches may be chosen to support a given loading condition. The analysis and design of this pedestrian bridge will utilize the three-hinged arch concept that 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. With the arch being statically determinate, the structure is not affected by settlement or temperature change leading to the three-hinged arch being an excellent option when designing an arch structure (Hibbeler, 2005).

## 4. Arch Bridge Modeling

In order to begin the detailed design of the arch bridge, the bridge was designed initially using a normal arch without any modifications. Designing the arch in the xz plane allows for verification that the structure is accurately modeled in SAP2000 with hand calculations based on structural analysis. The first step in making sure that the bridge is modeled correctly is to verify that the load on the deck is transferred correctly to the arch members. Hand calculations of the support reactions using the principles of structural analysis showed a difference from the SAP2000 analysis of approximately 5%. The deformed shape and mode shapes were logical. Therefore, it was determined that the model is a reasonable illustration of the proposed pedestrian bridge.

Although the arch is drawn as a parabolic arch, the shape is not completely parabolic. 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 slight shear and moment forces to be introduced into the arch. It is the effects of these forces that cause portion of the slight variance in the SAP2000 analysis versus structural analysis.

A user defined grid system was established to draw the bridge members that ensure the height requirements of 5.35 m (17.6 ft) from top of pavement to bottom of lowest bridge member. The required span of the bridge was 64 m (210 ft) with a maximum height of 13.4 m (44 ft). 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, 1999).

Hence, the grid was set up as follows: 211 X-units at 0.3 m (1 ft) spacing, 2 Y-units at 3.0 m (10 ft) spacing, and 45 Z-units at 0.30 m (1 ft) spacing.

Immediately after defining the grid system, the materials and members used in the model were defined. This allows for the design to go smoothly since all member shapes and sizes are defined prior to drawing any of the members. Since the main forces carried in the arch members are compression forces, hollow steel sections (HSS) were chosen for the main arch for their known performance in supporting large compressive forces. From the accompanying database included in SAP2000, it was able to import various HSS standard sections into the model. Once the sizes were brought into the model, an auto select list named "HSS" was defined to allow for 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 where the software will optimize the member size eliminating the need for a "trial and error" approach. Using the draw frame member option, the arch members were modeled as continuous curved frames with HSS section type. In defining the curved arches, when clicking on the initial reference point (0,0,0), and then dragging the mouse to (64,0,0), a dialog box appears prompting the user for some information in determining the shape of the curved frame members. In the box for curve type, the "Parabolic Arch – 3$^{rd}$ Point Coordinates" was selected in order to draw a parabolic arch. Selecting the 3$^{rd}$ point coordinate as (0, 32, 13.4) 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.

Preliminary trial analysis and design for the bridge showed that keeping the arch as a single object leads to inaccurate analysis. Instead, the arch members are modeled as multiple equal length objects. It was decided to use 16 similar sized members to form the main arches, and the ends of these segments will be used as the joints where the cables would transfer the bridge deck loads to the arch. In addition, an internal pin connection was defined at the apex of the arch so that the structure could be analyzed as a three-hinged arch.

With the first arch member in place, the second arch was created by linearly replicating the first arch at a distance of 12.9 m (42.4 ft) in the Y-direction. The final design was intended to have the bridge composed of arched members that angle into the center of the walkway to give a more aesthetically pleasing look. Angling the members was conducted by replicating the arch along the Y-axis 23º into the center along the line that makes up the base of the structure as shown in Fig. 3.

In order to consider the fact that the cables can only support tensile stresses, a frame compression limit of 0 was assigned to all of the angled cable members. To achieve this, 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 they generate tensile force only in the cables; however, for all of the dynamic loading cases (wind load cases and 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 for the cables were selected from an auto select list, only this time they are defined as "ANGLE". Drawing the angled cables from the arch down to where the bridge deck will be was made easier due to the carefully defined grid system. Also during this step, the lateral supports between the arch members were drawn (Fig. 3), 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.

The bridge deck slab materials were defined as 28 MPa (4 ksi) normal weight concrete. Prior to modeling the bridge deck slab, its thickness was determined to be 0.15 m (6 in.) based on deflection considerations provided by the ACI 318-08 code. All of the deck sections were drawn in with the "Quick Area" tool in SAP2000. All of the deck sections are the same with dimensions of 3 m (10 ft) wide and 4 m (13.1 ft) long to allow for easier prefabrication and constructability. The concrete slab can be accurately defined in the "Areas Section: Shell Section Data". In this menu, for more accurate analysis, the slab was defined as a layered shell element which takes into account the composite nature of the concrete slab. After drawing all of the members, they were auto meshed using the meshing function.

