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14. ABSTRACT
The Anchorage Group (also referred to herein as “the Group”) has provided engineering, architectural, construction and financial consulting services for the renovation and design of the new Charles River Crossing in Cambridge, Massachusetts. In this report the Group provides the client with a detailed design that aims to provide a safe crossing of the river and a way to bypass the major roads for non-vehicular traffic. The Group has also taken into account the need to renovate the existing bridges at this location.

This report has been broken into four sections; the first section describes the project background and the general conditions of the site. The second section details the new river crossing while the fourth details the new road bypass. Finally, the fourth section presents a short summary of the Group’s proposed design and methods.

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CHARLES RIVER CROSSING

BY

THE ANCHORAGE GROUP

APRIL 6, 2012

Pierre Dumas, Timothy James, Nnamani Nnabuihe, Stephen Pendrigh, Erika Yaroni

1.562 MEng Project - High Performance Structures

EXECUTIVE SUMMARY

The Anchorage Group (also referred to herein as "the Group") has provided engineering, architectural, construction and financial consulting services for the renovation and design of the new Charles River Crossing in Cambridge, Massachusetts. In this report the Group provides the client with a detailed design that aims to provide a safe crossing of the river and a way to bypass the major roads for non-vehicular traffic. The Group has also taken into account the need to renovate the existing bridges at this location.

This report has been broken into four sections; the first section describes the project background and the general conditions of the site. The second section details the new river crossing while the fourth details the new road bypass. Finally, the fourth section presents a short summary of the Group's proposed design and methods.

The Anchorage Group proposes a three span arch bridge, with the bridge deck suspended from the arches via cables. The arches have been designed to 'hop' across the bridge deck from one side of the traffic to the other, while also seeming to form a wave in the air by connecting the arches together with non-structural members. The bridge has been designed to temporarily take one lane of light-weight traffic during the renovation of the two existing bridges. Two separate schedules and cost estimates have been developed. The first, a fast-track method, estimates 6.5 months completion with a construction cost of $2.8 million. The second follows a sequential sequence and is estimated to take about 11 months to complete with a $3.0 million construction cost.

The Group proposes hinged underpasses for the road bypass. There will be a total of 4 underpasses which will pass under the outer arches of the Western Street and River Street Bridges. The underpasses have been designed to lift up, allowing for maximum river use during competitions such as the Head of the Charles. One underpass is estimated to take 63 days to construct and cost around $632 thousand. Given the qualifications of the team described in this report, The Anchorage Group is capable of addressing all aspects of the project, from the design phase through budgeting, scheduling, and construction.

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THE COMPANY

ANCHORAGE Group, Ltd

Consultants in Civil Engineering

Anchorage-New York-London-Lagos

All information that is provided about the company is purely fictional and is provided for realism.

COMPANY OVERVIEW

The Anchorage Group provides engineering, architectural, construction and financial consulting services to private and institutional entities willing to change the built environment. The Group specializes in providing clients with state of the art turn-key solutions through the duration of the project: from conception to project completion.

We deliver the most economical solutions as well as signature projects that make the Group one of the most recognized and respected design-build construction firms in the world. Helping clients meet their goals and completing breathtaking projects is the Group's daily motivation. This commitment is reflected in the company's motto: "make it happen".

CORE SKILLS AND OFFERINGS

Since its inception in 1948, the Anchorage Group has been able to combine its expertise in architecture, structural engineering, and project management to deliver world-class projects on time and under budget. Individual resumes for the members of design team for this project can be found in Appendix O. The experience gathered over the years has given the Group expertise in the following areas:

- Environment and Sustainability: As a matter of priority, the Anchorage Group keeps up with global trends in sustainability. The Group strives to meet the most demanding standards anywhere in the world by limiting the impact of projects on the natural environment and targeting the Leadership in Energy and Environmental Design (LEED) certification.
- Cost optimization: Relying on the technical knowledge, equipment and resources at its disposal, the Anchorage Group has the capacity to deliver finished projects within budget. The best practices developed over the years executing technically intensive projects gives the Group the unique knowhow to implement the most cost effective methods to tackle any structural and construction challenge.
- Structural Engineering: The Anchorage Group has developed the reputation for specializing in and leading the development of the most complex structural projects. The Group can confidently rely on its technical prowess and its international network of colleagues and associates to deliver innovative solutions in a timely manner.

PORTFOLIO

The Anchorage Group boasts a long and proud history of successfully designing iconic footbridges around the world. Several projects are highlighted below.

DNA BRIDGE, MARINA BAY SANDS, SINGAPORE

Figure 1: DNA Bridge

This modern marvel redefines the limit of artistic creativity and engineering genius. Completed in 2009, it is the world's first bridge based on the double helical structure of human DNA. The bridge spans 280 meters over the Marina Bay area and is equipped with computer-controlled lighting to enhance the visual appearance.

Although it functions as a standard beam bridge, the architectural façade highlights the Group's ability to be creative in tackling commonplace challenges. Its low profile also ensures that the current skyline around Marina Bay is not drastically altered.

LÉOPOLD SÉDAR SENGHOR BRIDGE, PARIS, FRANCE

Figure 2: Leopold Sedar Senghor Bridge

The "Passerelle" Leopold Sedar Senghor is an arch bridge situated right in the heart of Paris linking the banks of the Orsay Museum with the Tuileries garden.

The Anchorage Group successfully executed this project in a highly populated area of the city. This shows the Group's ability to work in busy parts of cities without significantly impacting the daily activities of residents and commuters. Additionally, the arch structure does not interrupt the navigational channel, which allows activities, like sailing, to proceed without obstruction.

HARBOR DRIVE PEDESTRIAN BRIDGE, SAN DIEGO, USA

Figure 3: Harbor Drive Pedestrian Bridge

This innovative bridge has become one of the landmarks of San Diego. It is a cornerstone of downtown San Diego's development and an iconic gateway to the city. It is one of the longest self-anchored pedestrian suspension bridges in the world.

This design illustrates the quality of the Anchorage Group's work and the diversity of solutions it is able to deliver in order to meet the demands of clients. It also depicts the Group's ability to develop cutting edge cable-stayed and suspension bridges that not only blend into a city's skyline but also help to increase the city's prestige.

PROJECT BACKGROUND

INTRODUCTION

The Western Avenue Bridge and River Street Bridge (Figure 4) are earth-filled, reinforced concrete arch bridges that cross over the Charles River, which flows between the cities of Boston and Cambridge (Figure 5). The two bridges were built in 1924 and 1925 respectively. Both bridges intersect with Memorial Drive and Soldiers' Field Road, and contain 3 lanes of traffic plus a pedestrian sidewalk on either side of the road. They both contain three arches to span the river, similar to other bridges upstream, allowing river traffic to pass beneath.

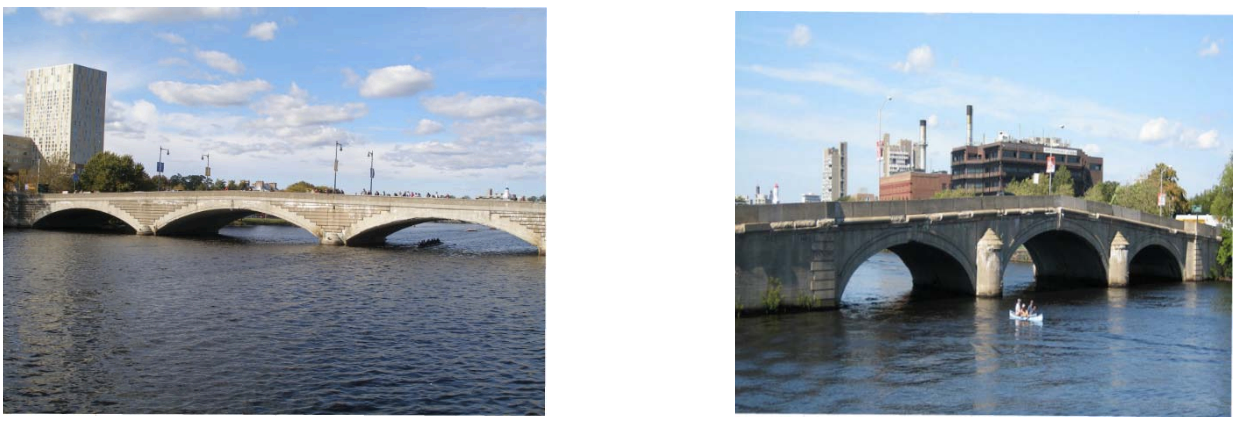


Figure 4: Western Avenue Bridge (left) and River Street Bridge (right)

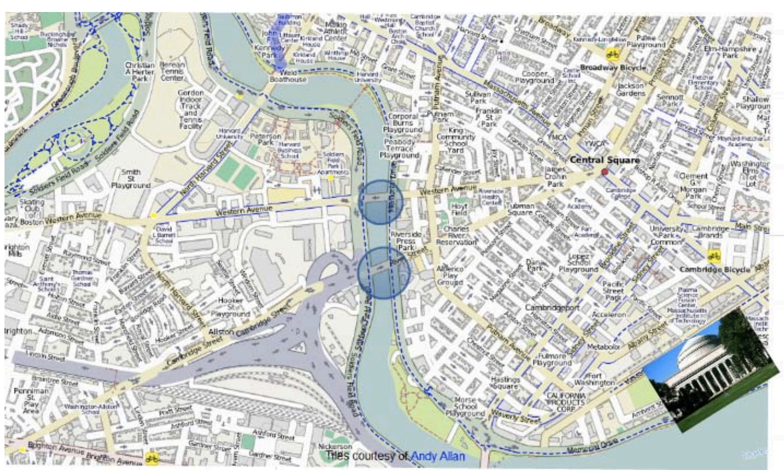


Figure 5: Arial Map of Site, Showing Existing Bridge Locations

Currently, both River Street Bridge and Western Avenue Bridge permit one-way traffic. River Street Bridge brings traffic from Cambridge to Boston, while Western Avenue Bridge allows traffic flow from Boston into Cambridge. There is a large volume of pedestrian traffic in the area, attributed to the local universities and residents enjoying the river walkways. Currently these trails require crosswalks and crossing lights at the foot of the bridges, which is disruptive to pedestrians, cyclists and motorists alike.

As both bridges have fairly low-lying arches, the river is navigable by small craft only. However, there is a significant amount of river traffic in the form of rowing shells and is generally part of major rowing competitions such as the Head of the Charles.

The two bridges are in need of significant renovation, with all the components of the River Street Bridge being listed in "fair" or "poor" condition by the Massachusetts Department of Transport (MassDOT). The Western Avenue Bridge is only slightly better with nearly all components in the same condition as the River Street Bridge (only the substructure and piers are listed as "satisfactory"). The MassDOT currently has plans to perform significant repairs to both bridges. The last renovation occurred in 1981 and only focused on road surface rehabilitation.

As part of this renovation project, there is a desire to ameliorate the bike and pedestrian access to the paths on either side of the river and to provide an additional bike and pedestrian river crossing. This additional crossing will allow for the removal of current sidewalks on the Western Avenue Bridge and River Street Bridge providing an additional driving lane.

The Anchorage Group had been asked to provide concept designs for the new bike and pedestrian river crossing and road crossing. Following consultation, the Group has been asked to provide further details on the accepted design.

EXISTING GEOMETRY

The Western Ave Bridge consists of three arches supported by concrete piers and spread footings set into granular soils and clay found underneath the river bed settlement. It carries both vehicular (three lanes) and pedestrian (two sidewalks) traffic across the Charles River and spans a distance of 329ft. The elevations of the top and bottom of the exterior arches are 20.42ft and

8.5ft respectively and are 60ft across. The interior arch has top and bottom elevations of 24ft and 8.5ft respectively and spans 75ft. The bridge deck's maximum elevation is 28ft and is 57ft wide; 40ft for vehicular traffic with 8.5ft sidewalks on either side.

The River Street Bridge also consists of three arches supported by concrete piers and spread footings and carries both vehicular (three lanes) and pedestrian (two sidewalks) traffic across the Charles River. This bridge spans a distance of 304ft. The elevations of the top and bottom of the exterior arches are 20.42ft and 8.5ft respectively and are 60ft across. The interior arch has top and bottom elevations of 24ft and 8.5ft respectively and spans 75ft. The bridge deck's maximum elevation is 28ft and is 57ft wide; 40ft for vehicular traffic with 8.5ft sidewalks on either side.

The average water level is 8ft above gauge height, with flood level reaching 8.5ft at the two bridges (which coincides with the bottom of the arches), with both bridges separated by a distance of 1100ft.

A detailed sketch of one of the existing bridges, River Street Bridge, can be found in Appendix B. This image includes dimensions for the clearance and span as well as other pertinent information.

DESIGN CONSTRAINTS

PEDESTRIANS/CYCLISTS

Both the river crossing and the road bypass should provide a safe, easy to use path for both pedestrians and cyclists. The road bypass should not interfere with vehicles at any of the four intersections of the existing bridges. Minimum width should be 10ft to allow for two way flow of foot/bike traffic.

VEHICLES

The river crossing should, ideally, include provision for temporary use of vehicles. Vehicle use of the river crossing will occur during renovation of the two existing bridges, Western Avenue Bridge and River Street Bridge, to ease traffic congestion of the local area. After renovations, no vehicular access of the new river crossing is needed.

Traffic flows along Soldiers Field Road and Memorial Drive should not be permanently rerouted to accommodate the new river crossing/road crossing unless deemed absolutely necessary.

River Traffic

River traffic should remain unchanged and the Anchorage Group should limit the amount of piers placed in the river. This is especially true for reducing the effect on large scale races such as the Head of the Charles, whose route passes through the area of interest.

Minimum Requirements

To comply with the Americans with Disabilities Act, the minimum gradients of all ramps shall be 1:12 for a maximum of 200ft. If the ramp should extend further than this, resting intervals shall be included.

The river crossing should be able to accommodate one lane of temporary traffic, which does not have to include trucks.

The minimum lane width to be used along the river crossing shall be 10ft, however if being designed for vehicular use the minimum lane width shall be 12ft. This minimum width shall allow for a single lane of vehicular traffic without pedestrian use. The clearance above the driving surface shall be at least 15ft.

To accommodate cyclists using the trail, a minimum turning radius of 100ft shall be used and a minimum clearance between piers shall be 44ft for river traffic; however, the ideal minimum should be 88ft to allow two rowing shells to pass simultaneously.

PROPOSED RIVER CROSSING

OVERVIEW

The Anchorage Group proposes a river crossing as depicted below in Figure 6. The proposed river crossing is a three span arch bridge, with a horizontal deck which is supported by the arches via cables. The main architectural feature of the bridge is the asymmetric form when cut along the longitudinal axis as seen in Figure 7.

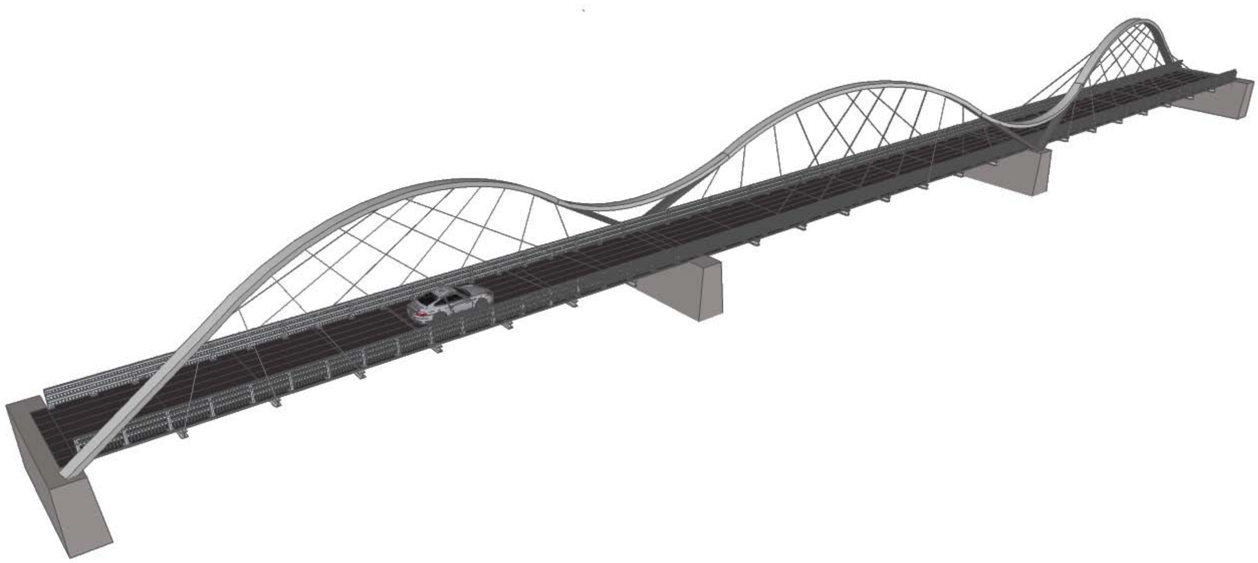


Figure 6: View of Arch Bridge

THE ARCHES

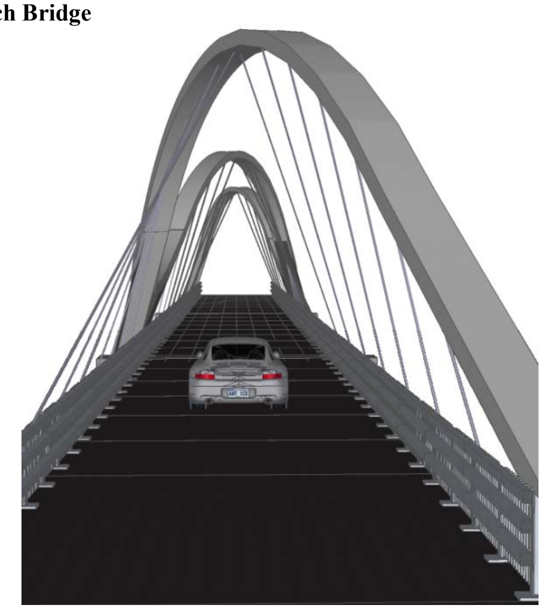


Figure 7: View Along Bridge

The idea of three arches was determined early on in the design stage, as the Group wished to mimic the style of the existing bridges along the river. The arches were then developed to 'hop' from one side of the road to the other as shown in Figure 7. This is not as noticeable when looked from afar up or downstream.

It was only a small change to link each of the arches together, to make it seem that the arches 'wave' across the river. The wave is then further improved when the bottom 'legs' of the arch, which only take axial load, are reduced in cross section and are finished in a different color, giving an appearance of the wave floating in space, as seen in Figure 8.

The shape of the arch was determined by finding a curve which produced zero moment throughout when subjugated to a uniform distributed loading. The Group then fitted this curve to the three known points (the two fixed ends and the height of the arch). The shape was then altered slightly so that the portions of the arch between the foundations and the first cables were straightened; this was done for aesthetics and ease of construction.

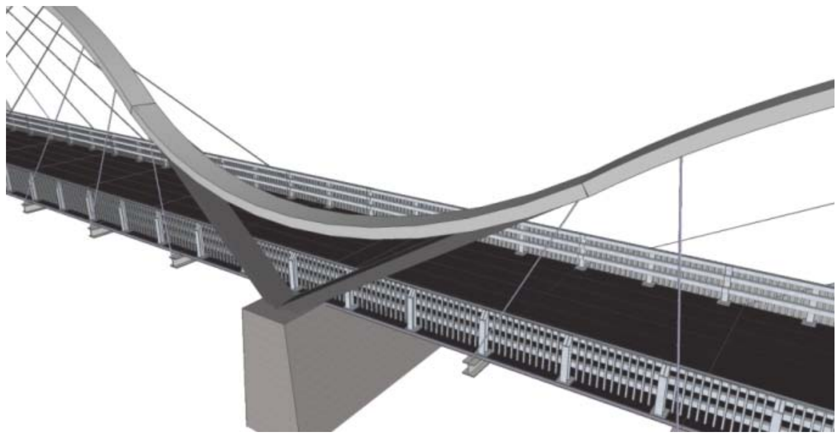


Figure 8: Connecting Arches

The Cables

The cables, as shown in Figure 9, have been designed so that one half of the bridge deck is attached to only one half of the arch and vice versa. This gives a very unique visual appearance for the bridge; however it also results in unwanted twisting of the arch. To counter this, the horizontal stiffness had to be increased by using a rectangular section with a larger width.

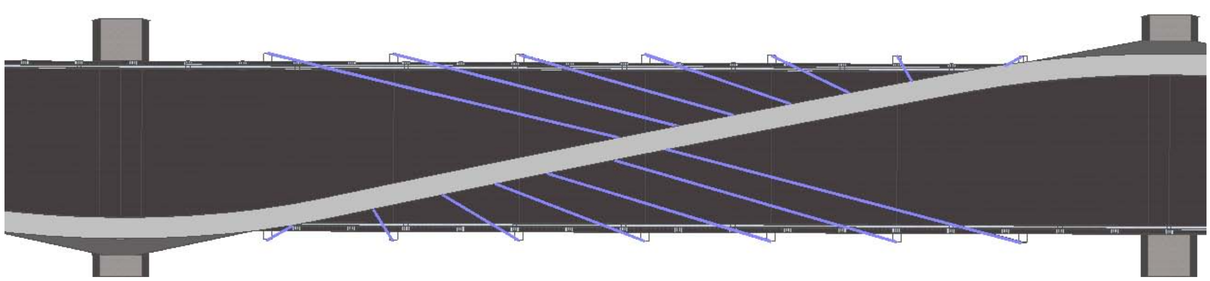


Figure 9: Plan View of an Arch Section (Middle Arch)

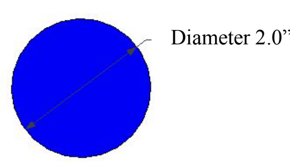
THE PIERS

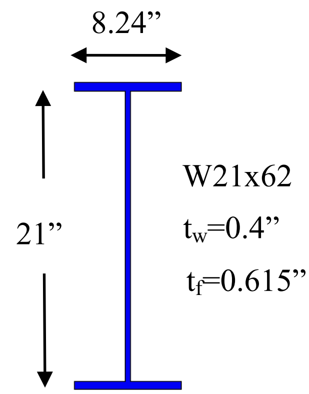
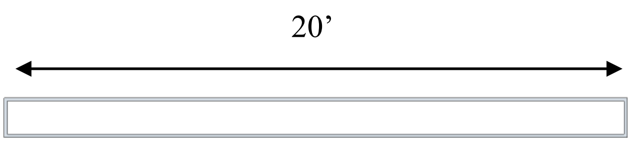
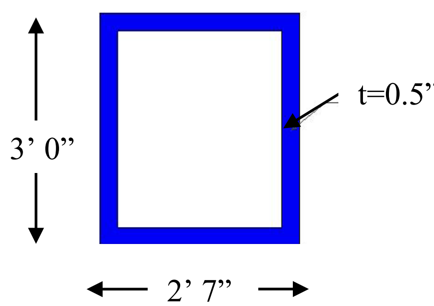
The original design of the bridge involved the legs of the arch meeting together at the bed of the river. This caused a lot of problems with the vertical and horizontal clearance which was needed for river traffic. The piers of the bridge were then designed so that the arches met at deck level and would also support the bridge deck at their locations, as shown in Figure 8. While solving the issues with clearances, this solution was very beneficial for ease of construction, where the bridge deck could be cantilevered off the pier, and for reducing twisting of the deck.

FINAL DESIGN

OVERVIEW

In order to size the various components of this bridge, the Group used both SAP2000 and hand calculations to achieve results that seemed reasonable, both in terms of cost and constructability. This process required multiple iterations and analyses, which will be described in detail later. With the aid of Microsoft Excel's Goal-Seek Analysis tool, the Group was able to try different size members that met the required moment and deflections limits. These new sizes were then incorporated into the SAP2000 model, analyzed, and revised when necessary. Using this process, the Group was able to find member sizes for every component of the bridge. The table below summarizes the final results.

Component	Dimensions
Cables	

Girder	
Deck Box	 Plate Thickness: 0.75 in Depth between plates: 7 in
Stiffeners	0.25 in thick every 2.5 ft (parallel to traffic) 0.25 in thick every 15 ft (perpendicular to traffic)
Arch	 *Tapers down to 21in x 21inx0.5 in at the supports

A detailed description of how these values were determined is outlined below.

CABLES

With each arch spanning a deck section 125ft long, the Group had to determine the number of cables desired per arch to hold up the deck. The number of cables was chosen rather arbitrarily, aesthetics being the main concern. It was feared too many cables would make the bridge look busy, while too few would require the cables to be larger than desired. With that said, the Group decided to go with 14 cables per arch, 7 on each side, as described previously in the description of the final design. An overhead view of the cable alignment can be seen in Figure 10.

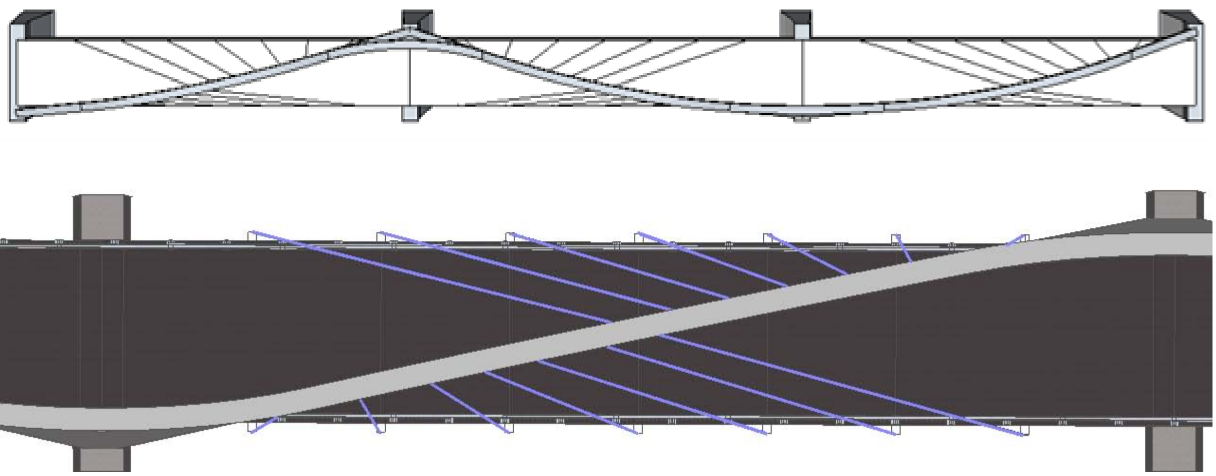


Figure 10: Cable Alignment

In order to size the cables, the Group had to determine the load that each cable would have to support. Each arch span supports a 125 ft long deck, composed of six 15ft sections and two 17.5ft sections. The deck sections and the positions of the cables on the deck can be seen in Figure 11 below. This was used to find the tributary area for each cable, which led to the minimum required area of the cables.

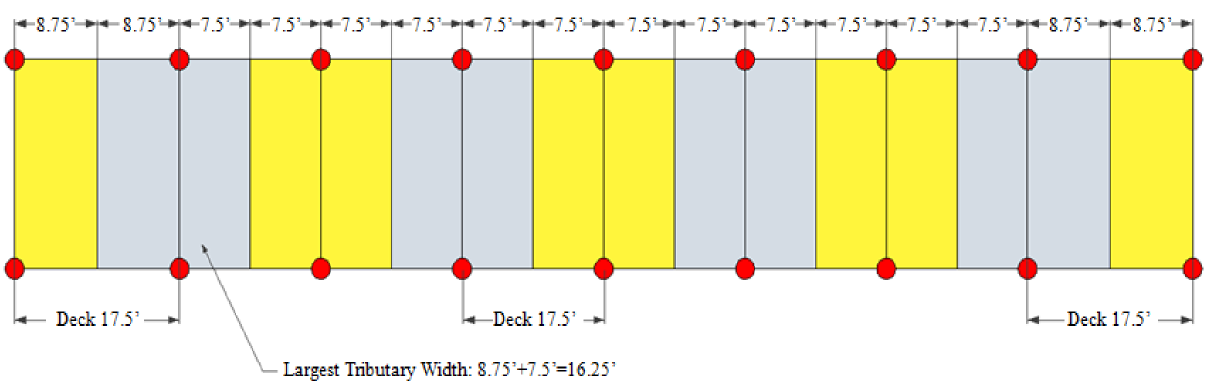


Figure 11: Tributary Area

As Figure 3 depicts, the maximum tributary width that any cable has to support is 16.25ft. With this width and the fact that the deck is 20 ft across, the total load can be determined. This calculation can be found in Appendix C, and shows a load of 57.58kips (composed of both the live and dead load the deck would experience with LRFD factors of 1.2 for dead load and 1.6 for live load). However, this is not the value needed to determine the size of the cable, for that the axial load needs to be calculated. The maximum axial load determined, was 79.45kips, and this calculation can also be found in Appendix C. Using a safety factor of 0.9 and a steel strength of 50ksi, the diameter of the cables was calculated to be 1.5inches using the following equation:

$$A = \frac{P}{\emptyset\sigma}$$

Because cables are not solid steel, as well as for additional safety, the Group decided to use 2.0 inch diameter cables.

It is important to add that for the first calculations completed for the cables, the Group assumed a deck cross-sectional area since it was unknown at the time. Therefore, the calculations were redone after the deck design was finalized to make sure the cables could withstand the actual dead load they would experience.

Girder

The deck is supported by girders which are hung from the cables as shown in Figure 12.

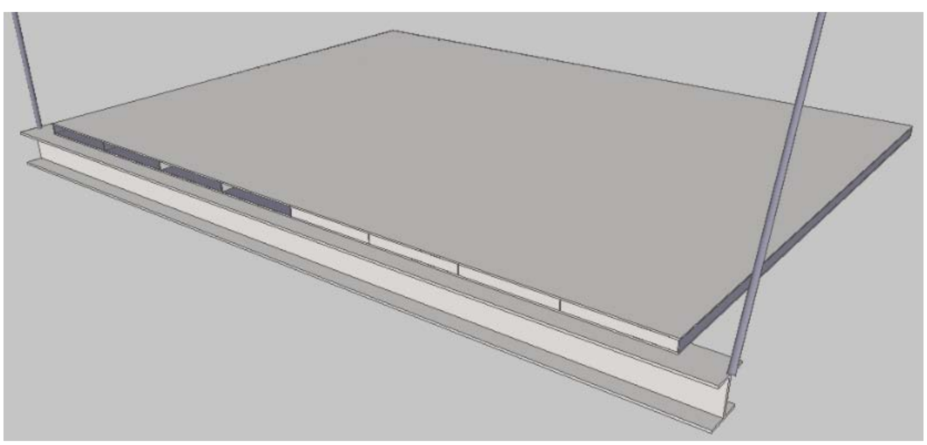


Figure 12: Girder

The cable-girder interface will consist of a couped girder and a ball-and-socket type bracket allowing bi-axial movement.

This correlates to a girder 17.5ft from the center of each foundation and then every 15ft thereafter. Similar to the method used for the cable sizing, the maximum tributary width was used to find the maximum load for the girders. The girder was analyzed as a simply supported beam with a uniformly distributed load composed of the live and dead load of the deck. A representation of this can be seen in Figure 13.

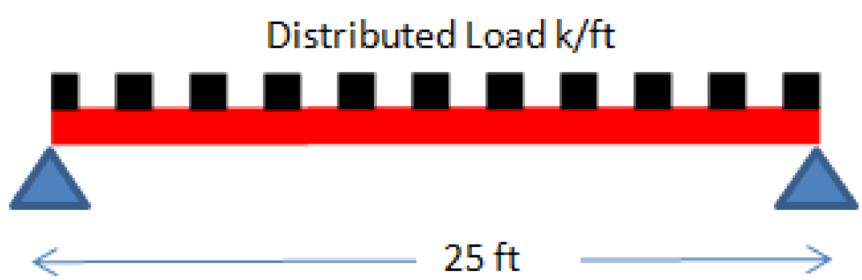


Figure 13: Girder Analysis

For a simply supported beam with distributed load, the maximum moment and maximum deflection both occur at the center of the beam. The values can be determined with the following equations:

$$M_{max} = \frac{1}{8}wL^2 \qquad \Delta_{max} = \frac{5wL^4}{384EI}$$

These maximum moment and deflection values had to be less than the allowable values, which are determined by the following equations:

$$M_{allow} = \frac{\sigma I}{y} \qquad \Delta_{allow} = \frac{L}{360}\ for\ LL\ only\ and\ \frac{L}{240}\ for\ Total\ Load$$

For cost efficiency, the Group decided to use a standard W-Section for the girder. In order to find the appropriate girder that would satisfy these requirements, the Group used trial and error. A W21x62 was chosen as it is the lightest W-Section that meets the deflection and strength requirements. The spreadsheet used to determine this can be seen in Appendix D. The girder reaches approximately 72% of its moment capacity and 90% of its allowable deflection. The final bridge design calls for 25 of these girders each 25ft long.

Deck Section

The deck section was one of the last components designed, since it required the most work. However, since the deck supplies most of the dead load experienced by the other bridge components, the Group had to verify all other components could withstand the actual dead load the deck would apply. The chosen shape for the deck was a hollow box with evenly spaced stiffeners. Figure 14 below shows what the cross section will look like.

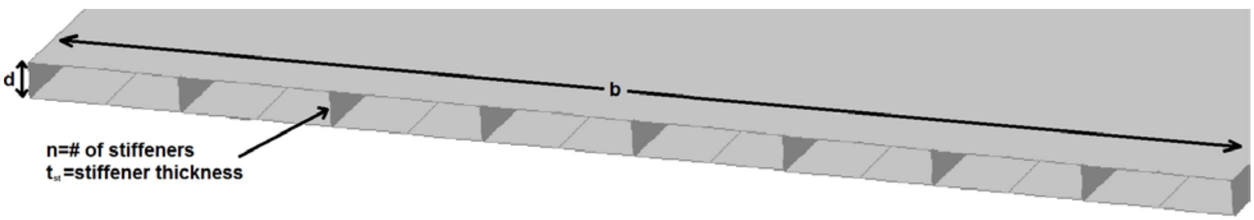


Figure 14: Deck Section

Like the girder, the deck section is modeled as a simply supported beam, so that the maximum moment and deflection occur at mid-span. The loads that the Group used for the hand calculations were 0.15ksf for pedestrian loads and 0.49kcf as the dead load of the steel. The moment and deflection relationship for a simply supported beam can be seen below in Figure 15.

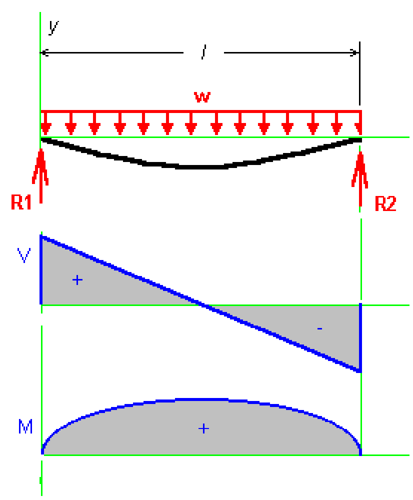


Figure 15: Simply Supported Beam w/ Distributed Load

Like the girder, the following calculations were used for the deck's initial analysis:

$$M_{max} = \frac{1}{8}wL^2 \qquad \Delta_{max} = \frac{5wL^4}{384EI} \qquad M_{allow} = \frac{\sigma I}{y}$$

$$\Delta_{allow} = \frac{L}{360}\ for\ LL\ only\ and\ \frac{L}{240}\ for\ Total\ Load$$

These deflection and moment criteria were checked for both directions: along the 15ft length and the 20ft width. For the 20 ft width, deflections were calculated for the deck spans between stiffeners.

The length of the deck section was set at 15ft, since this was the span between girders: the simple supports for the deck. Additionally, the dimension "b" in Figure 14 was set at 20ft: the width of the deck. Therefore, the only variables the Group modified were the depth, "d", the thickness of the box section, "t", and the thickness and number of stiffeners, "t_{st}" and "n" respectively. From this point, the Group decided to set the box thickness at 0.5inches and the inside depth to be 7inches. This left the number of stiffeners and their thicknesses as the only variables. Trying different values, the Group decided to limit the thickness of the stiffeners to 0.25inches and solved for the required number of stiffeners. Comparing the different constraints, it was determined that the controlling factor was the live load deflection between the stiffeners. Therefore, the group applied the Goal Seek tool so that this maximum live load deflection equaled the allowable by changing the number of stiffeners. This method resulted in 7.13 stiffeners placed 2.805ft apart. Since the number of stiffeners must be discrete, and the Group preferred even spacing for ease of construction, it was decided to use 8 stiffeners placed 2.5ft apart. The calculations showing the Goal Seek and the check of 8 stiffeners can be found in Appendix E.

Once the box section was designed, the Group also had to check for lateral torsional buckling. These calculations can also be found in Appendix E. It turned out that lateral torsional buckling was not a controlling factor and therefore the values determined from the design process described above were not affected.

The last check required for the deck section was the bolt connections between deck segments. As described earlier, the deck is comprised of 15ft and 17.5ft sections, and these sections will be attached to one another with bolts. The Group decided to use 0.75in diameter bolts. Calculations were carried out for both 84ksi and 68ksi bolts. The following equations were used:

$$R_n = F_n A_b \qquad N_{bolts} = \frac{Max\ Shear}{R_n}$$

The number of bolts, N_{bolts}, was determined using the maximum axial force in the deck. This was obtained by dividing SAP2000's maximum moment in the deck by the depth. In the above equation, F_n equals the shear capacity, which is the 84ksi or 68ksi, of the bolt. The detailed calculations can be found in Appendix F. Using 84ksi bolts requires 12 bolts placed 1.5ft apart along the 20ft width of the deck. Likewise, 68ksi bolts, requires 15 bolts placed 1.3ft apart.

Connection plates between deck sections were also designed, and checked for shear, yielding, and rupture capacities. Using SAP2000's maximum shear output and the axial force described for the bolt calculations, these capacities were checked with the following equations.

$$Shear: V = 0.6\phi F_y wt \qquad Yileding: P = \phi F_y wt \qquad Rupture: P = \phi F_u A_g U$$

The Group determined that these connection plates would be governed by yielding. The calculations can also be found in Appendix F. While the required thickness for these plates was calculated to be only 0.057in, the Group decided that a more practical value would be 0.5in. Therefore, above and below each girder connection (where the bolts connect) there will be 0.5in thick Grade 36 steel plates.

ARCH

Previously, it was described how the shape of the arch was determined based on minimizing moment. However, zero moment was not achieved and there will still be minimal moments observed in the arch. Therefore, the cross section of the arch needs to be designed accordingly. The moments in the arch when the deck is fully loaded can be seen below in Figure 16.

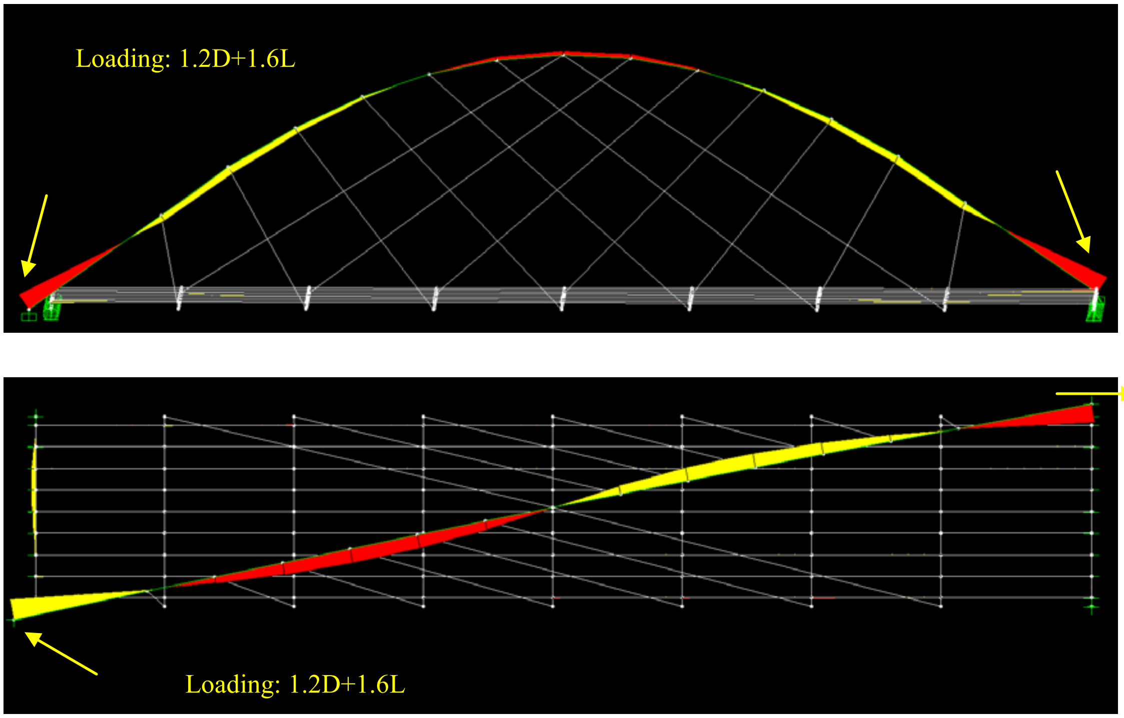


Figure 16: Fully Loaded Moments

The images in Figure 17 show the arches with the maximum moments they would experience.