As shown in Figure 3, the restraints at all four points of the arches 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, which allowed for the software to analyze the joint as a pinned-connection. Another restraint used for all of the angled-cable members, the lateral supporting HSS members, and the side beams of the bridge deck was that they were all released from any moment forces at their ends to be analyzed as pin-pin connections at all joint locations. The final modification used in the model was the release of any compression forces that may form in the cable members as outlined earlier.

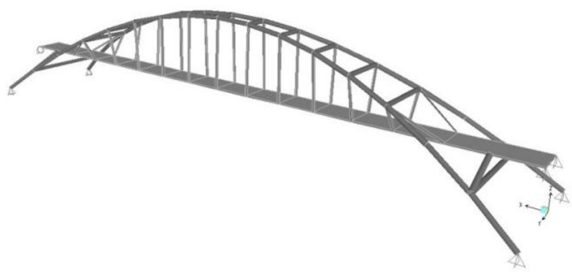

Fig.3. Complete model of the pedestrian bridge showing the major structural members

## 5. Loading

For the pedestrian bridge, 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. Since the deck will be made from pre-cast concrete, once the bridge is built on the site, an overlay will need to be added in order to provide a smooth surface. For this, the contractor may decide to coat the top of the concrete with an overlay; therefore, an additional load of 0.5 kPa (10 psf) has been added. In addition to the load from the deck, there was also a 1.22 kN/m (90 plf) load applied on either side of the walkway that takes into account any railing/fencing. The railing/fencing load was transferred to the structure by a user defined load of 1.22 kN/m (90 plf) on the edge beams that support the concrete deck. Since the edge beams will be designed through hand calculations, a load was also applied to the edge of the deck for the self weight of the beams. With the edge beams designed as 0.25x0.4 m (10x16 in.) rectangular sections, 4 kN/m (300 plf) was added to either side of the deck. Finally, the self weight of the structure itself, including all HSS, Angles, and Beams, is calculated in SAP2000.

There were three types of live loads applied to the bridge: pedestrians, wind, and a moving service vehicle. As described in the LRFD Guides Specifications for Pedestrian Bridges, the specified live load for a pedestrian bridge is 4.3 kPa (90 psf). Previously, the LRFD specified 4.0 kPa (85 psf); however, with the changing factors that the LRFD has used over the years, it has been found that a 4.3 kPa (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. 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). Therefore a live load of 4.0 kPa (85 psf) was applied with a check on the dynamic response.

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 a maintenance vehicle needs to access the walkway (i.e. removal of snow on the concrete deck). As detailed in AASHTO LRFD code, with the walkway on the bridge being only 3 m (10 ft) wide, the code recommends using an H5 design service vehicle. The AASHTO LRFD code states also that the service vehicle load is not combined with the pedestrian live load. In order to apply the service vehicle load, lanes were defined on the bridge deck where the vehicle would travel on. Two lanes were defined on the bridge deck, each 0.75 m (2.5 ft) centered from the exterior edge of the deck. The design vehicle was modeled in SAP2000 by creating a new service vehicle in the software since the H-5 vehicle was not a standard vehicle.

For any structure, the force applied on it by the wind is a major concern. Unlike the loads previously discussed, the wind loading is applied perpendicular to the structure, and not in the direction of gravity. To determine the force from the wind, the maximum wind speed that the bridge should be designed for was found using the basic wind speed maps found in ASCE 7-05. Based upon this map, the design wind speed for the structure is 145 km/h (90 mph). Applying the wind load to the bridge was performed in SAP2000 through the user defined loading patterns. In this menu, three different wind load conditions were defined: WIND, WIND2, and WIND3. Each was defined in SAP2000 as wind loads, and based the conditions on the ASCE 7-05. Surfaces of the structure that would be exposed to wind were then defined by entering the frame and area objects of the bridge that would be exposed while the structure itself would be open. The next step 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, 90º, and 45º, respectively. The final step to complete in this box is to determine the wind coefficients. The exposure type of the bridge was defined to be "B", since the bridge would be located in an urban environment. For the cables, in order for the compression limit of 0 to be considered, the wind loading has to be calculated using a nonlinear analysis. This step was completed in SAP2000 through the “Define Load Case” box, and then defining each of the wind load patterns to be performed as a nonlinear analysis. Table 1 shows a detailed summary of the loads applied to the structure.