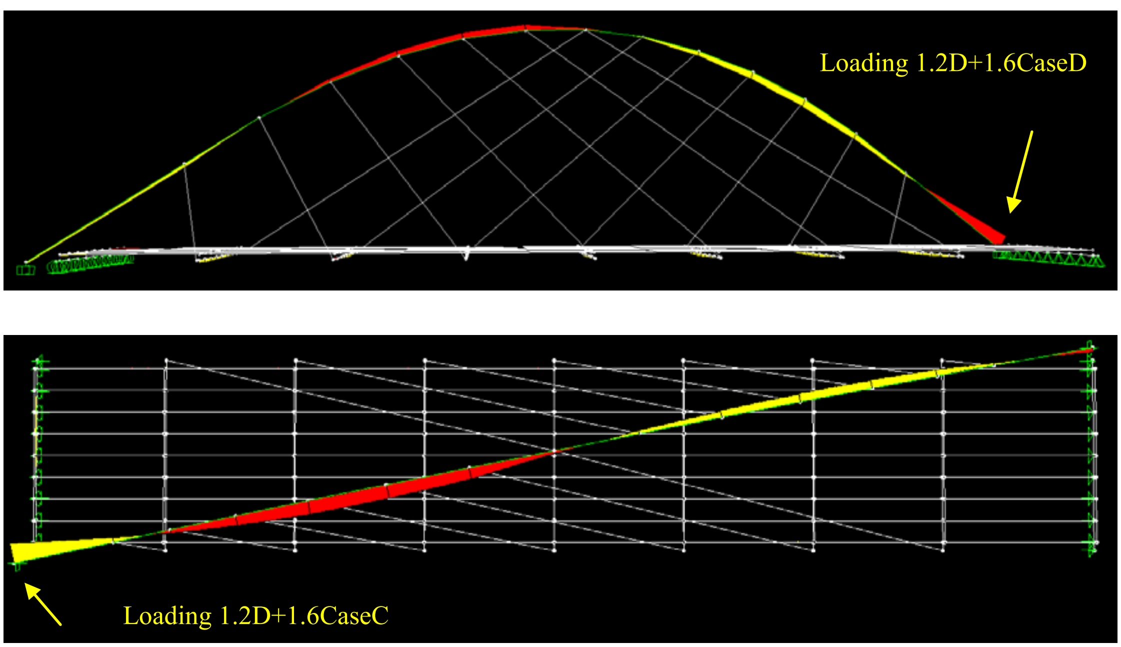


Figure 17: Maximum Moments

It is important to note that these moment values from SAP2000 are dependent on the component cross sections. Therefore, this had to be checked every time a change was made to a component. The figures and values above are for the final dimensions determined by the Group. Appendix G shows the calculations that led to the final arch section. The arch is a hollow box section with stiffeners placed to keep the plates of the box from buckling. While it was not designed, these stiffeners will most likely be solid steel plates.

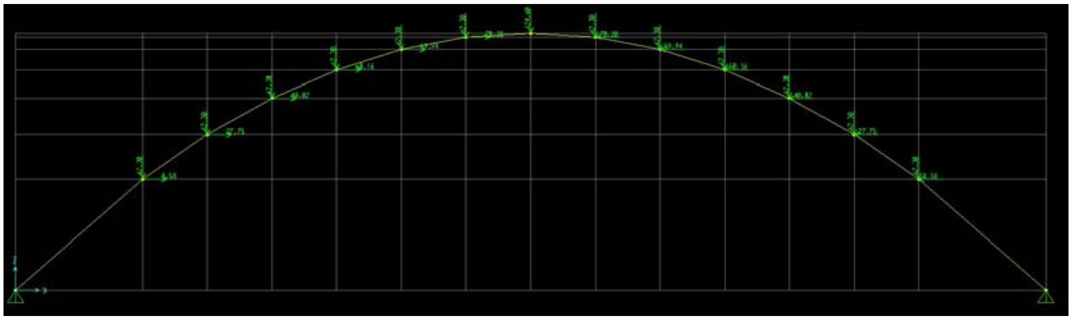
The box cross section was designed mostly by trial and error, going back and forth between Excel and SAP2000. The design group tried to keep the arch as square as possible, but because of the torque created by the cables, the arch ended up being longer in one direction than the other. After defining the dimensions of the arch cross section, the moment was checked with the following equation:

$$M_{allow} = \frac{\sigma I}{y}$$

This value was then compared with the SAP2000 outputs mentioned above to check for failure. This process took several iterations until the final dimensions listed in the earlier table were achieved.

MODELING AND ANALYSIS IN SAP2000

SAP2000 was utilized as the main modeling and analysis software. In order to maximize the programs potential, the group created several different models for different analysis purposes. Initially, a very simple model was used, which involved a 2-D arch modeled with point loads at the cable connections. Using geometry, the Group determined the X, Y, and Z components of the forces that the cable would transfer to the arch. The loads applied to the simple model can be seen in Figure 18 below.



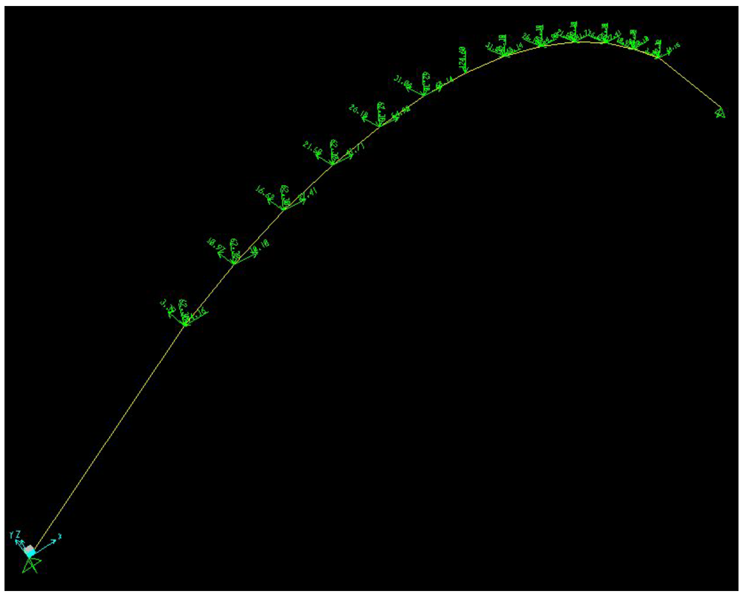


Figure 18: Simple Model

This simple model was used primarily to determine the best spacing of the cables and how the arch would react. Appendix H includes a snapshot of the Excel file that was used to transfer the gravity load per cable into the components inputted into the SAP2000 model.

Once the cable spacing was finalized, the Group was able to create a more detailed model for further analysis. Two separate models were created, and both included all components of the bridge: girders, cables, deck section, and deck stiffeners. One model was of the complete bridge, with all three arch spans (Figure 19) while the other was just of a single arch span (Figure 20). Most analysis was conducted with just the single arch model since each arch span is identical.

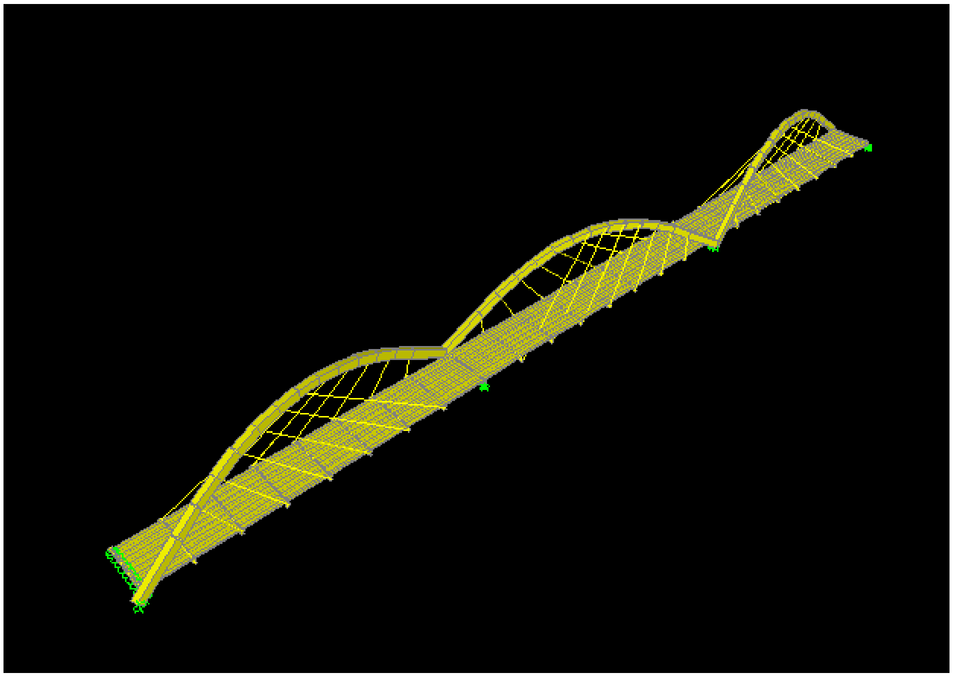


Figure 19: Complete Model

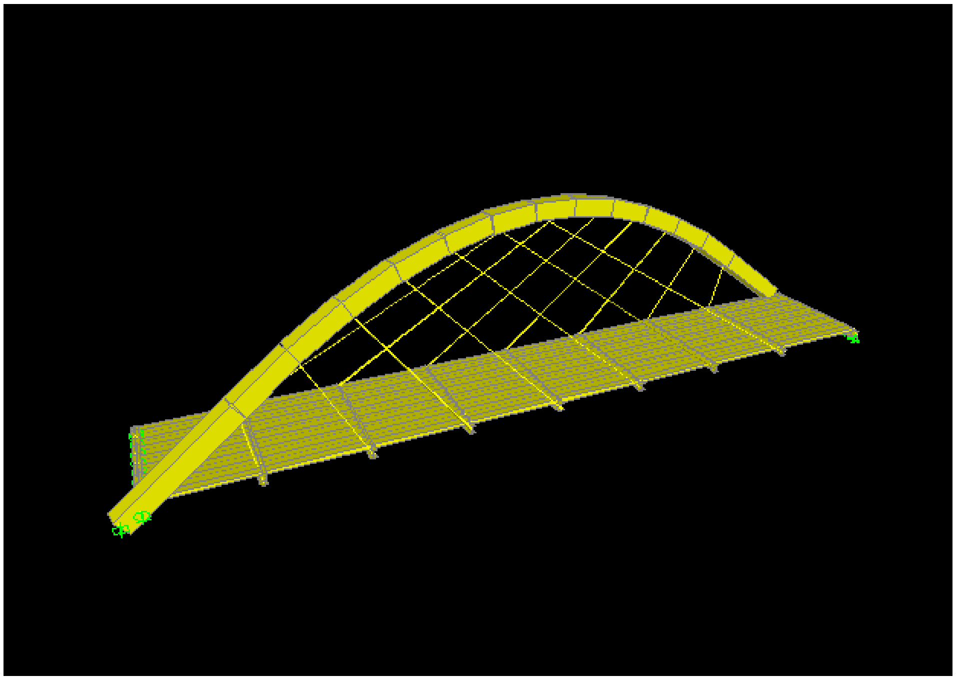


Figure 20: Single Arch Model

Figure 21 depicts the coordinate system that is referred to through the report.

X: Along deck, parallel to direction of traffic

Y: Perpendicular to direction of traffic

Z: vertical

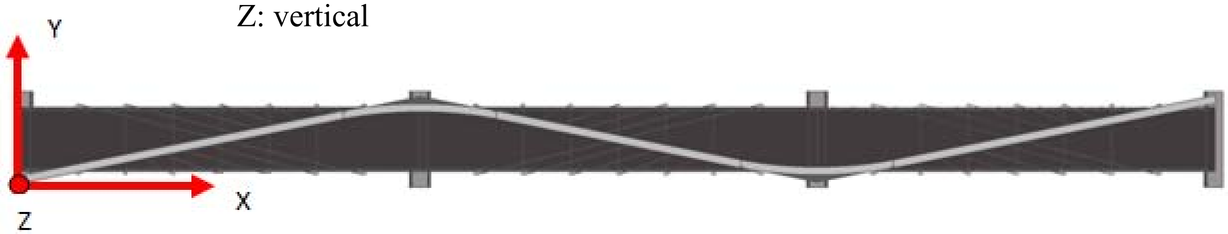


Figure 21: Coordinate System

Throughout the design process, the design group went through a series of analyses to make sure that the bridge could withstand any force that it could possibly experience. This list of analyses includes:

1. Gravity Loads
2. Lateral Loads
3. Non-uniform Loads
4. Modal Analysis
5. Seismic Analysis
6. Moving Load Analysis

All of these analyses were conducted with the single arch model (a set of non-uniform loads was also analyzed on the full bridge model). While these analyses were carried out, the Group had to continuously go back and check that the previously designed members still work. This involved checking that the moments, axial forces, and shear forces that members experienced fit within the limits described earlier. A detailed description of the process and results of each of the analysis follows below.

GRAVITY LOADS

The gravity load analysis was the simplest of all the analyses listed above. It consisted of applying the live and dead loads to the members and making sure that the members didn't fail. This analysis was discussed previously in the "sizing member" section. The loads used for each member were the same:

Pedestrian Live Load: 150 pounds per square foot

Vehicle Live Load: AASHTO HL-83 Design Tandem consisting of a two axle vehicle with 25 kips on each axle spaced by 4 ft

Self-Weight Dead Load: Steel density of 0.49 kips per cubic foot

For some of the members, the self-weight could be cambered out, but for completeness the group designed the members so that this would not be required. Instead, all members were designed to withstand both the live and dead load completely. For hand calculations, the live and dead loads were determined for each component, while in the SAP2000 model all loads were applied to the deck surface, which is where they apply in real life. Figure 22 below shows what the model looks like with the loads applied.

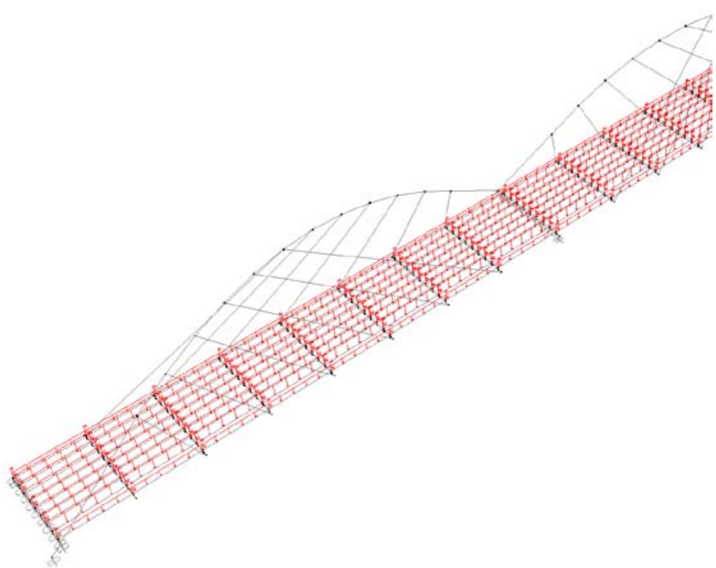


Figure 22: Gravity Loads

Lateral Loads

The lateral loads analyzed include wind and seismic. The focus here is on wind loads, seismic analysis will be described in more detail later. Using ASCE 7-10 guidelines, the Group designed for 140mph wind loads. The calculations for this can be found in Appendix I. The calculated wind force was applied only to the deck of the bridge. Figure 23 below shows how the wind loads were applied to the SAP2000 model.

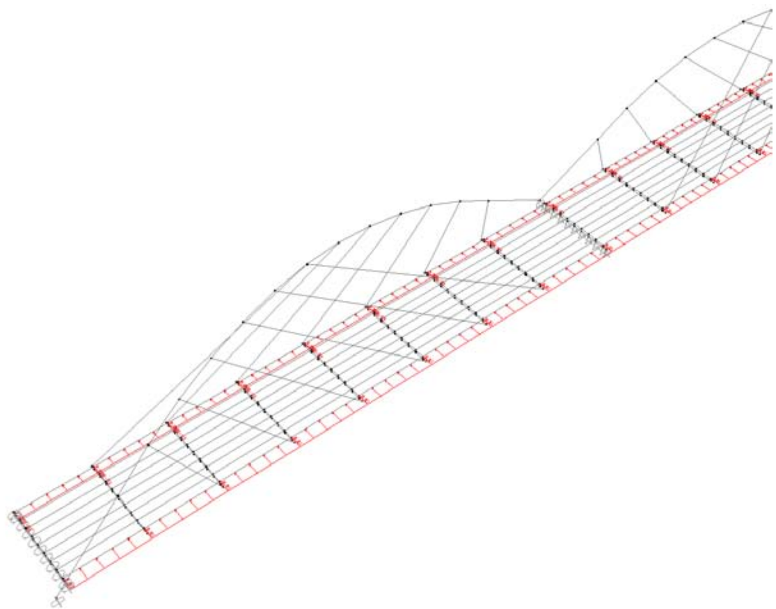


Figure 23: Wind Loads

The windward force was calculated to be 24.84psf and the leeward -15.52psf. As part of the analysis, the deflection and moment in the deck were checked with the limiting factors.

Non-Uniform Loads

Non-uniform loading of the bridge was an important check that the Group had to conduct to ensure that the bridge would not fail under different loading patterns. Pedestrians could gather on one spot of the bridge and produce uneven loadings that have substantial effects because of the crossing arches. Such a scenario could exist if people gather to watch a boat race or fireworks in the river. Therefore, the Group came up with multiple scenarios that could exist for both the single arch model and the complete model. Figure 24 below depicts these scenarios:

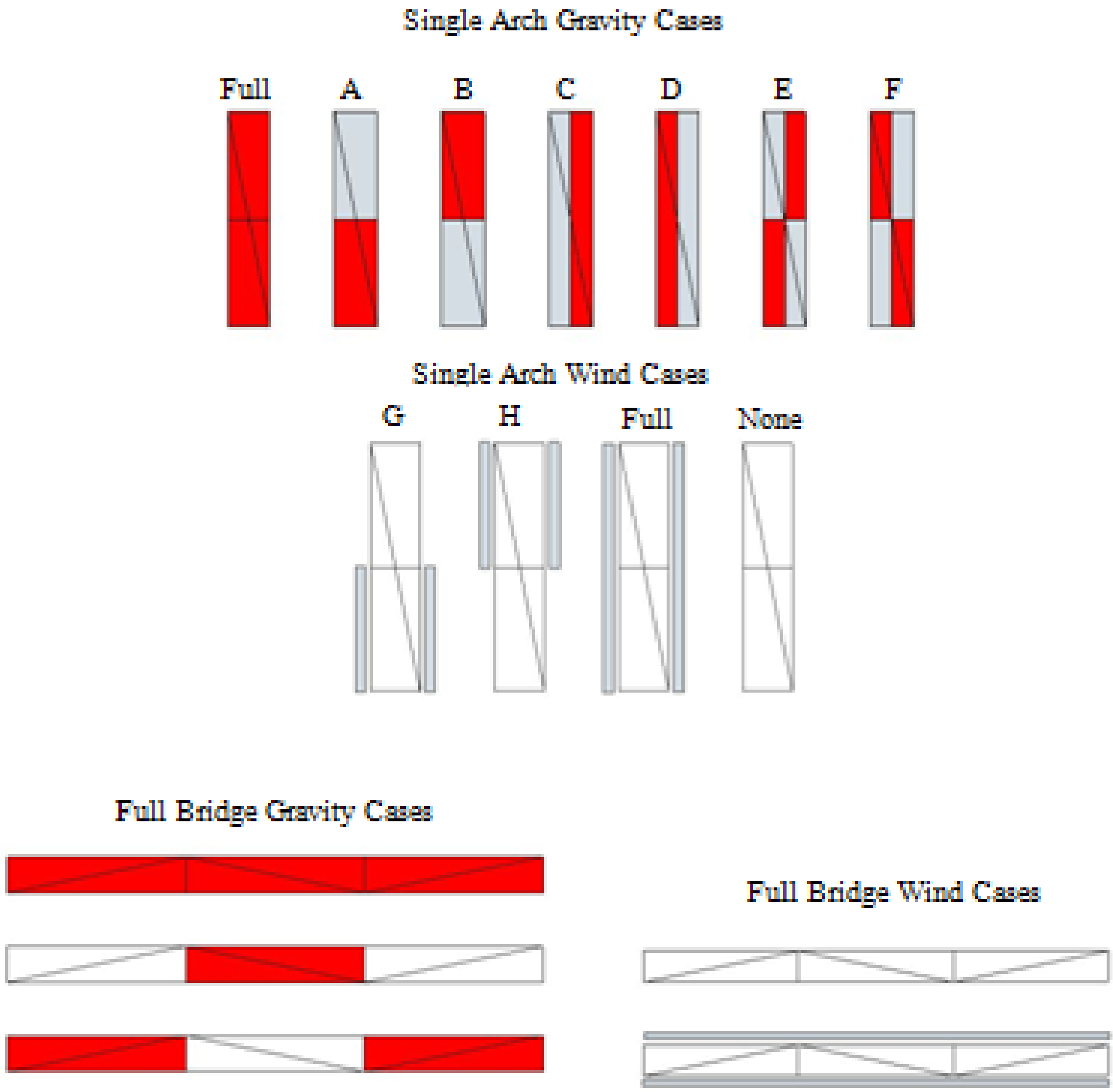


Figure 24: Non-Uniform Loading

Different combinations were made between the gravity cases and the wind cases. This amounted to 26 cases for the single arch case and 6 for the full bridge. After analyzing SAP2000 results, it was de termined t hat t he c ontrolling l oad c ase w as G ravity C ase B with no w ind l oading (1.2D+1.6CaseB). Appendix J contains the deflections for each combination checked.

MODAL ANALYSIS

Once the model was built with all materials and cross sections specified, SAP2000 ran the modal analysis and outputted the various mode shapes and their corresponding frequencies and periods.

The Group wanted to analyze all modes up to a frequency of 20 cyc/sec, which amounted to 20 modes for the single arch model, and 60 modes for the full model. Below is a table showing the frequencies and periods for the 20 modes of the single arch model.

Mode #	Frequency	Period
	Cyc/sec	Sec
1	2.557	0.391
2	4.180	0.239
3	4.715	0.212
4	4.770	0.210
5	6.294	0.159
6	6.527	0.153
7	8.905	0.112
8	9.361	0.107
9	9.668	0.103
10	10.949	0.091
11	12.153	0.082
12	14.051	0.071
13	15.361	0.065
14	16.260	0.061
15	16.884	0.059
16	17.938	0.056
17	18.022	0.055
18	19.384	0.052
19	21.106	0.047
20	21.723	0.046

As seen in the table, the fundamental mode has a frequency of 2.557 cyc/sec and period of 0.39sec. A similar process was done for the complete model. This resulted in a fundamental frequency of 2.499 cyc/sec and a fundamental period of 0.400 seconds, which is very similar to the values of the single model analysis. The complete model has three similar modes for each mode type, one for each arch. Therefore, modes 1, 2, and 3 all have frequencies around 3 cyc/sec and periods around 0.4sec. This also explains why 60 modes were needed to achieve a frequency of 20 cyc/sec while only 20 were needed for the single arch.

In addition to frequency and period values, SAP2000 also provided participation factors for each mode and direction. This data was then used to conduct the seismic analysis.

SEISMIC ANALYSIS

Two separate seismic analyses were carried out during the design process. The first analysis made use of the modal participation factors mentioned before and data collected from USGS. The second analysis was done entirely in SAP2000 by running a time history analysis of a recorded earthquake from the SAP2000 library.

The United States Geological Survey (USGS) provides earthquake data for every region in the US. In addition, they provide seismic design maps for engineers that are applicable to both buildings and bridges. The Anchorage Group made use of the free software USGS provides that is called “AASHTO Seismic Design Parameters”. This program provided graphs of peak ground acceleration and spectral acceleration for an earthquake with 7% probability of exceedance in 75 years for the Boston area. The program interface and output can be seen below in Figure 25.

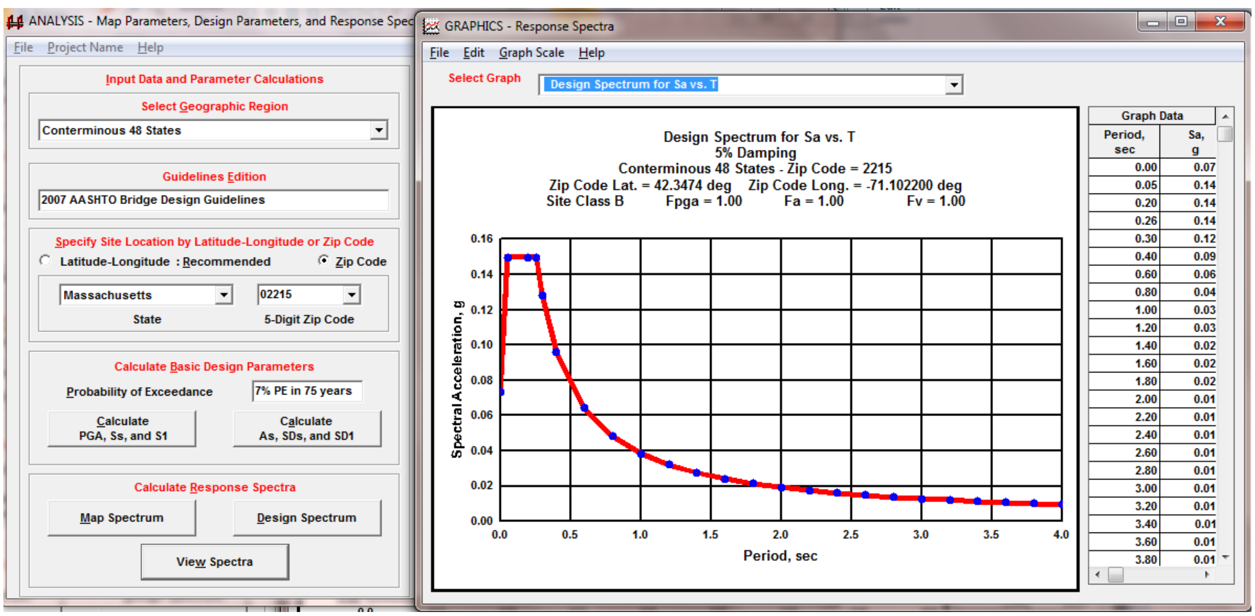


Figure 25: USGS Data

The USGS program also provided spectral displacement values which were then made into a graph. With this graph, the Group was able to find a relationship between period, T, and spectral displacement, S_d:

$$S_d[in] = 0.3771T - 0.0037$$

Using this equation, the displacement for each mode was determined using the following equation:

$$U = \Gamma S_d \phi$$

$$\Gamma = \text{Participation Factor} \quad S_d Spectral\ Displacement \quad \phi = Shape\ Factor$$

Appendix K includes the calculations for each mode, which show that the fundamental mode suffers the most deflection: U_x=0.186 in, U_y=1.803 in, and U_z=1.365 in.

This analysis showed that the bridge would not experience any significant damage from this earthquake. In fact, all deflections are well below the allowable limits.

The second seismic analysis made use of SAP2000 library of earthquake data. Because the previous seismic analysis showed that the Y-direction would experience the most deflection, the Group selected the Y-direction values for earthquake data, specifically the Santa Monica City Hall Grounds earthquake. Figure 26 below shows the time history of this earthquake.

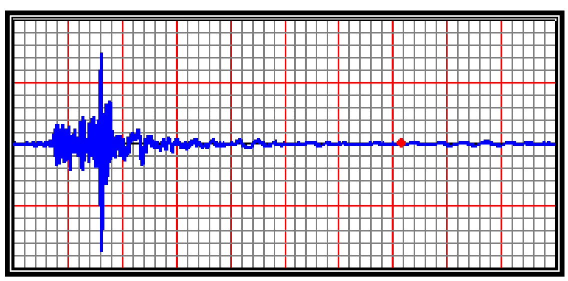


Figure 26: SAP2000 Time History Earthquake

After running the earthquake load case, the Group analyzed the displacements of the various members. Once again, the bridge did not experience any major deflections. The maximum deflections were: U_x=0.26 in, U_y=2.31 in, and U_z=1.09 in. It is important to note that these values differ from the previous seismic analysis because the data is from a different earthquake.

CONSTRUCTION SEQUENCE

The basic method of construction for this bridge will be to fabricate components of the bridge off site and then deliver them to the site just in time for assembly and installation.

Each arch will be fabricated in three sections: two identical "lower" sections and one "upper" arch. Dimensions of the sections are: lower – 49.5ftx3.5ft, upper – 42.5ftx5.2ft. These sections will then be delivered to the site via truck. Two sections will fit on one truck. After delivery, the arches will be assembled and lifted into place by crane (one pick per arch, with 3 lifting points). The total weight of one arch is 15.4 tons.

The deck will also be fabricated off site in 24 sections; the largest sections being 17.5ftx20ft. These sections will be bolted together on site and lifted into place. Bridge cables will then be attached and the crane removed, allowing the remaining bridge cables to be attached.

This is the basic method for construction. A detailed construction sequence follows:

While the bridge components are being fabricated in the shop, foundation construction will begin on-site. A barge will be delivered to the site, a barge crane erected and cofferdams will be installed to enable the construction of the bridge piers. The banks of the river will also be prepped for foundation construction.

After the cofferdams are complete and dewatered, form work and rebar will be installed and concrete poured. The concrete will be allowed to cure for at least 14 days before proceeding with the next portion of work. At this point, formwork will be removed and prep for arch installation will be conducted. Figure 27 shows a rendering at foundation completion.

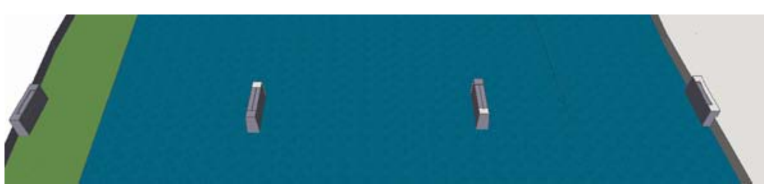


Figure 27: Foundations Complete

Upon completion of the foundations, the arches will be lifted into place by crane; exterior arches first, followed by the interior arch. Figure 28 shows a rendering after arch installation.

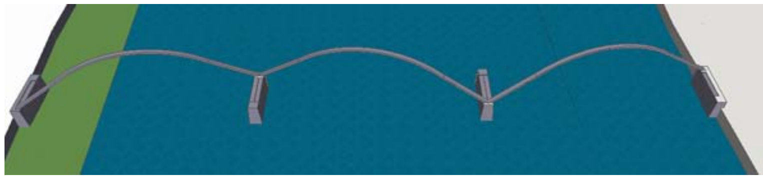


Figure 28: Arches installed

The next step will be to install deck sections on the piers. These portions of the deck will be assembled on shore and each consists of 4 deck sections and weighs 32.5 tons. The assemblies will be lifted into place with the crane, attached to the piers and end cables and then the crane will release them. Figure 29 indicates the crane lifting points, in red, and the bridge cable connection points, in green. The cables will be connected before the crane is released and the remaining cable connections will be made after.

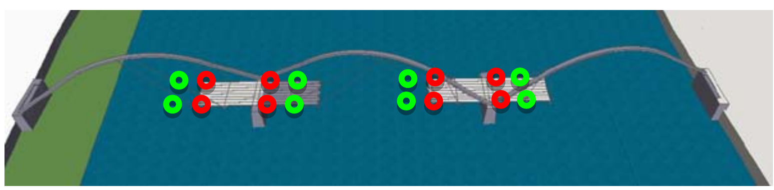


Figure 29: Interior deck sections installed

Next, deck assemblies will be installed at the shore foundations. These assemblies each consist of 2 deck sections and weigh 16.7 tons. They will be lifted into place with the crane and connected in a similar manner as discussed above. Figure 30 shows a rendering after exterior deck assembly installation.

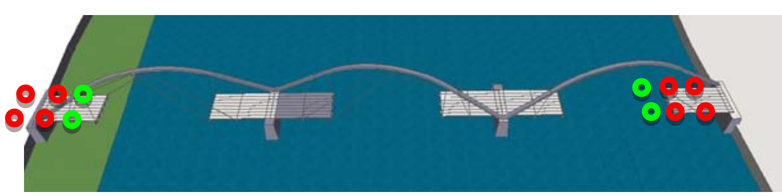


Figure 30: Exterior Deck Sections Installed

Once all four "cantilevered" deck sections are complete, the remaining "mid-span" assemblies will be installed. Each of these assemblies consists of 4 deck sections and weighs 28.8 tons. Because the arches will interfere with the crane and it would be difficult to remove a spreader bar from among the cables once the deck was installed, the Group recommends these assemblies be installed by lifting from below. Figure 31 shows a rendering after mid-span deck assembly installation.

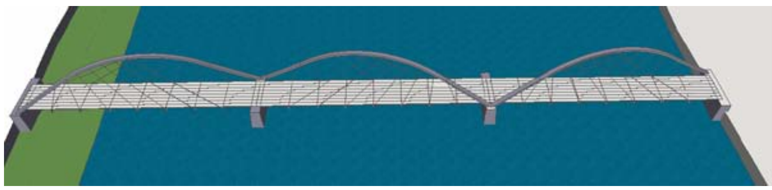


Figure 31: Mid-Span Deck Sections Installed

At this point the structural components of the bridge are all complete.

Next, the "swooping" portions of the arches will be installed. These are non-structural and for aesthetics only. These are the same dimensions as the upper portion of the arches. This dimension match, the color scheme of the bridge, and the fact that the structural arches taper from the last cable connection to the supports produces a fluid feel to the arches. Figure 32 shows a rendering after aesthetic arch installation.

Figure 32: Swooping Arch Sections Installed

The final step is installation of guardrails. Figure 33 shows a rendering of the completed bridge.

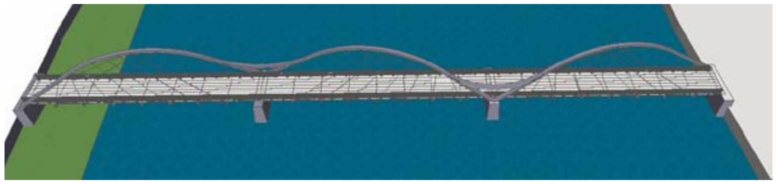


Figure 33: Completed Bridge

SCHEDULE

The Group developed two separate schedules and corresponding cost estimates. Both schedules make the following assumptions:

- 5 day work weeks, 10 hour work days
- 50% efficiency gain halfway through fabrication
- No delays (weather, unforeseen conditions, etc.)

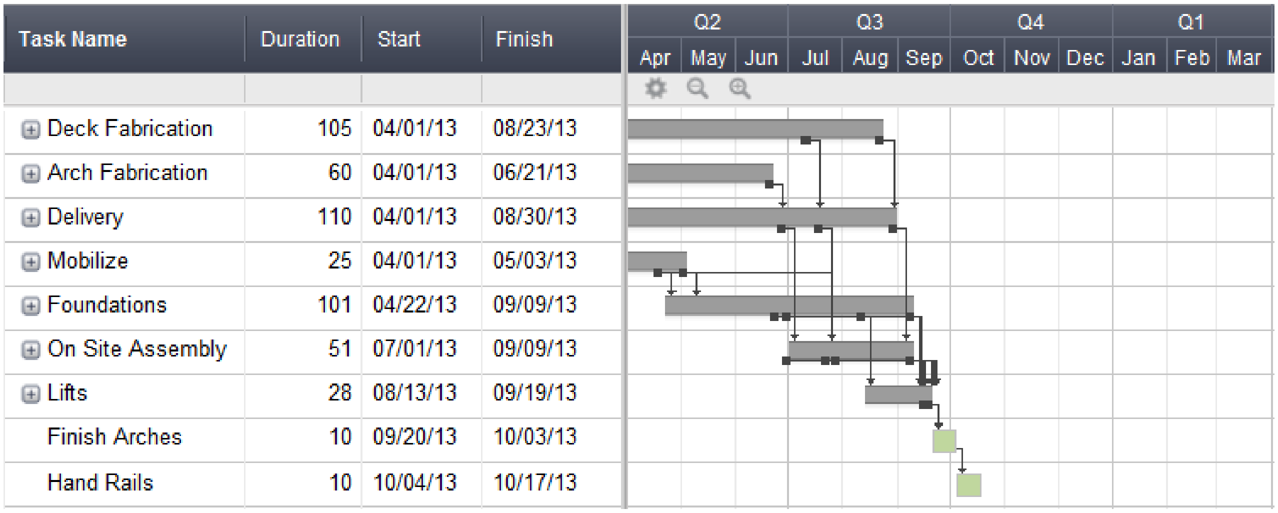
Detailed Gantt charts for both schedules showing activity duration, predecessor-successor relationships, lag, etc. can be found in Appendix L.

SCHEDULE 1

The first schedule assumes that multiple crews in each discipline are available, enabling simultaneous work. For example, there will be three jigs made for arch sections and a separate crew will work on each; completing 1 arch in 1 cycle of using these jigs.

This schedule also assumes on site assembly is dealt similarly; deck sections will be connected 2 at a time, etc.

This schedule is 143 work days or 6 ½ months long. Assuming an April 1st start, construction completes October 17th. Figure 34 shows the basic Gantt chart for Schedule 1.



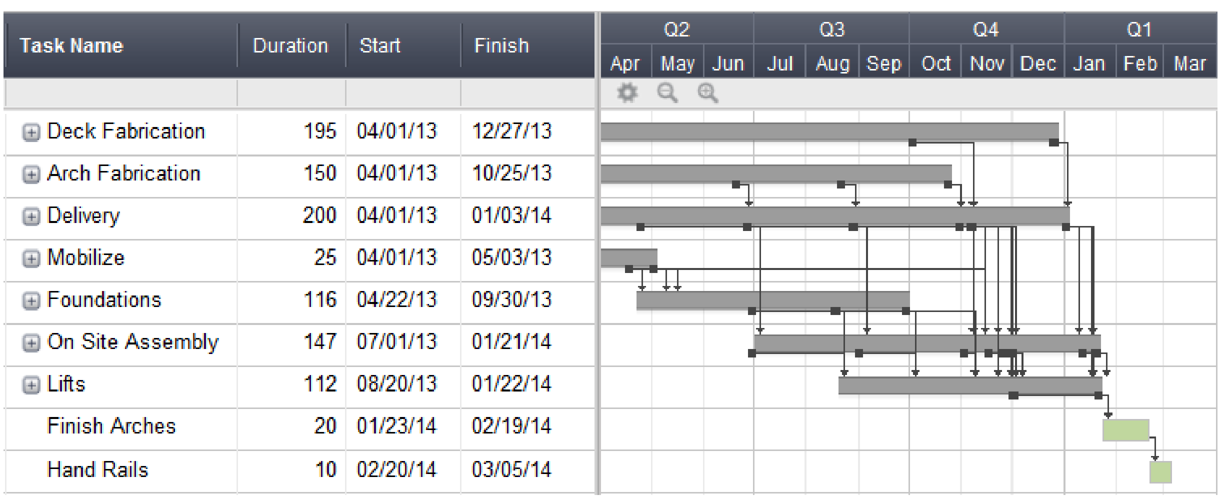
Task Name	Duration	Start	Finish
Deck Fabrication	105	04/01/13	08/23/13
Arch Fabrication	60	04/01/13	06/21/13
Delivery	110	04/01/13	08/30/13
Mobilize	25	04/01/13	05/03/13
Foundations	101	04/22/13	09/09/13
On Site Assembly	51	07/01/13	09/09/13
Lifts	28	08/13/13	09/19/13
Finish Arches	10	09/20/13	10/03/13
Hand Rails	10	10/04/13	10/17/13

Figure 34: Schedule 1

SCHEDULE 2

The second schedule assumes one crew in each discipline. In other words, it will take 3 times as long to fabricate each arch, 2 times as long to assemble each deck section on site, etc.

This schedule is 242 work days or 11 months long. Assuming an April 1st start, construction completes March 5th. Figure 35 shows the basic Gantt chart for schedule 2.