Table 1: Summary of loads applied to structure

| Loading pattern | Weight |
|---|---|
| Dead load | Self weight of structure<br>Concrete = 2400 kg/m$^3$ (150 pcf)<br>Overlay/Surface = 48 Pa (10 psf)<br>Railing/Fencing = 1220 N/m (90 plf) |
| Live load | 4 kPa (85 psf) (pedestrian/snow) |
| Moving load | H5 service vehicle = 44.5 kN (10 k) |
| Wind loading conditions | |
| Auto lateral load pattern | ASCE 7-05 |
| Wind speed | 145 km/h (90 mph), 3s 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 |

## 6. Structural Analysis

Once the geometry and load cases are modeled in SAP2000, the next step was to perform the structural analysis on the bridge. When moving to this step, the software has the option of analyzing all, or only one, of the loading cases. Following is a summary of the major SAP2000 analysis results.

## 7. Deformed shape

In order to verify that the structure is modeled correctly, the deformed shape given in SAP2000 with the anticipated deformed shape must be compared. If the deformed shape of the structure looks abnormal, then something is modeled incorrectly; however, if the deformed shaped looks accurate, the chances are high that the structure was modeled correctly. Figures 4 and 5 display various deformed shapes for the different load cases. The deformed shapes for both the dead and live load cases (Fig. 4) are similar with the only difference being the magnitude of the deflection in each case. 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 magnified to show how the bridge is expected to deform. 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. H5-2 load case controlled the steel design of the structure. The various wind load cases WIND, WIND2, and WIND3 all have different deformations (Fig. 5) associated with them since each case represents a different angle at which the wind acts on the bridge.

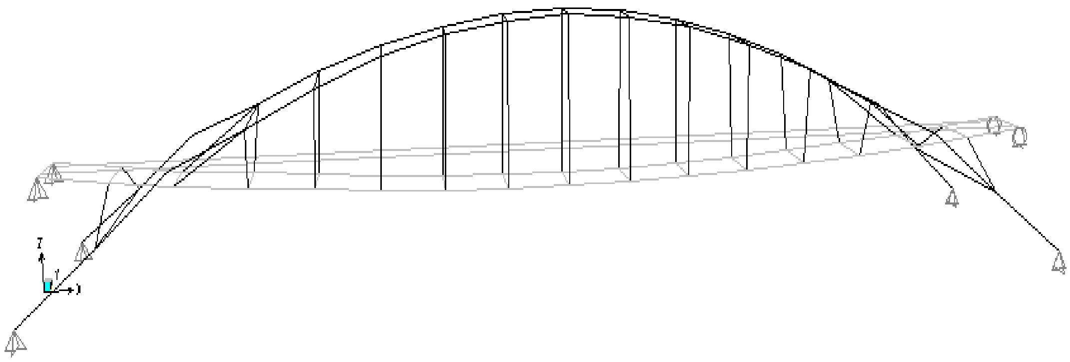

Fig.4. XZ-plane view of deformed shape under dead and live loads

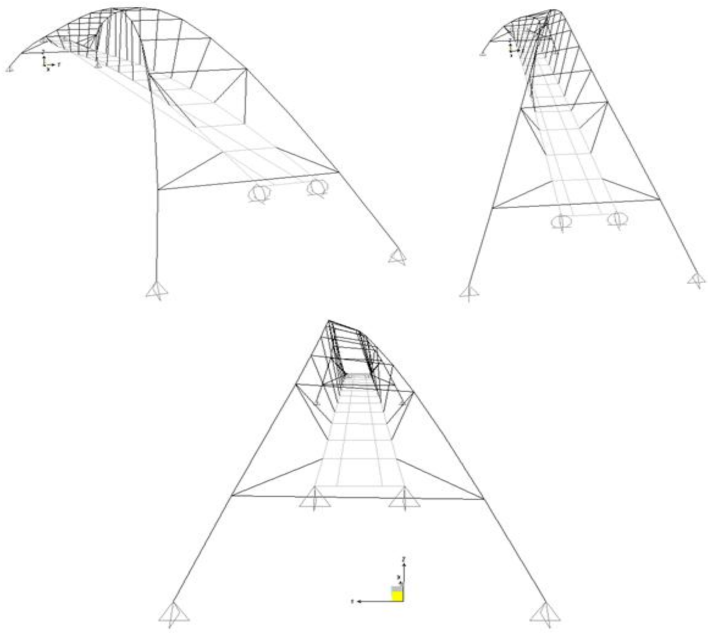

Fig.5. Deformed views for WIND (top left), WIND2 (top right), and WIND3 (bottom)

Reactions and internal forces

After checking the validity of the deformed shapes for the various loading conditions, the next option was to investigate the support reactions and the internal forces. The reactions were symmetrical at both ends for the dead load case. Similar to the dead load case reactions, support reactions for the various load cases were reasonable in terms of magnitude and directions. The forces that the cable members were experienced were displayed for typical load cases (Fig. 6). 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, indicating that the compression limit of zero was executed correctly. Also the results show that the arches are in compression for the gravity load cases, while portions of the arches experience some tension during the wind load cases.