Task Name	Duration	Start	Finish
Deck Fabrication	195	04/01/13	12/27/13
Arch Fabrication	150	04/01/13	10/25/13
Delivery	200	04/01/13	01/03/14
Mobilize	25	04/01/13	05/03/13
Foundations	116	04/22/13	09/30/13
On Site Assembly	147	07/01/13	01/21/14
Lifts	112	08/20/13	01/22/14
Finish Arches	20	01/23/14	02/19/14
Hand Rails	10	02/20/14	03/05/14

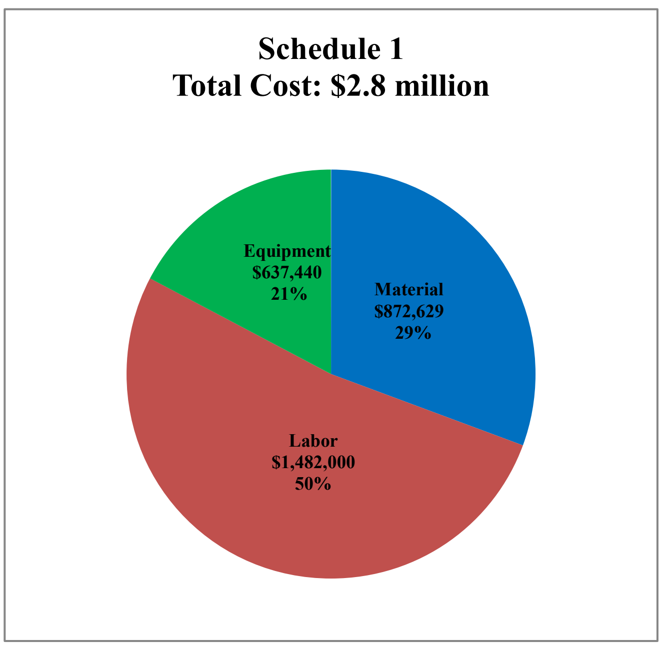
Figure 35: Schedule 2

COST

The following assumptions were made in producing both cost estimates:

- Labor - $75/man-hour
- Steel - $1,136/ton
- Concrete - $250/cubic yard
- Barge - $1,000/day
- Crane - $1,800/day

The difference in cost between Schedule 1 and Schedule 2 is in construction equipment rental time. Both schedules require the same amount of man-hours, so labor cost is the same. Material cost is also the same, since the product isn't changing. The cost estimate shows that the sequential construction costs $200,000 more. A detailed breakdown of the cost can be found in Appendix M. Pie charts with a breakdown of the cost components for each schedule can be seen in Figure 36, along with the total project cost for each schedule.



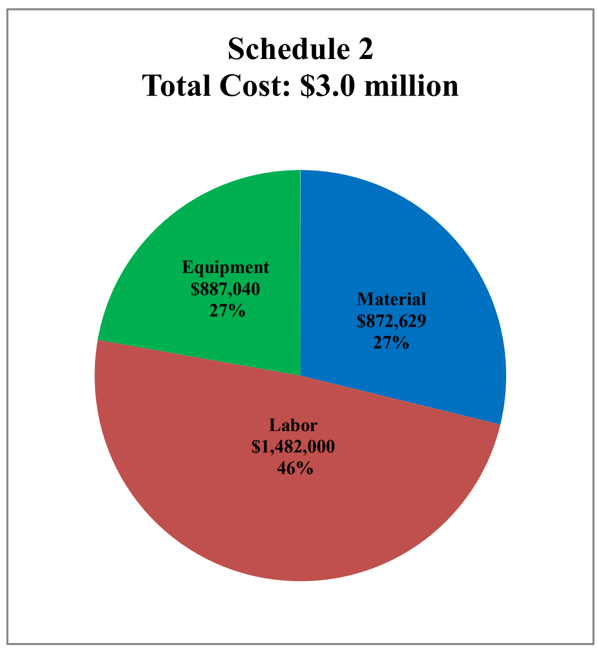


Figure 36: Cost Breakdown

PROPOSED ROAD BYPASS

OVERVIEW

The solution the Group developed for the road crossing is an underpass. The primary objective of this concept is to move pedestrian and cyclists from one side of the intersection to the other without obstructing vehicular traffic and without having to wait for traffic lights. In addition to the aforementioned objective, the group also sought a solution that would not only be slender, with a low profile almost invisible from the road, but also provide limited obstruction to water traffic in the Charles River, especially for large events such as the Head of The Charles. In evaluating the functionality of the final design against these requirements, the Group is confident that this solution adequately addresses each one.

The underpass reroutes pedestrians and cyclists under the outer arches of both existing bridges as shown in Figure 37.

Figure 37: Aerial View of Underpasses (shown in red)

It will be supported by steel columns near the shore and suspended from the existing bridge by a cable system underneath the arch. An overview of one of the underpasses can be seen in Figure 38. In order to meet regulations that require a minimum height clearance of 10ft for the pathway underneath the arches, the underpass had to be moved to the center of the outer arches, which leaves just over 25 ft of waterway for river traffic. The Group realized that this could be an impediment to water traffic during events like the Head of the Charles. To rectify this, the final design includes a cable and hinge system underneath the existing arches that will allow the underpass to be lifted out of the way, to expand the waterway for river traffic. This can be seen in Figure 39.

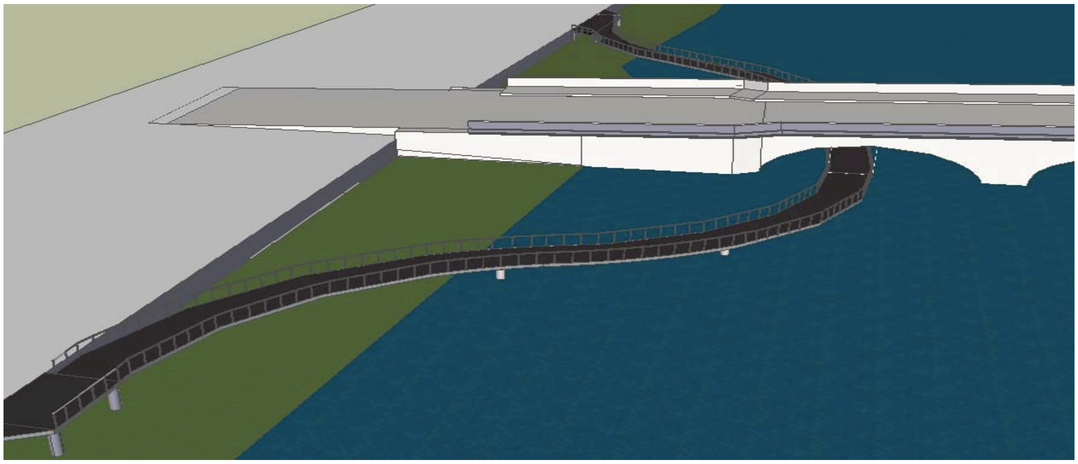


Figure 38: Underpass final design with support system

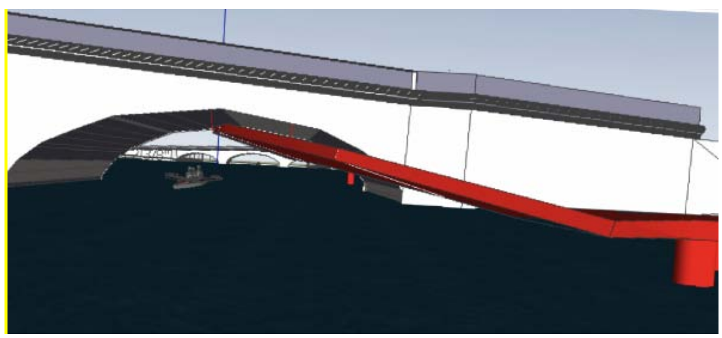


Figure 39: Lifted Underpass

To ensure structural stability and guarantee structural integrity of the structure, gravity dead and live loads were applied, and modal and seismic analyses were carried out.

GRAVITY LOAD ANALYSIS

Gravity load analysis was performed by the Group in order to verify that the dimensions of the structural system were sufficient to satisfy the deflection limit of L/360 and bending moment capacity of the structure. The governing combination was 1.2D+1.6L. Calculations showed the magnitude of the live load was four times greater than that of the dead load. The live load was determined by taking the minimum required uniform loading of 150psf for pedestrian traffic and multiplying by the deck width of 12ft, which yielded a linear loading of 2.88kips/ft. The linear loading for dead load was determined by taking the density of steel, 0.484kip/ft^3, and multiplying by the cross sectional area of the section. Additionally, the Group assumed the dead load deflection could be cambered out and only considered the live load deflection. The governing load case for the underpass, including the span under the existing bridge, occurs when every other bay is loaded uniformly, creating a maximum moment in the structure of 2935 kips/ft.

Deflection governs this design: the deflection limit was determined to be 0.308ft by taking the length of the longest unsupported span and dividing by 360. A rendering of the underpass deflection can be seen in Figure 40.

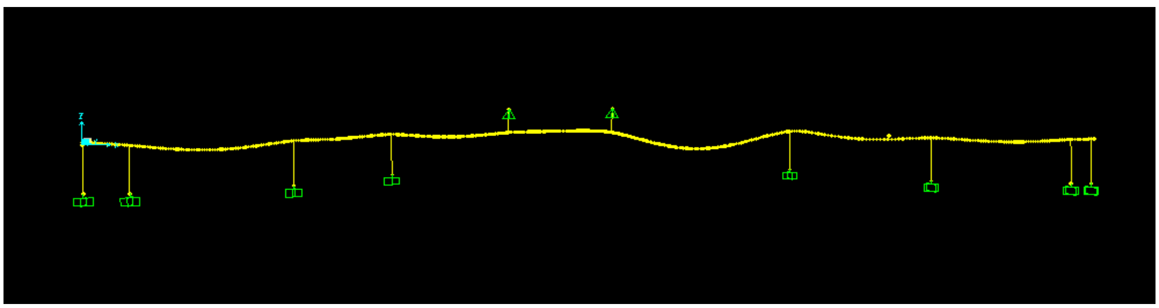


Figure 40: Deflection Diagram of the Structure

However, to meet this deflection limit, cross sectional dimensions for a box girder type bridge needed to be 12ft wide and 1.5ft deep. The linear dead load for this section was 0.65 kips/ft and the moment capacity was 3574kip-ft. Figure 41 shows the moment diagram for the entire underpass. Although the SAP analysis revealed a maximum deflection of 0.3ft, which is just below the deflection limit, this design didn't fit the Group's initial goal of creating a slender structure.

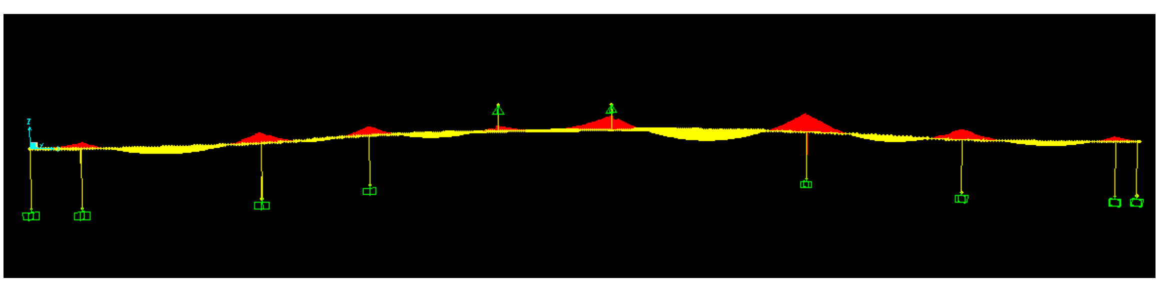


Figure 41: Moment Diagram of the Structure

FINAL DESIGN

MODIFIED DECK

To resolves this issue, the Group decided to incorporate the handrail into the structural system of the underpass. This modified section, consisting of a 3.5ft vertical truss and a 7” diameter hollow tube handrail with a wall thickness of 0.15 ", had a much higher moment of inertia and was therefore able to better resist the bending moments in the structure, which reduced the total deflection observed in the deck.

As a result, the Group was able to reduce the deck’s depth by 33% of the initial design, from 1.5ft to 0.5ft. Cross sections for both the initial and final deck section can be seen in Figure 42 and Figure 43 respectively. The linear dead load for the new section is 0.64 kips-ft and the moment capacity is 12,786kip-ft.

In order to resist buckling under compression in the handrail, the first design called for a solid 6” diameter tube. However, given the weight of a solid handrail, the Group changed the design to a hollow tube with a very small thickness. The value of the critical load in the compression zone was determined to be 141.63 kips. For a 7" diameter handrail with a wall thickness of 0.15", this is equivalent to a maximum design compressive stress of 42.8 ksi while the maximum stress in the handrail was calculated to be 37.44 ksi. In effect, since the design compressive stress of the handrail is greater that the maximum compressive stress in the handrail, the structure passes for buckling. However, due to the size of the handrail, a smaller, non-structural, supplementary handrail will be attached to the structure for pedestrian use (Figure 43).

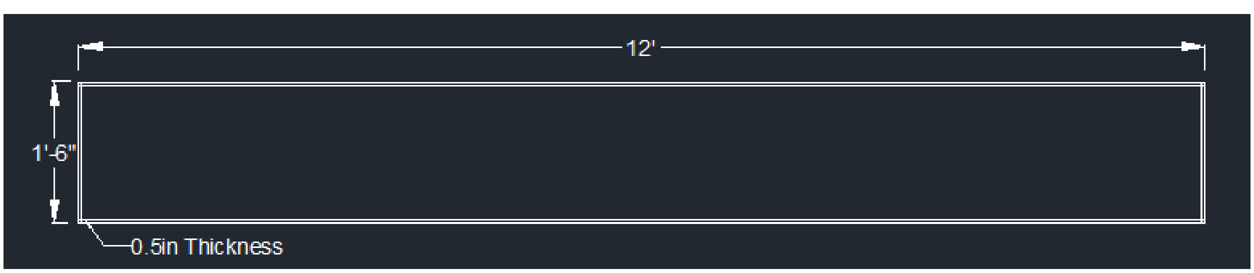


Figure 42: Initial Deck Cross Section

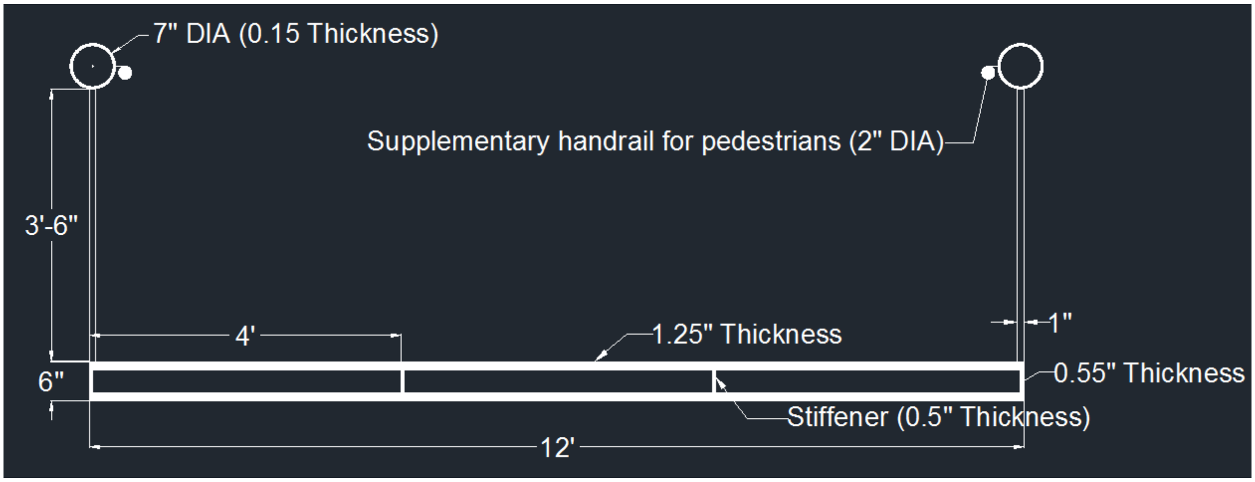


Figure 43: Final Deck Cross Section

GLOBAL DECK DEFLECTION

The Group also evaluated the transverse deflection of the 12ft wide deck and defined the deflection limit as $\Delta L = \frac{L}{360} = 0.033$ ft. The deflections were computed by considering a 1ft section of the deck and using the equation, $\Delta L = \frac{wl^4}{384EI}$. In this case, the moment of inertia is taken as the cumulative moment of inertia of both the top and bottom deck. For a flange thickness of 1.25" at the top and bottom of the deck, the actual global deflection is equal to 0.006ft, well within our deflection limit.

LOCAL TRANSVERSE DEFLECTION OF THE UPPER PLATE OF THE DECK

In addition to the global transverse deflection, the Group also evaluated the local deflection of the upper plate of the deck. In this case, the moment of inertia is taken as just the moment of inertia of the top flange. Without the stiffeners, the deflection was 0.524ft, much greater than the deflection limit of 0.0333ft. As a result, stiffeners were added to the deck to reduce the unsupported span in the deck. The initial modification consisted of adding one stiffener to divide the deck into equal halves. With a free span of 6ft the deflection limit was 0.0167ft however, the actual deflection was determined to be 0.0332ft, still above the deflection limit. The final modification included 2 stiffeners further reducing the unsupported span to 4ft. With this configuration, the deflection in the deck is 0.0066ft, which is smaller than the calculated limit of 0.0111ft.

	Acual Deflection	Limit	Length
	ft	ft	ft
Δ(local)	0.524	0.033	12
	0.033	0.017	6
	0.006	0.011	4

Summing the deflection of both the local and global transverse deflection, the maximum deflection that the deck can experience is determined to be 0.0126ft, which is still below our deflection limit of 0.033ft.

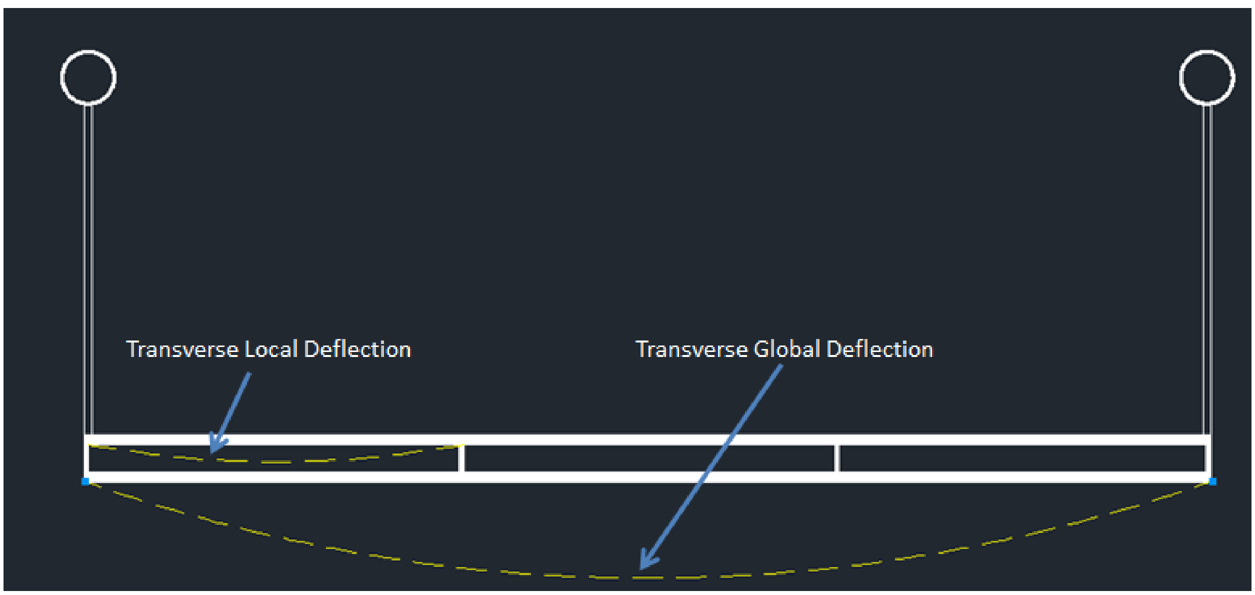


Figure 44: Transverse Local and Global Deflection

COLUMNS

As shown in Figure 38, two columns will be used to support the deck where the deck rests on columns. Each column will be a 2ft diameter cylindrical steel member with a wall thickness of ¾” (Figure 45). The columns for the underpass will be constructed out of 60ksi steel and will have a maximum height of 22ft from the river bed. As can be seen in the Appendix N the columns have been designed for compressive strength and for flexural buckling. Calculations showed a maximum allowable dead load for columns of 391 kips. SAP analysis revealed that the maximum reaction was 284 kips which is within the design compressive strength of the column.

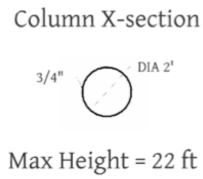


Figure 45: Underpass Columns

They will be driven into position with a barge-mounted pile-driver.

CABLES

As noted earlier, 4 steel cables are required to support the deck spans underneath the arch (Figure 46). The cables will be attached to winches that will be used to raise the deck to create additional waterway in the outer arches as needed. To adequately size the cables, they were modeled as frame elements in SAP. A 2D analysis was used: each frame section representing two cable elements. The maximum tension in the frame sections under full service loading is 316 kips. However, when the bridge is lifted and temporarily out of service, only dead load was considered and the maximum tension in the frame elements is 59.2 kips.

The tensile force, derived from SAP, was divided by the yield strength of 60 ksi, to calculate the minimum required area. Taking into account a safety factor of 0.9, it was determined a cable diameter of 1.2" is required.

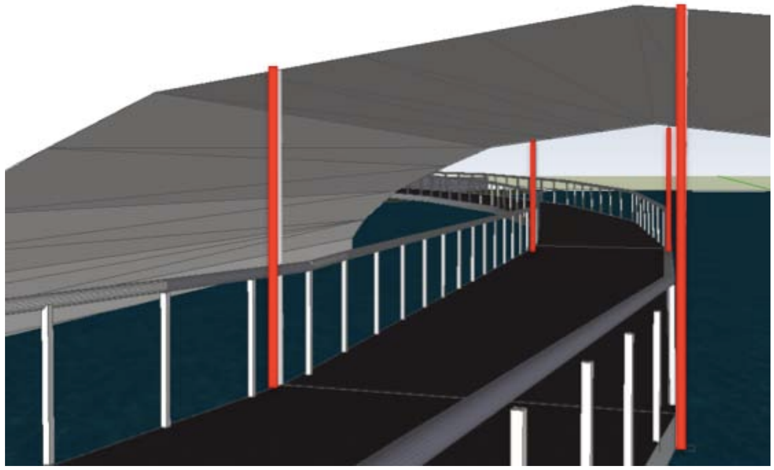


Figure 46: Underpass Cables

CONNECTION

Deck sections will be connected on the top, bottom and the sides. The connections on the sides of the deck are designed to resist the maximum shear force in the structure (Figure 47). SAP analysis revealed a maximum shear force of 174kips. The initial design of the shear connection required 8 (2 rows and 4 columns) 1.25in diameter bolts, spaced at evenly at 3inches. This configuration was determined to have a design strength of 211.3kips for the bolt group. However, for a 1.25in diameter bolt, the code specifies a minimum edge distance of 2.5in. This meant that the shear plate would need to be at least 8in, much greater than the height of the deck. Therefore, E70XX electrodes, with a fillet weld size of 13/16", will be used to weld a 1in thick plate (5" high and 3" wide) to the sides of the deck. The resulting shear strength of the connection is 289.52kips, greater than the max shear stress in the structure.

To resist the tension forces in the structure connections will also be installed on top and underneath the deck (Figure 48). From SAP, the max tensile force in the structure was determined to be 2312.2kips. Similar to the shear connection, The Group decided to use an E70XX electrode with a tensile strength of 70kips/in to design the tensile connection. According to AISC table Table J2.4, for a base material with thickness over 0.75in, the minimum size of fillet weld size may not be less than 5/16in. Therefore a 6/16in was chosen as the weld size. Design weld strength of 8.35kips/in was determined to control the design since it is less than the base material shear yield strength and rupture strength. Finally, two 18in longitudinal welds and two 140in transverse welds would be used to resist the tensile forces. The resulting design tensile weld strength of the connection is 3841kips, much greater than the maximum tension force in the structure.

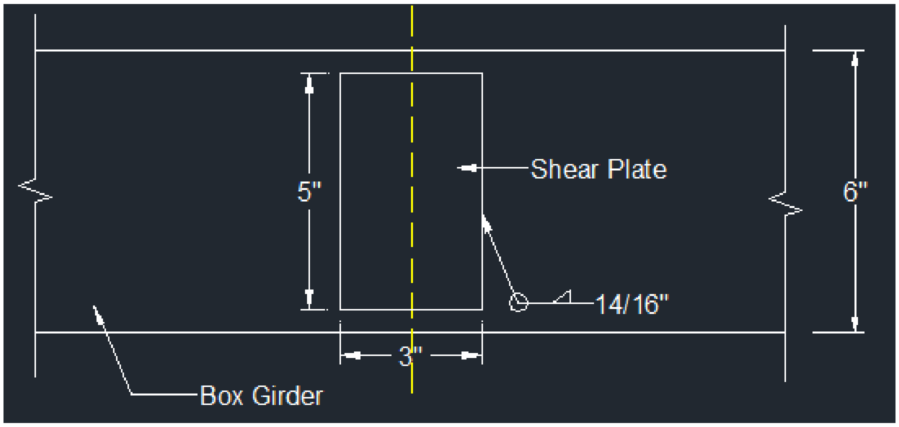


Figure 47: Shear Connection (Side view)

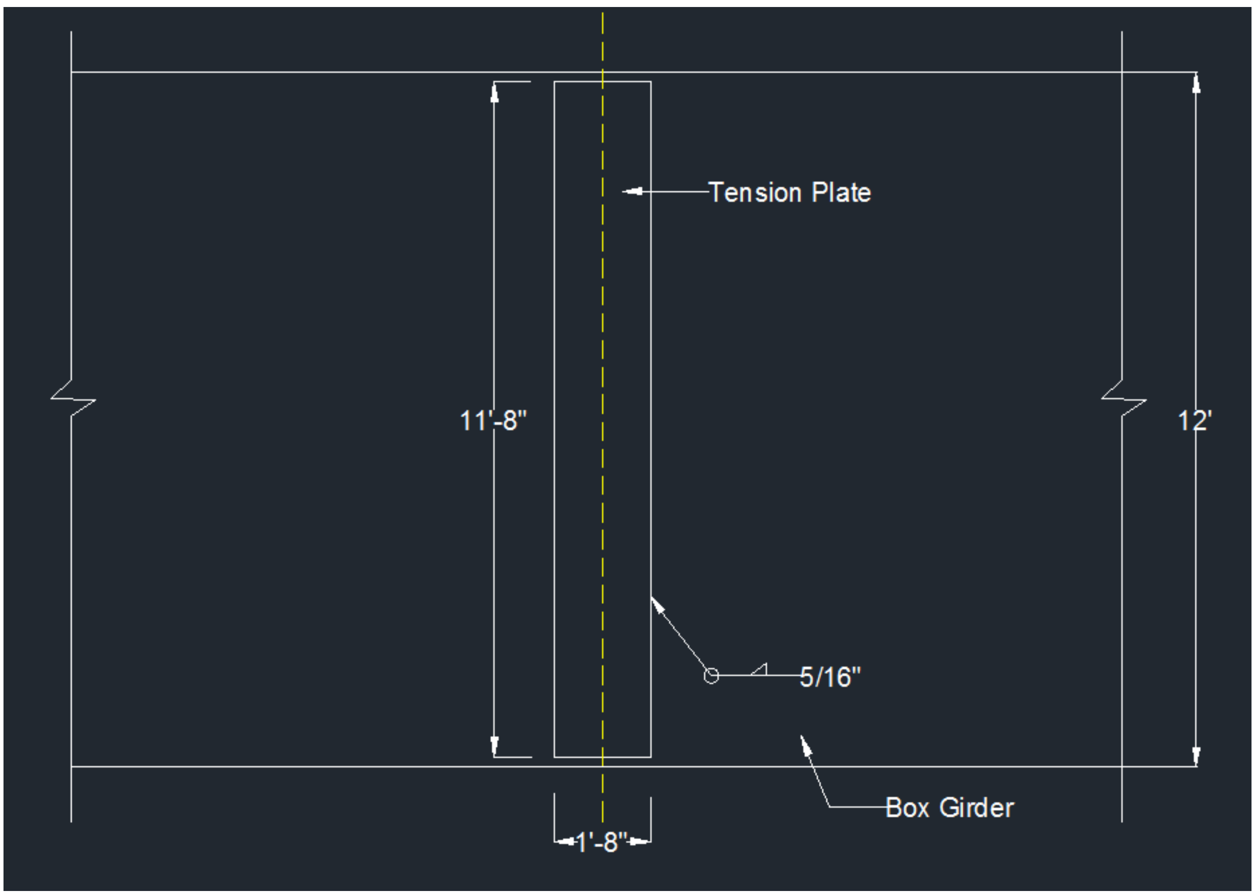


Figure 48: Tensile Connection (Aerial view)

HINGE

To allow the bridge span underneath the arch to be lifted for events such as "The Head of the Charles", a hinge was incorporated into the design to permit rotation, as seen in Figure 49.

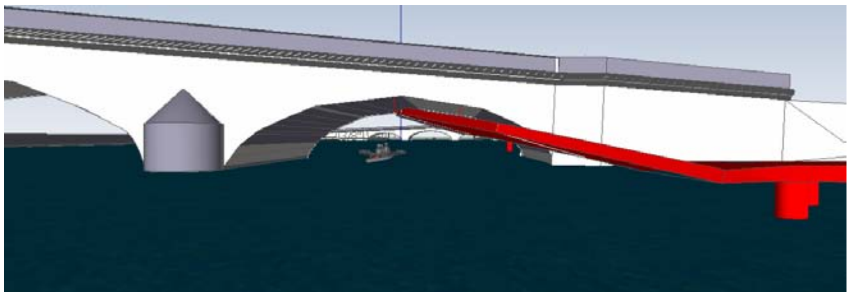


Figure 49: Bridge lifted to allow for river traffic (color added for clarity)

The first step of the hinge design was to determine the location for the hinge that would minimize variations in the bending moment: a location where the bending moment is negative under all load cases was preferable. Several locations were considered but the SAP analysis confirmed that placing the hinge directly over a support column ensured the hinge would always carry a negative moment.

The hinge dimension was calculated based on the shear force obtained from SAP (Figure 50) at the proposed location and it was greater on the right hinge with a value of V=171kips. For ease of constructability, both hinges were designed to resist the maximum shear in the structure.

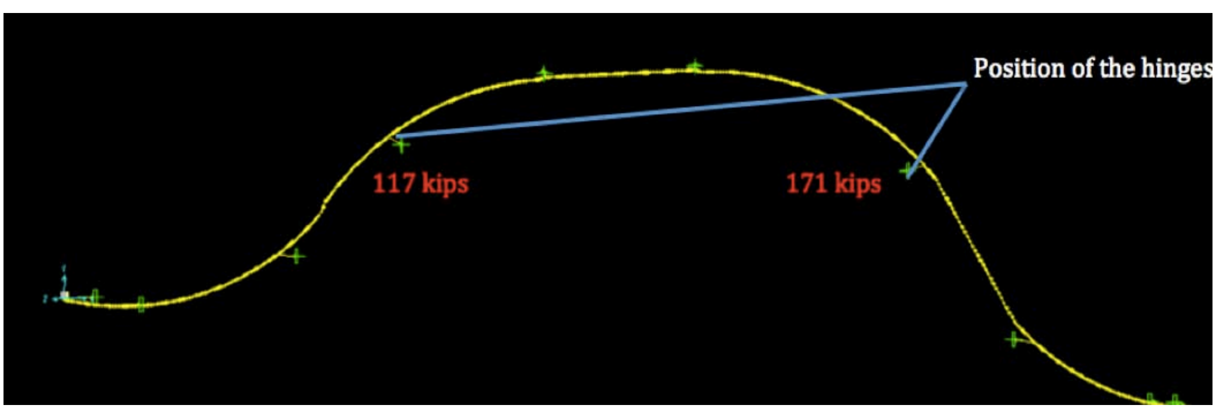


Figure 50: Shear at hinge location

The Group was able to compute the required cross-sectional area for the hinges: $A = \frac{V}{0.58 \times f_y}$ where 0.58 represents the safety factor and f_y denotes the yield strength of steel. A minimum required area of 4.91 square inches is calculated for two connections in double shear.

When the bridge is not lifted, the hinge will be pinned to prevent it from rotating.

Also, at the location of the hinge the handrail could not be continuous as it would require a tedious process of unbolting and re-bolting the handrails. Additionally, at the location of the hinges, the handrail is in tension so there is the added safety concern that unbolting the connections could transform the bolts into projectiles, capable of causing great harm. Therefore the Group came up with a solution, depicted in Figure 51, whereby the handrails could be eased from tension by releasing the link between the two adjacent deck spans. Specifically, as the bridge is lifted with cables, the handrail uncouples, requiring no complicated procedure to disengage the connection.

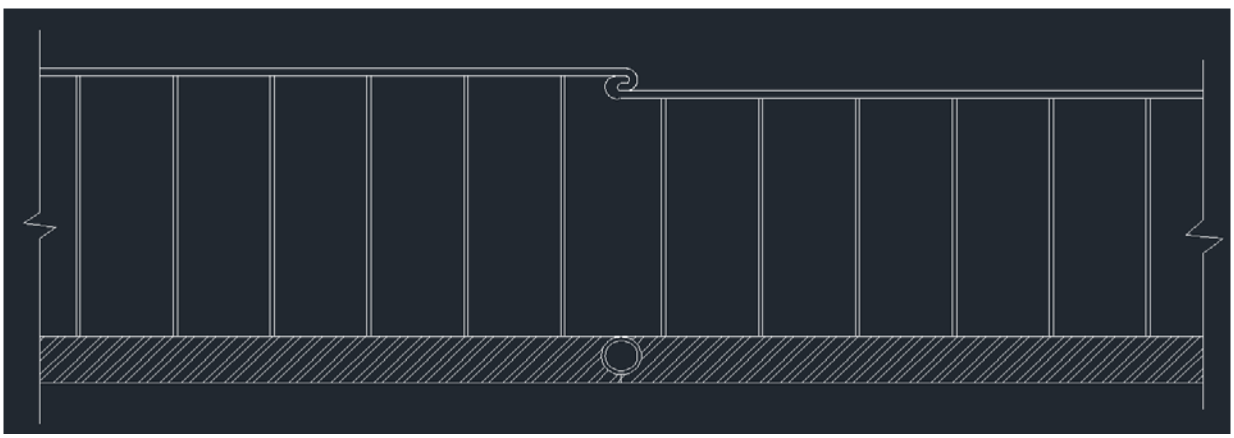


Figure 51: View of hinge and handrail *(not to scale)*

The hinge will be connecting the deck sections at two points, shown below in Figure 52. Pins will be placed about 0.2ft from the axis of rotation and will be provided by the manufacturer.

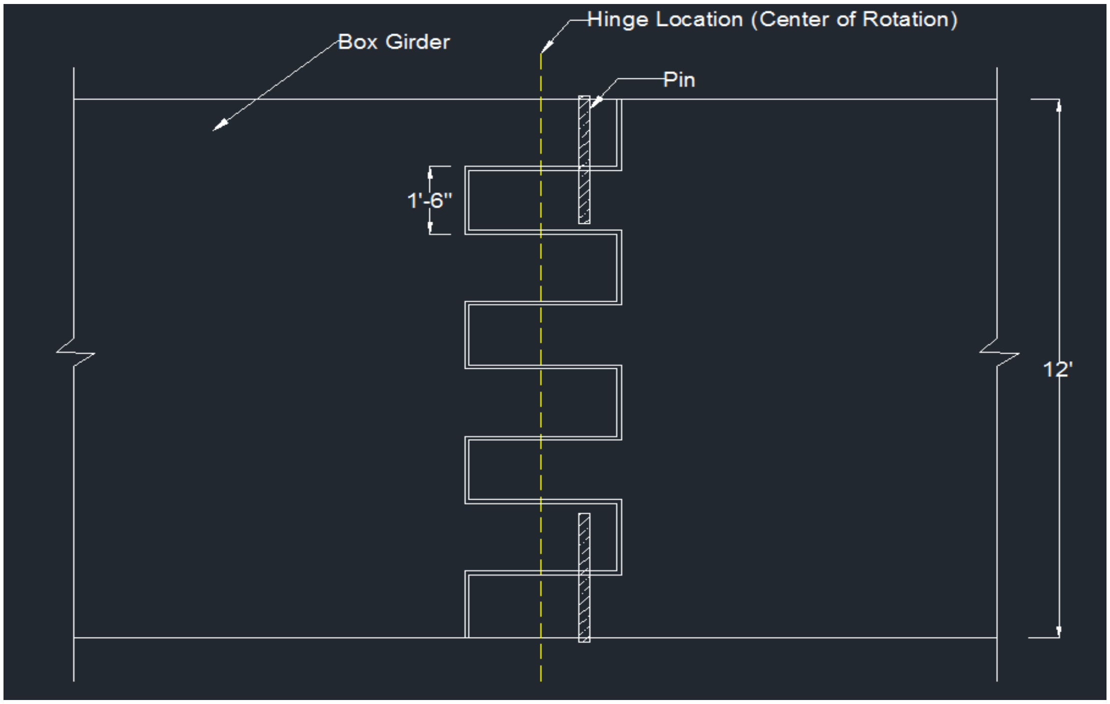


Figure 52: Hinge Connections (aerial view)

MODAL ANALYSIS

After completing static analysis, a modal analysis was performed by the Group. In total, fifteen modes were analyzed with periods ranging from 0.05s to 0.996s. Most of the modes are vertical modes; however, due to safety concerns for end users, the Group paid particular attention to the lateral modes. For example, mode 6 has a period of 0.11s and could be excited during an earthquake, potentially endangering pedestrians. In case the lateral drift was very important, solutions to stiffen the deck would have been implemented. Depending on the amplitude of the displacement solutions to stiffen the underpass would be implemented. Figure 53, Figure 54, and Figure 55 show modes 1, 2, and 3 respectively.

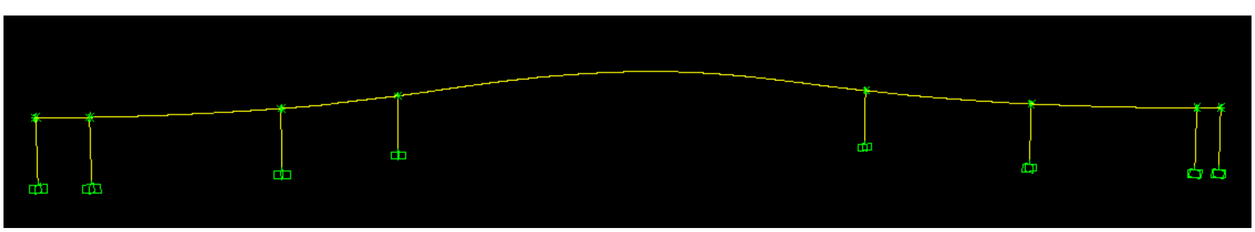


Figure 53 : Mode 1 (T=0.996 s)

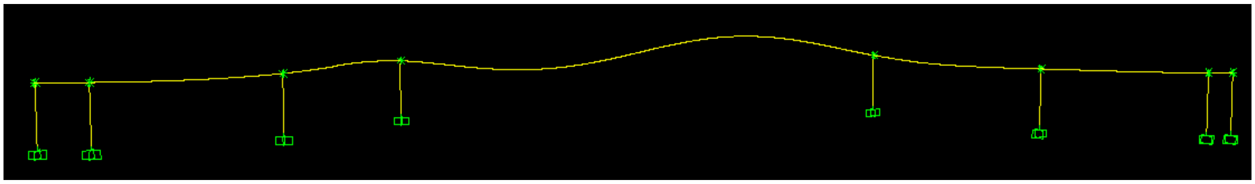


Figure 54: Mode 2 (T=0.35s)

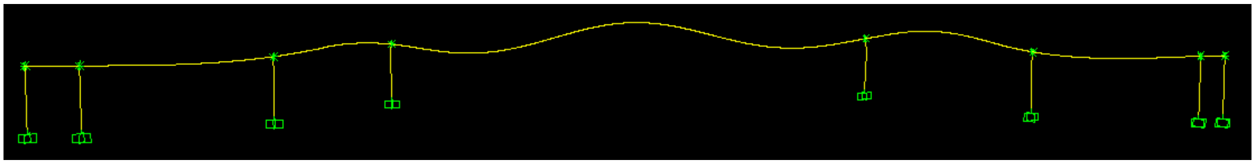


Figure 55 : Mode 3 (T=0.186 s)

Seismic Analysis

Using data provided by USGS, the Group evaluated the impact of an earthquake on the lateral modes. Analysis revealed the period of T=0.11s was exactly in the amplification zone of the Spectral Acceleration function shown in Figure 56 .