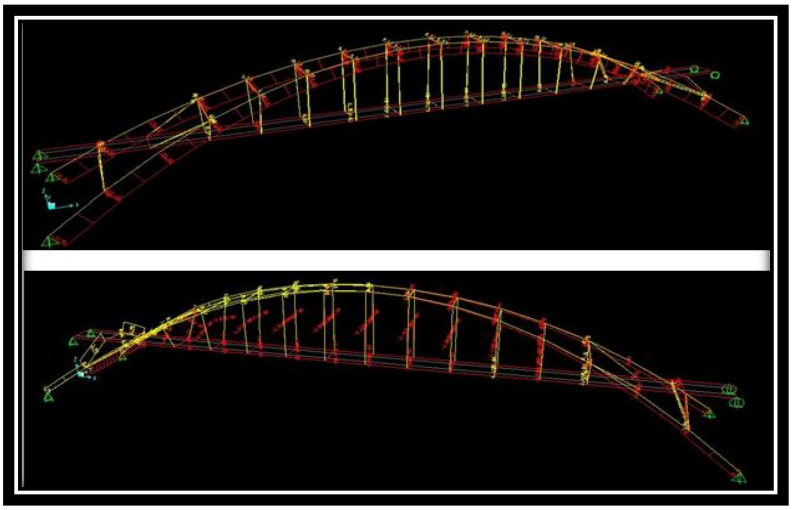

Fig.6. Axial forces in the structural members

## 8. Influence Lines

Yet another powerful tool with SAP2000 is its ability to display influence lines for joints and structural members. A quick glance at the influence lines serves as an aide to where on the structure the member is most affected by the moving load. In addition, it is another verification of the accuracy of the modeling. Figure 7 shows typical influence lines generated by SAP2000 for the support reaction at the start of the span (top), axial force in the cable member at the mid-span (middle), and axial force for the arch members at the end of the span (bottom). The obtained influence lines were all reasonable which confirms the accuracy of the modeling.

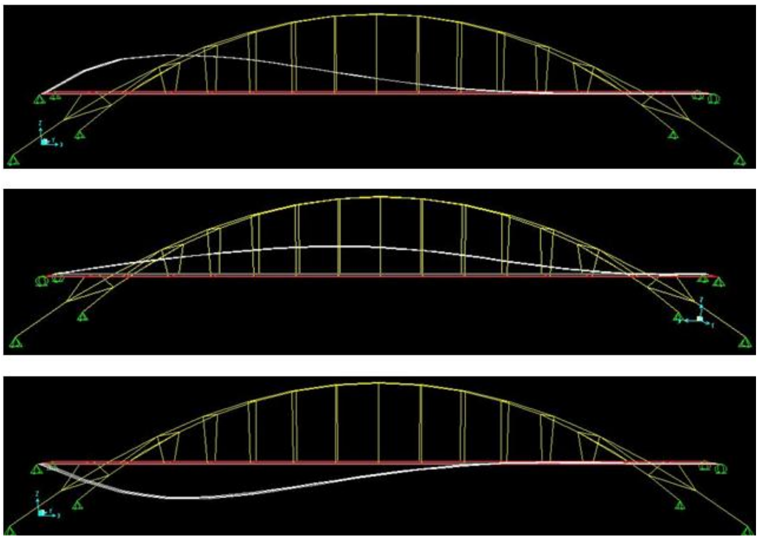

Fig.7. Influence lines for the support reaction (top), cable member at the mid-span (middle), and axial force in the arch at the end of the span (bottom).

## 9. Structural Design

### 9.1 Load combinations

The default load design combinations for bridges that are integrated in the SAP2000 were used 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. According to the AASHTO LRFD code, some of the load combinations are used for strength design while others for serviceability issues and fatigue design. Of the many different load 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 was due to fatigue loading. It was noticed from the deformed shapes that large deformations occur 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 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 H5 moving load was analyzed 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 16x0.375 to HSS 18x0.375.