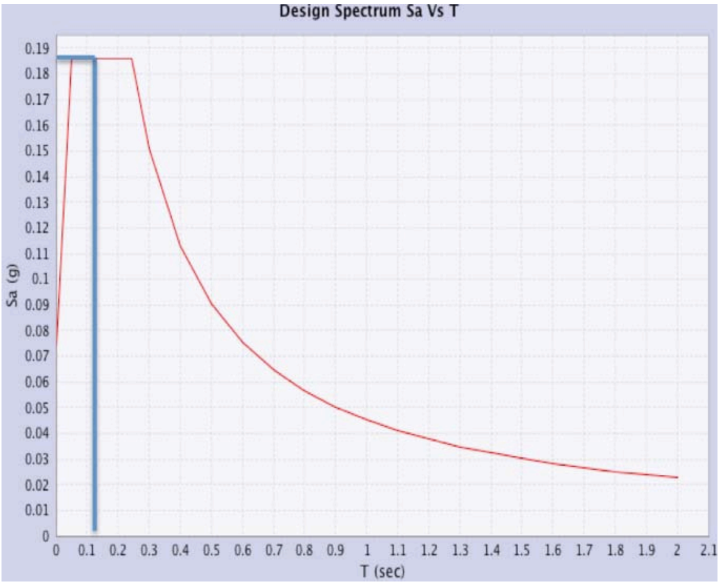


Figure 56: USGS Data- Amplification Zone

Using the modal participation factors obtained from the SAP modal analysis, the lateral drift was calculated to be 0.135". In order to get this result, the value of the displacement is derived by multiplying the modal participation factor, given by SAP, and the spectral displacement for the given mode. The resulting displacement of 0.135" is well within acceptable limit. Had the drift been over the limit, the Group would have tried to correct the problem by raising the stiffness of the underpass or by increasing its mass. In the figure below, the calculation and results obtained are shown:

Period	Sd	Shape Factor		Modal Participation Factor		Actual Displacement	
		ϕ(x)	ϕ(y)	Ux	Uy	Max Ux	Max Uy
Sec	in	in	in	%	%	in	in
0.118	0.041	3.204	3.576	0.747	0.922	**0098**	**0,135**

CONSTRUCTION SEQUENCE

Figure 57 will be referred to throughout the discussion of the construction sequence for the underpass. First, the piles (shown in green) will be driven into bedrock. Then, the deck sections (shown in red) will be placed on those piles. It should be noted that the deck sections where the hinges are located will be prefabricated along with the hinges and then installed in similar fashion to the red deck sections. Next, the cable system will be installed. After the cable system is in place, the portion of the underpass that will be under the arch (shown in turquoise) will be, placed on a barge in two pieces and assembled. That assembly will then be moved under the arch, connected to the cable system and lifted off the barge. Finally, intermediate sections (shown in yellow) will span the gaps. The largest section will be 50ft in length and weigh approximately 40 tons. All sections will be transported by flatbed truck and lifted into place by crane.

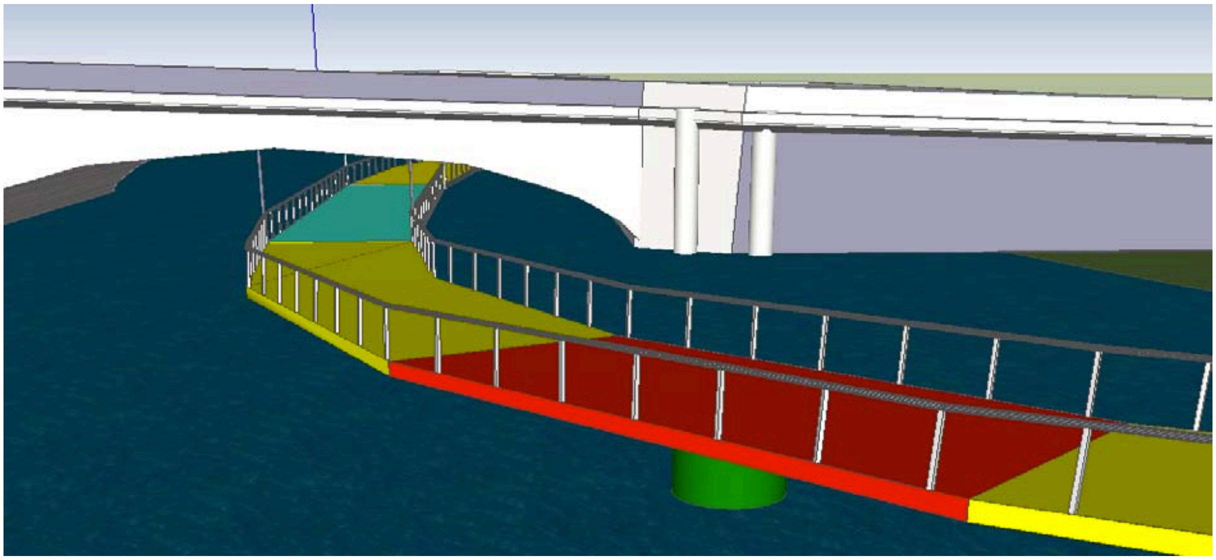


Figure 57: Underpass Construction

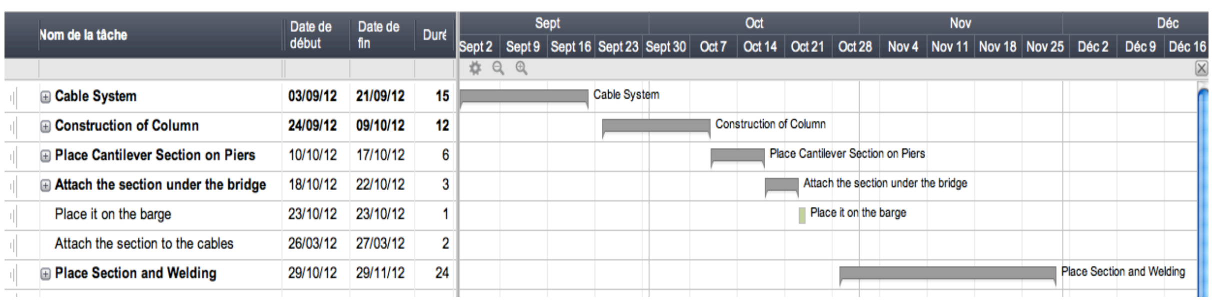
SCHEDULE

In order to limit the on-site construction duration, deck sections will be prefabricated off-site and delivered just-in-time for assembly and installation. The schedule assumes that the parts of the underpass will be ready to be assembled at the start of construction in addition to a minimum labor force of twenty workers on site five days a week.

Given these assumptions and a possible start date of September 3rd 2012 one underpass will be delivered by November 23rd 2012 or a total of sixty three (63) working days.

Task name	Start date	End date	Duration
Cable System	**03/09/12**	**21/09/12**	**15**
Place the winches on the bridge	03/09/12	17/09/12	11
Place the cables	18/09/12	21/09/12	4
Construction of Column	**24/09/12**	**09/10/12**	**12**
Drive Column 1-2	24/09/12	25/09/12	2
Drive Column 3-4	26/09/12	27/09/12	2
Drive Column 5-6	28/09/12	01/10/12	2
Drive Column 7-8	02/10/12	03/10/12	2
Drive Column 9-10	04/10/12	05/10/12	2
Drive Column 11-12	08/10/12	09/10/12	2

Place Cantilever Section on Piers	10/10/12	17/10/12	**6**
Place Section 1	10/10/12	10/10/12	1
Place Section 2	11/10/12	11/10/12	1
Place Section 3	12/10/12	12/10/12	1
Place Section 4	15/10/12	15/10/12	1
Place Section 5	16/10/12	16/10/12	1
Place Section 6	17/10/12	17/10/12	1
Attach the section under the bridge	18/10/12	22/10/12	**6**
Tie the two section	18/10/12	22/10/12	3
Place it on the barge	23/10/12	23/10/12	1
Attach the section to the cables	26/03/12	27/03/12	2
Place Intermediate Section and Welding	29/10/12	10/01/13	**24**
Place Section 1	29/10/12	01/11/12	4
Place Section 2	02/11/12	07/11/12	4
Place Section 3	08/11/12	13/11/12	4
Place Section 4	14/11/12	19/11/12	4
Place Section 5	20/11/12	23/11/12	4
Place Section 6	26/11/12	29/11/12	4
Project Summary	03/09/12	29/11/12	**63**



Nom de la tâche	Date de début	Date de fin	Duré
Cable System	**03/09/12**	**21/09/12**	**15**
Construction of Column	**24/09/12**	**09/10/12**	**12**
Place Cantilever Section on Piers	10/10/12	17/10/12	6
Attach the section under the bridge	18/10/12	22/10/12	3
Place it on the barge	23/10/12	23/10/12	1
Attach the section to the cables	26/03/12	27/03/12	2
Place Section and Welding	29/10/12	29/11/12	24

Cost

In estimating the cost of construction, the Group made the following assumptions:

- Labor – $75 per day
- Barge – $1000 per day
- Crane – $100 per hour
- Steel – $900 per tonne (assume market price remains constant)

- Multiplied by factor of 1.6 to take into account manufacturing and transportation

	Unit Price	Quantity(day)	Total Cost
Total for Equipment (Barge, crane)			$134 400,00
Barge	1000	48	$48 000,00
Crane	1800	48	$86 400,00
Total for Material			$308 611,92
Column	5000	15	$75 000,00
Steel for Deck Section and Handrail	412,73	315	$208 015,92
Cables	1399	4	$5 596,00
Winch	10000	2	$20 000,00
	Cost per day	Number of workers	
Workforce	75	20	$189 000,00
PROJECT COST ESTIMATE (Before Tax)			**$632 011,92**

Under these assumptions, the before tax cost estimate is $632,000 for one underpass and a total project cost of $2,528,000.

TRAFFIC FLOW

Once construction of the new river crossing and underpass are complete, renovations of the existing adjacent bridges can commence. The proposed rerouting of traffic during the renovation is shown below in Figure 58 and Figure 59.

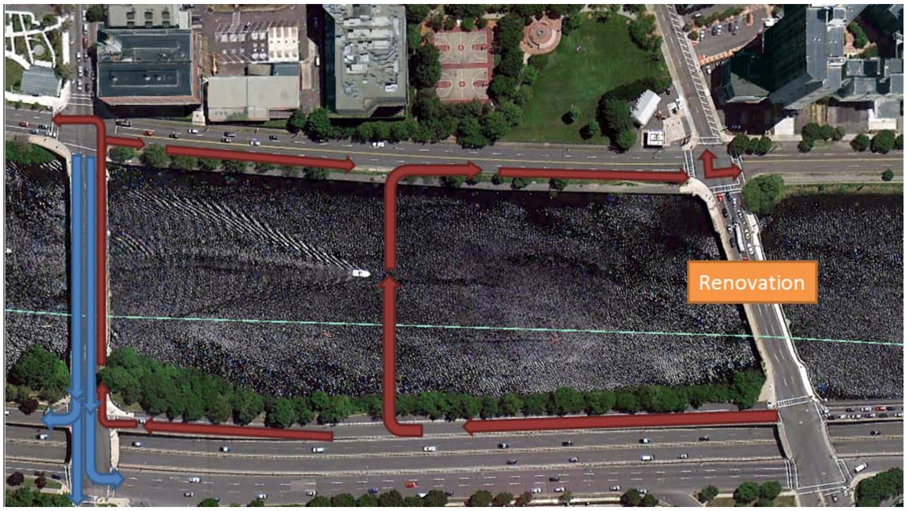


Figure 58: Traffic flow while renovating of River Street Bridge

Figure 59: Traffic flow while renovating Western Avenue Bridge

SUMMARY

The bridge and bypass proposed herein by the Group integrate seamlessly with the surrounding natural and built environment.

The bridge pulls both modern and historic elements together to connect to the existing neighboring bridges and add aesthetic structural elements that are interesting and complex; The three arches mirror the simplicity of the River Street and Western Avenue bridges while the leaping arches are structurally complex yet elegant.

The bypass is visually unobtrusive as it slopes out of sight of vehicle traffic and brings pedestrians close to the water. The hinge system allows the bridge to move out of the way for river, while the remainder of the bypass provides additional vantage points spectators.

The Anchorage Group hopes this comprehensive and elegant solution meets the needs of all stakeholders.

APPENDIX A: RESUMES

Nnabuihe Nnamani

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EXPERIENCE

THE ANCHORAGE GROUP — **Anchorage, AK**
Construction and Engineering Project Manager — **June 2005 – December 2011**

- **DNA Bridge**, Singapore
  Managed the construction of this masterpiece bridge. Supervised the different mechanical, light, structural engineers. Coordinated the work of the different companies.

- **Harbor Drive Pedestrian Bridge**, San Diego
  Carried out quality control inspections to ensure that recommended procedures were followed in correcting concrete defects such as cracks and honeycombs

- **Passerelle Leopold Sedar Senghor**, Paris, France.
  Managed the construction of this bridge situated in a very busy area of Paris. Supervised environmental risk assessment and the impact on the Seine river.

EXXON-MOBIL — **Ras Laffan, Qatar**
Project Manager — **June 2000 – April 2005**

- Managed construction of a gasification plant for EXXON-MOBIL in Qatar. Completed the project under-budget and one year in advance.

ENI-SAIPEM — **Port-Harcout, Nigeria**
Off-Shore Structural Engineer — **June 1998 - May 2000**

- Assisted manager in designing off-shore structures for super major oil companies
- Supervised the finite element analysis of the team within ENI

EDUCATION

Massachusetts Institute of Technology (MIT) — **Cambridge, MA**
Civil and Environmental Engineering Department — **June, 1998**
Master of Engineering in High Performance Structures

The George Washington University — **Washington, DC**
Bachelor of Science in Civil and Environmental Engineering — **May, 1997**

AWARDS AND PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers
National Society for Black Engineers

SKILLS

Computer: Microsoft Office, STAAD Pro, RISA 3D, AutoCAD, MATLAB
Foreign Languages: Igbo (fluent) and French (conversant)

Stephen Pendrigh

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EXPERIENCE

THE ANCHORAGE GROUP **Anchorage, AK**
Construction Engineer June 2005 – December 2011

- **Marina Bay Sands**, Singapore
  Supervised the construction on this very large-scale project. Planned and scheduled the work on the building site. Coordinated the different companies on site.

- **Hoover Dam Bridge**, Las Vegas
  Led the team during the construction of the Hoover Dam bridge. Used extremely innovative solutions to make this project a succes and a state of the art bridge.

- **Passerelle Leopold Sedar Senghor**, Paris, France.
  Managed the construction of this bridge situated in a very busy area of Paris..

AECOM **Hong-Kong**
Construction Engineer June 2000 – April 2005

- Managed renovation of Kai Tak airpot in Hong Kong. Completed the project under-budget and one year in advance.

ARUP **London, UK**
Structural Engineer June 1998 - May 2000

- Participated to the solution given to the Millenium Bridge problem in London

EDUCATION

Massachusetts Institute of Technology (MIT) **Cambridge, MA**
Civil and Environmental Engineering Department June, 1998
Master of Engineering in High Performance Structures

University of Cambridge, Queens' college **Cambridge, UK**
Bachelor of Science in Civil and Environmental Engineering May, 1997

AWARDS AND PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers.
Licensed PE in Structural Engineering in MA, AK.
Member, Boston Society of Civil Engineers.

SKILLS

Computer: Microsoft Office, STAAD Pro, RISA 3D, AutoCAD, MATLAB
Foreign Languages: Spanish (fluent) and German (conversant)

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EXPERIENCE

THE ANCHORAGE GROUP — **Anchorage, AK**
CEO and Head of Design — June 2005 – December 2011

- **Zaragoza Bridge Pavilion**, Spain
  Supervised the design of this project.

- **Hoover Dam Bridge**, Las Vegas
  Responsible for the design of the bridge and its visual integration in the environment.

- **Calatrava's Bridge**, Valencia, Spain
  Managed the design of this innovative bridge.

FOSTER+PARTNERS — **London, UK**
Senior Partner — June 2000 – April 2005

- Managed the design of the Viaduc de Millau in France which is the higher bridge in the world and one of the most emblematic state of the realization of Foster+Partners

ZAHA HADID ARCHITECTS — **London, UK**
Associate Architect — June 1998 - May 2000

- Participated to the design of the CMA-CGM headquarters in Marseille.
  Was in charge of the relation with the clients and the engineers.

EDUCATION

Massachusetts Institute of Technology (MIT) — **Cambridge, MA**
Department of Architecture — June, 1998
Master of Architecture

Ecole Spéciale des Travaux Publics — **Paris, France**
Bachelor of Science in Civil and Environmental Engineering — May, 1997

Lycée Pasteur — **Neuilly-sur-Seine, France**
Intensive Mathematics and Physics — May 1995

AWARDS AND PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers.
Licensed Architect
Member, Boston Society of Civil Engineers.

SKILLS

Computer: Microsoft Office, SAP, AutoCAD, MATLAB

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EXPERIENCE

THE ANCHORAGE GROUP **Anchorage, AK**
Structural Engineer June 2005 – December 2011

- **DNA Bridge**, Singapore
  Supervised the design of this project.

- **Hoover Dam Bridge**, Las Vegas
  Responsible for the design of the bridge and its visual integration in the environment.

- **Calatrava's Bridge**, Valencia, Spain
  Managed the design of this innovative bridge.

THORNTON TOMASETTI Inc. **NYC, USA**
Senior Partner June 2000 – April 2005

- Responsible of design for multi-unit condominium projects. Supervised 20 structural engineers

ARUP **NYC, USA**
Associate Structural Engineer June 1998 - May 2000

- Participated to the design of the Lincoln Center in NYC

EDUCATION

Massachusetts Institute of Technology (MIT) **Cambridge, MA**
Department of Civil and Environmental Engineering June, 1998
Master of Engineering in High Performances Structures

Stevens Institute of Technology **Hoboken, NJ**
Bachelor of Engineering in Civil and Environmental Engineering, High Honors, GPA 3.74/4 May, 1997

AWARDS AND PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers.
Professional Engineer
Member, Boston Society of Civil Engineers.

SKILLS

Computer: Microsoft Office, SAP, AutoCAD, MATLAB

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EXPERIENCE

THE ANCHORAGE GROUP — **Anchorage, AK**
Senior Structural Engineer — June 2006 – December 2011

- **DNA Bridge**, Singapore
  Managed the structural design of this masterpiece bridge.

- **Gateshead Millenium Bridge**
  Executed the entire design of this spectacular bridge in the UK.
  Won the *IStructE Supreme Award*

- **Passerelle Leopold Sedar Senghor**, Paris, France.
  Applied technical expertise and common sense evaluation of new requirements to ensure the project was coordinated

NAVALE MOBILE CONSTRUCTION BATTALION 74 — **Afghanistan**
Project Manager — June 2000 – April 2005

- Managed 106-person workforce consisting of military construction and engineering personnel at 13 forward operating bases (FOBs) spread across Afghanistan.