### 9.2 Design of arch members

Using the steel design function in SAP2000, the steel arch members were adequately sized 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 controlling load case combinations. The maximum axial compression force in the arch members was calculated to be 1467 kN (330 kip). In order to support this load, the software selected an HSS 18x0.375 for the arch members. As stated previously, when using only the DSTL2 loads, HSS 16x0.375 would be adequate, but by performing an advanced dynamic analysis of the moving vehicle load, SAP2000 designed the arch members to be the larger 0.48 m (18 in) diameter HSS. The SAP2000 was also asked to use similar structural steel shape for all of the members used in the arch including the cross members that are used for the lateral support of the two main arches HSS 18x0.375. Although this size is way more than adequate for the loads that are in the cross members, but it will create a more aesthetically pleasing look for the bridge.

### 9.3 Design of cables

Since the cable members were modeled as angle members in SAP2000, hand calculations were performed to determine the size of cables needed to support the bridge deck. The maximum force applied to the cables for the various load combinations was determined in SAP2000 and used for the design of the cables. Based upon a maximum tensile force of 119 kN (26.7 k) from the load case DSTL2, the required diameter of the cable members was determined to be 25 mm (1 in) using A36 steel with $F_u$= 400 MPa (58 ksi). The cables will tie into the concrete edge beams and 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.

### 9.4 Design of slab & edge beams

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. Forcing the deck to be simply supported allows for performing a set of hand calculations to determine all relevant design information for the concrete deck. Based upon the calculations using the ACI 318 code, the deck was designed to have 0.15 m (6 in) thickness. The main reinforcing steel required for the longitudinal direction was determined to be No.3 rebars, placed at 18 cm (7 in) center to center. The minor steel reinforcement that is needed for shrinkage and temperature was determined to be No.3 rebars, placed at 250 mm (10 in) center to center. The concrete edge beams carry the loading transferred from the concrete deck to the cables hanging from the arch members. The edge beams were designed to support 7.1 kN/m (1.6 k/ft) (from factored live and dead loads) which transfers the loading from the decks to the beams, a 0.4 kN/m (0.09 k/ft) for railing, and their self weight. The design of the edge beams results in the use of a 0.25x0.4 m (10x16 in) rectangular cross-section with three No.4 bottom steel rebars and two No.4 top steel rebars. As far as the shear reinforcement that is required, No.3 at 160 mm (6½ in) spacing was appropriate.

### 9.5 Vibration limitations

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 Hz, in the absence of any applied live loads. If the fundamental frequency does not satisfy this limit, then a more in depth look at the dynamic performance of the bridge must be undertaken. The LRFD Guide has the following simple formula to determine the fundamental frequency of a pedestrian bridge:

$$f \geq 2.86 \ln\left(\frac{180}{W}\right) \tag{1}$$

Where $f$ is the fundamental frequency (Hz) and W is the total weight of the supported structure (kip). 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 approximate weight of the concrete deck was determined. 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. The weight of the deck was calculated to be 2067 k. Using this weight in the frequency equation gives $f$ = -6.98 Hz, which exceeds the minimum value of 3.0 Hz. Therefore, no further vibration analysis is required for the structure.

## 9.6 Deflection limitations

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 maximum deflection was found to be less than 6.4 mm (¼ in). Using the maximum allowable deflection per NCHRP 20-07 for a pedestrian bridge being L/500, it is determined that this pedestrian bridge meets the deflection requirements. This was calculated from the maximum member length of the concrete being 4 m (13.1 ft) leading to a maximum allowable deflection of 8 mm (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.

## 9.7 Final design

Another feature of SAP2000 is the ability to prepare advanced technical reports for the structure that the engineer is designing. 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. For the final design of the structure, the model of the bridge was imported from SAP2000 into AutoCAD where the structure was rendered as shown in Figure 8.

Fig. 8: D rendering of the final design.

## 10. Construction Techniques

As described in 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 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, it is more reasonable to use the shop control procedure due to the relatively short span 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 and the pier to support any loads carried by the members. For the false framework 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 best construction method would be the tie back method with minimal impact on traffic.

## 11. Concluding Remarks

Based upon considering the advantages and disadvantages of various alternatives, it was concluded that the arch-type pedestrian bridge is the most appropriate design for the intended pedestrian bridge on Coliseum Boulevard. Its unique features include its aesthetically pleasing look, ability to be constructed in a short time with minimal impact on the traffic flow, and the effectiveness of the arch members in resisting the applied loads mainly in compression. The use of computer analysis software such as SAP2000 is a very effective and innovative approach for the design of pedestrian bridges especially when wind and dynamic loads are to be considered. Care must be taken to verify the accuracy and reasonability of the model before proceeding to the final design. Construction methods must be considered during the design process to allow for smooth project progress without potential interruptions and to reduce the overall project cost.

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