NAVAL FACILITIES ENGINEERING COMMAND FAR EAST — **Yokosuka, Japan**
Project Manager — June 1998 - May 2000

- Managed 40+ projects valued at over $50M
- Evaluated project designs for constructability and provided technical input to Architect/Engineer

EDUCATION

Massachusetts Institute of Technology (MIT) — **Cambridge, MA**
Civil and Environmental Engineering Department — June, 2006
Master of Engineering in High Performance Structures

University of Alaska — **Anchorage, AK**
Bachelor of Science in Civil and Environmental Engineering — May, 1997

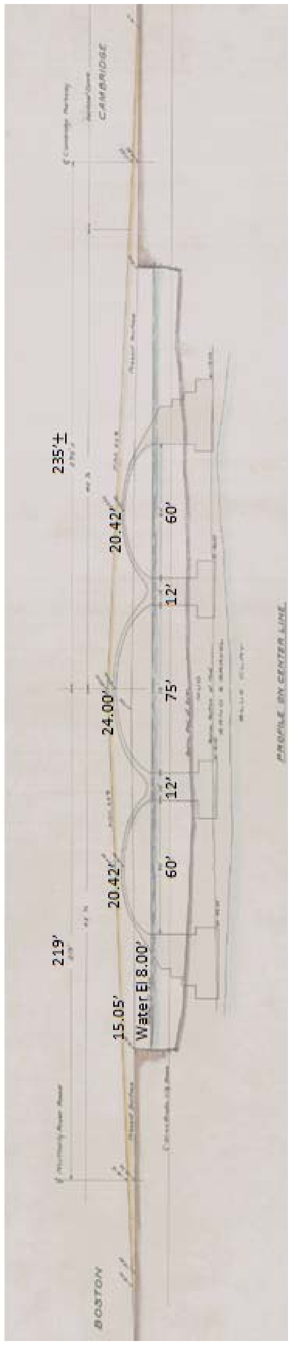
AWARDS AND QUALIFICATIONS

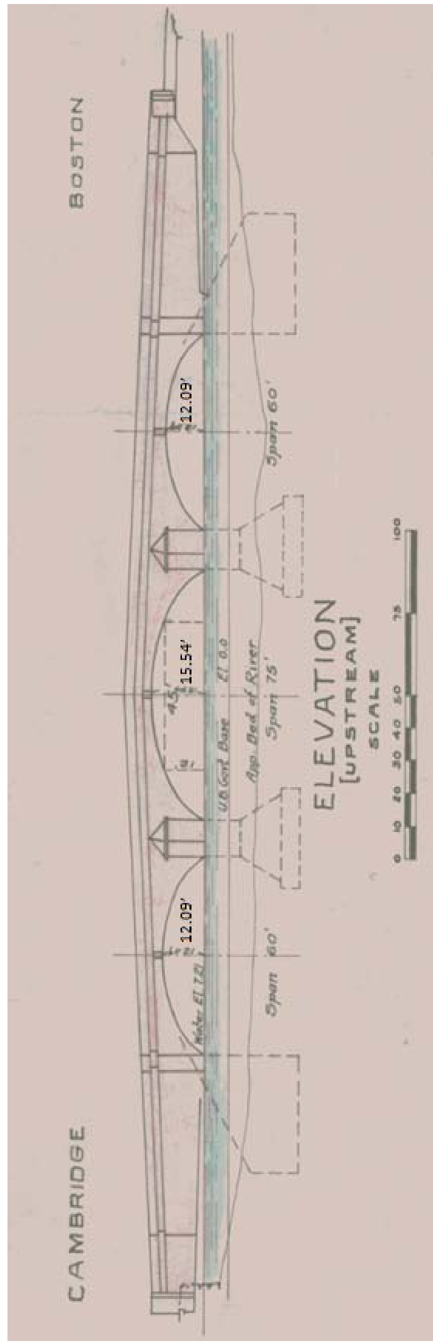
PE (AK)
American Society of Civil Engineers
Top Secret Clearance

SKILLS

Computer: Microsoft Office, STAAD Pro, RISA 3D, AutoCAD, MATLAB
Foreign Languages: Mandarin (fluent) and Japanese (fluent)

APPENDIX B: EXISTING BRIDGES





APPENDIX C: BRIDGE CABLES

Max Gravity Cable Load		
Tributary Width	16.25	ft
Tributary Length	10	ft
Live Load	0.15	ksf

$$LL_{cable} = LL_{ksf} \times TributaryArea$$
$$LL_{cable} = LL_{ksf} \times TributaryWidth \times TributaryLength$$

Live Load per Cable	24.375	kips
Dead Load	0.49	kcf
# Girders per cable	0.5	#
Girder Length	25	ft
Area per girder	0.188	ft^2
Deck Area	1.8	ft^2

$$DL_{cable} = \left(DL_{kcf} \times \#Girder \times A_{girder} \times L_{girder}\right)$$
$$+\left(DL_{kcf} \times A_{deck} \times TributaryWidth\right)$$

Dead Load Per Cable	15.48	kips
Using Factors: 1.2DL+1.6LL		
Total Gravity Load Per Cable	57.58	kips

Cable Angles and Axial Loads				
	Height	**Length**	**Angle**	**Axial Load**
	ft	**ft**	**radians**	**kips**
Cable 1	27.36	35.18	0.89	74.05
Cable 2	29.87	33.83	1.08	65.22
Cable 3	31.37	33.27	1.23	61.06
Cable 4	31.88	33.41	1.27	60.35
Cable 5	31.37	34.19	1.16	62.76
Cable 6	29.87	35.63	0.99	68.68
Cable 7	27.36	37.75	0.81	79.45
Maximum Axial Load	79.45 kips			

$$A_{cable} = \frac{P}{\phi\sigma} = \frac{79.45kips}{(0.9)50ksi} = 1.77in^2$$

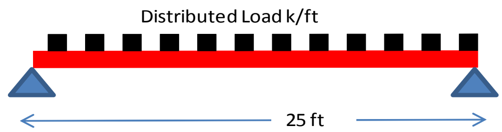
$$d_{cable} = \frac{\sqrt{4A_{cable}}}{\pi} = 1.50in$$

As added safety, the group chose 2.0" cables

APPENDIX D: BRIDGE GIRDERS

Max Girder Load		
Tributary Width	17.5 ft	
Live Load	0.15 ksf	
$LL_{girder} = LL_{ksf} \times TributaryWidth$		
Live Load Distributed per Girder	2.625 kip/ft	
Dead Load	0.49 kcf	
Deck Area	1.8 ft^2	
$DL_{girder} = DL_{kcf} \times A_{deck} \times TributaryWidth$		
Dead Load per Girder	15.44 kips	
Dead Load Distributed per Girder	0.6174 kip/ft	
Using Factors: 1.2DL+1.6LL		
Total Gravity Load Per Cable	4.94 kip/ft	
Using W16x31 (assuming girder self weight is cambered out)		
L	25 ft	
F_y	50 ksi	
Z_x	54 in^3	
Mmax	386 kip-ft	
ϕMallow	202.50 kip-ft	
*Moment does not work: $M_{max}>\phi M_{allow}$		
Try, W16x57 (assuming girder self weight is cambered out)		
L	25 ft	
I	758 in^4	
F_y	50 ksi	
Z_x	105 in^3	
Mmax	386.01 kip-ft	
ϕMallow	393.75 kip-ft	98.03%
*Moment criteria met: $M_{max}<\phi M_{allow}$		
E	29000 ksi	
Δ max TL	1.98 in	
Δ allow TL	1.25 in	158.04%
Δ max LL	1.05 in	
Δ allow LL	0.83 in	125.95%
*Deflection criteria met: $\Delta_{max}>\Delta_{allow}$		
Try, W21x62 (assuming girder self weight is cambered out)		
L	25 ft	
I	1330 in^4	
F_y	50 ksi	
Z_x	144 in^3	
Mmax	386.01 kip-ft	
ϕMallow	540.00 kip-ft	71.48%
*Moment criteria met: $M_{max}<\phi M_{allow}$		
E	29000 ksi	
Δ max TL	1.13 in	
Δ allow TL	1.25 in	90.07%
Δ max LL	0.60 in	
Δ allow LL	0.83 in	71.78%
*Deflection criteria met: $\Delta_{max}<\Delta_{allow}$		
Girder Chosen: W21x62		

Note: Others did work, but chose section with smallest depth and weight



$$M_{max} = \frac{1}{8} wL^2$$

$$M_{allow} = \frac{I\sigma}{y}$$

$$\Delta_{max} = \frac{5wL^4}{384EI}$$

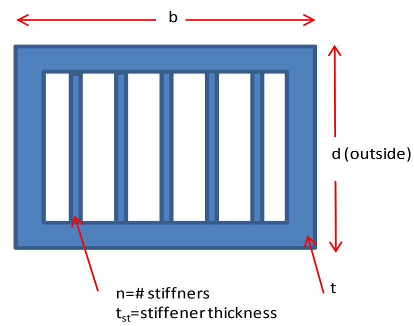
$$\Delta_{allowTL} = \frac{L}{240}$$

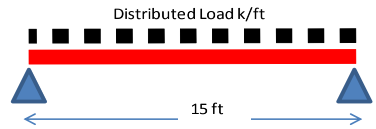
$$\Delta_{allowLL} = \frac{L}{360}$$

APPENDIX E: BRIDGE DECK

Goal Seek Table

Deck Box Section w/ Stiffeners		
b	20 ft	
d	0.67 ft	
t	0.04 ft	
L	15 ft	
n	7.131363489 #	
t_{st}	0.021 ft	
Area	1.8 ft^2	
I	0.17 ft^4	
Loading		
LL	0.15 ksf	
	3 kip/ft	
1.6LL	4.8 kip/ft	
DL	0.49 kcf	
	0.88 kip/ft	
1.2DL	1.06 kip/ft	
Total Load	5.86 kip/ft	
σ	60 ksi	
E	29000 ksi	
Deflection Criteria		Pass?
Δ max TL	0.067 in	Yes
Δ allow TL	0.750 in	8.87%
Δ max LL	0.054 in	Yes
Δ allow LL	0.500 in	10.90%
*Deflection criteria met (LL only): $\Delta_{max}<\Delta_{allow}$		
Moment Criteria		
Mmax	164.8 kip-ft	Yes
ϕMallow	4324.4 kip-ft	3.81%
*Moment criteria met: $M_{max}<\phi M_{allow}$		





$$M_{max} = \frac{1}{8}wL^2$$

$$M_{allow} = \frac{I\sigma}{y}$$

$$\Delta_{max} = \frac{5wL^4}{384EI}$$

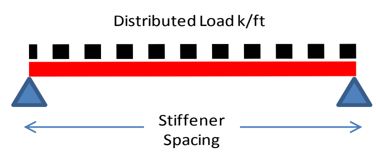
$$\Delta_{allowTL} = \frac{L}{240}$$

$$\Delta_{allowLL} = \frac{L}{360}$$

Deflection Between Stiffeners (Top Plate 15 ft)

I (top plate)	0.0000904 ft^4
Spacing	2.805 ft
b	15 ft

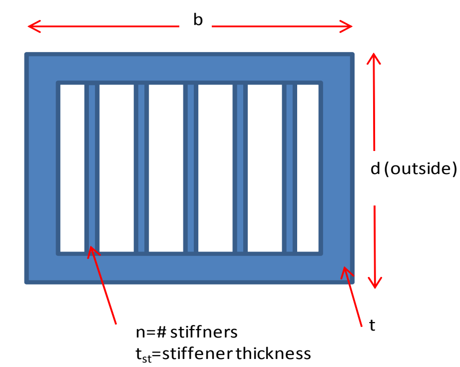
Loading		
LL	0.15 ksf	
	2.25 kip/ft	
1.6LL	3.6 kip/ft	
DL	0.49 kcf	
	0.31 kip/ft	
1.2DL	0.37 kip/ft	
Total Load	3.97 kip/ft	
σ	60 ksi	
E	29000 ksi	
Deflection Criteria		Pass?
Δ max TL	0.102 in	Yes
Δ allow TL	0.140 in	72.43%
Δ max LL	0.092 in	Yes
Δ allow LL	0.093 in	98.58%
*Deflection criteria met (LL only): $\Delta_{max}<\Delta_{allow}$		
Moment Criteria		
Mmax	3.9 kip-ft	Yes
ϕMallow	37.5 kip-ft	10.40%
*Moment criteria met: $M_{max}<\phi M_{allow}$		

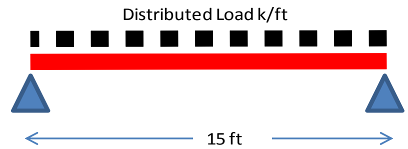


Using "Goal-Seek"
- Discovered that LL Deflection Criteria is the controlling factor (not TL Deflection or Moment Criteria)
- Using several iterations, using goal seek to set Max LL Deflection=Allowable LL Deflection by changing number of stiffeners
- Value determined is n=7.13 stiffners at 2.8 ft apart
- Therefore, rounding to 8 stiffeners at 2.5 ft apart

Values Used in Design

Deck Box Section w/ Stiffeners		
b	20 ft	
d	0.67 ft	
t	0.04 ft	
L	15 ft	
n	8 #	
t_{st}	0.021 ft	
Area	1.8 ft^2	
I	0.17 ft^4	
Loading		
LL	0.15 ksf	
	3 kip/ft	
1.6LL	4.8 kip/ft	
DL	0.49 kcf	
	0.89 kip/ft	
1.2DL	1.07 kip/ft	
Total Load	5.87 kip/ft	
σ	60 ksi	
E	29000 ksi	
Deflection Criteria		Pass?
Δ max TL	0.066 in	Yes
Δ allow TL	0.750 in	8.86%
Δ max LL	0.054 in	Yes
Δ allow LL	0.500 in	10.88%
*Deflection criteria met (LL only): $\Delta_{max}<\Delta_{allow}$		
Moment Criteria		
Mmax	165.0 kip-ft	Yes
ϕMallow	4332.2 kip-ft	3.81%
*Moment criteria met: $M_{max}<\phi M_{allow}$		





$$M_{max} = \frac{1}{8} wL^2$$

$$M_{allow} = \frac{I\sigma}{y}$$

$$\Delta_{max} = \frac{5wL^4}{384EI}$$

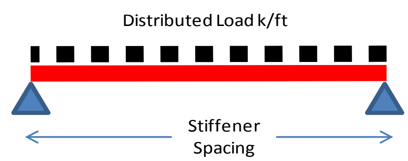
$$\Delta_{allowTL} = \frac{L}{240}$$

$$\Delta_{allowLL} = \frac{L}{360}$$

Deflection Between Stiffeners (Top Plate 15 ft)

I (top plate)	0.0000904 ft^4	
Spacing	2.5 ft	
b	15 ft	

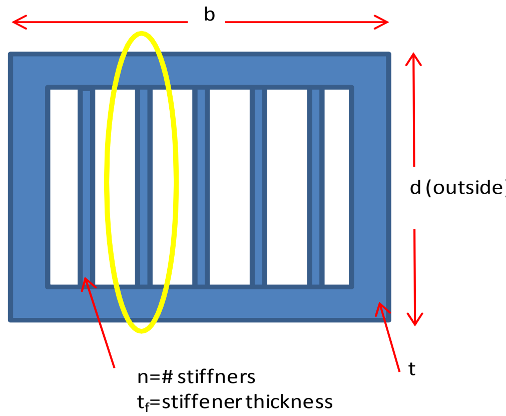
Loading		
LL	0.15 ksf	
	2.25 kip/ft	
1.6LL	3.6 kip/ft	
DL	0.49 kcf	
	0.31 kip/ft	
1.2DL	0.37 kip/ft	
Total Load	3.97 kip/ft	
σ	60 ksi	
E	29000 ksi	
Deflection Criteria		Pass?
Δ max TL	0.064 in	Yes
Δ allow TL	0.125 in	51.30%
Δ max LL	0.058 in	Yes
Δ allow LL	0.083 in	69.83%
*Deflection criteria met (LL only): $\Delta_{max}<\Delta_{allow}$		
Moment Criteria		
Mmax	3.1 kip-ft	Yes
ϕMallow	37.5 kip-ft	8.27%
*Moment criteria met: $M_{max}<\phi M_{allow}$		

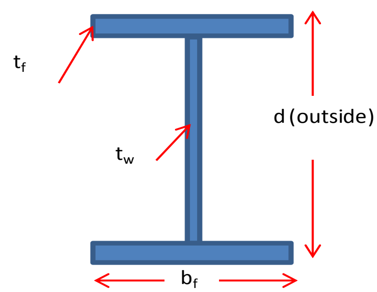


Check Lateral Torsional Buckling

* Check between stiffeners, treat as equivalent I beam

b_f	30	in
d	8	in
t_f	0.5	in
t_w	0.25	in
h_c	3.875	in
I_x	429.6	in^4
I_y	2250.0	in^4
A	31.75	in^2
r_y	8.42	in
J	2679.7	in^4
h_o	8.5	in
S_{xc}	212.8	in^3
S_{xt}	212.8	in^3
Z_x	112.5	in^3
F_y	60	ksi
E	29000	ksi





1. Compression Flange Yielding

$$\boxed{M_p = F_y Z_x \le 1.6 F_y S_{xc}}$$

M_p	6750.00	kip-in
M_{yc}	12768.75	kip-in
R_{pc}	0.529	
M_n	6750	kip-in

2. Lateral Torsional Buckling

aw	0.06	in
rt	8.61	in
Lp	208.32	in
	17.36	ft
FL	42	ksi
Lr	19963.46	in
	1663.6218	ft

Therefore, Lb=10ft.
Decide to place stiffeners every 15 ft to correspond to girder locations.

3. Compression Flange Buckling

*Non Compact Section

M_p	6750.00	kip-in
M_{yc}	12768.75	kip-in
R_{pc}	0.529	
λ	30	in
λ_{pf}	8.35	in
λ_{rf}	21.98	in
M_n	10224.8	kip-in
M_{max}	1979.7	kip-in

M_n>M_{max}

Therefore, not controlling

Check Compactness

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$$

$$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}}$$

$$\lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}}$$

$$\lambda_{rw} = 5.70\sqrt{\frac{E}{F_y}}$$

Flange	
λ_{pf}	8.35
λ_{rf}	21.98
Noncompact	

Web	
λ_{pw}	82.66
λ_{rw}	125.31
Noncompact	

4. Tension Flange Yielding

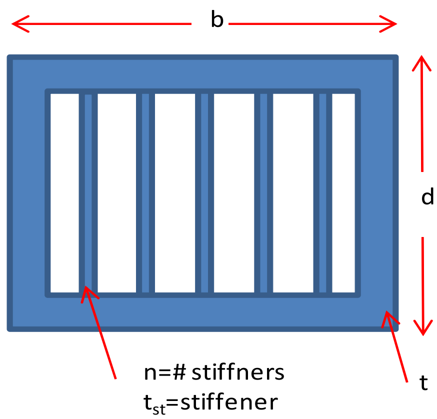
S_{xt}=S_{xc}

Therefore, doesn't apply

APPENDIX F: BRIDGE DECK BOLTS

Bolt Calculations

Using 84 ksi bolts	
ϕ	0.75
Bolt Diameter	0.75 in
A_b	0.44 in^2
$F_n=F_{nv}$	84 ksi
$R_n=F_nA_b$	37.11 kips
From SAP	
Max Moment	295 kip-ft
Max Bolt Shear	442.5 kips
Arm	6.25 in
# Bolts	11.92 bolts
	12 bolts
Total Length	20 ft
Bolt Spacing	1.5 ft



Using 84 ksi bolts: 12 bolts spaced 1.5 ft

Using 68 ksi bolts	
ϕ	0.75
Bolt Diameter	0.75 in
A_b	0.44 in^2
$F_n=F_{nv}$	68 ksi
$R_n=F_nA_b$	30.04 kips
From SAP	
Max Moment	295 kip-ft
Max Bolt Shear	442.5 kips
Arm	6.25 in
# Bolts	14.73 bolts
	15 bolts
Total Length	20 ft
Bolt Spacing	1.3 ft

Using 68 ksi bolts: 15 bolts spaced 1.3 ft

Connection Plates

Shear Check	
From SAP	
Max Bolt Shear	566.4 kips
ϕ	0.9
F_y	36 ksi
w	240 in
t	0.121 in

$$V = 0.6\phi F_y wt$$

Yielding	
From SAP	
Max Axial	442.5 kips
ϕ	0.9
F_y	36 ksi
w	240 in
t	0.057 in

$$P = \phi F_y wt$$

Rupture (84 ksi bolts)		
From SAP		
P	442.5	kips
d_{bh}	0.875	in
U	1	
ϕ	0.75	
F_u	58	ksi
w	240	in
t	0.044	in

$$P = \phi F_u A_g U$$

$$t = \frac{P}{\phi F_u \left(w - nd_{bh}\right)U}$$

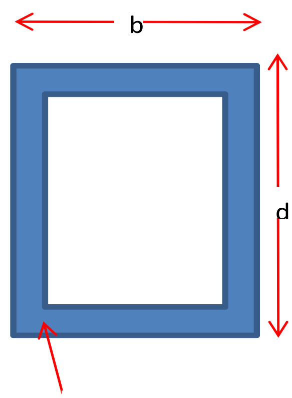
Yielding governs. Thickness of plates need to be >0.057in, the Group is choosing to use 0.5in steel plates

APPENDIX G: BRIDGE ARCH SECTION

Original Dimensions

Arch Section		
b	18.81	in
d	24.81	in
t	0.50	in
Area	43	in^2
I_3	3901.8	in^4
I_2	2548.8	in^4

Design arch to meet moment criteria in both directions.
Then check deflection output from SAP2000 model



Moment Check			
σ	60	ksi	
E	29000	ksi	Pass?
ϕM3	16988.26	kip-in	Yes
	1415.69	kip-ft	77.21%
SAP2000 MAX M3	1093.00	kip-ft	Capacity
ϕM2	14638.24	kip-in	Yes
	1219.85	kip-ft	99.93%
SAP2000 MAX M2	1219.00	kip-ft	Capacity

Axial Load Check			
σ	60	ksi	Pass?
ϕP	2300.96	kip	Yes
			28.03%
SAP2000 MAX P	645.00	kip	Capacity

SAP2000 Deflection: Uy too large (over 1 ft in arch)

Final Dimensions

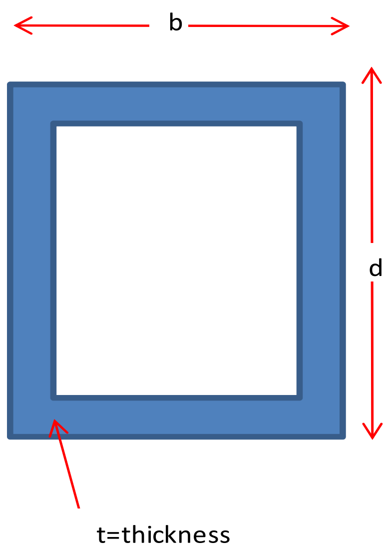
Arch Section		
b	31.00	in
d	36.00	in
t	0.50	in
Area	66	in^2
I_3	13340.5	in^4
I_2	10623.0	in^4

Through trial and error, changed dimensions of box section to satisfy diflection criteria (and moment)

Moment Check			
σ	60	ksi	
E	29000	ksi	Pass?
ϕM3	40021.50	kip-in	Yes
	3335.13	kip-ft	32.77%
SAP2000 MAX M3	1093.00	kip-ft	Capacity
ϕM2	37009.16	kip-in	Yes
	3084.10	kip-ft	39.53%
SAP2000 MAX M2	1219.00	kip-ft	Capacity

Axial Load Check			
σ	60	ksi	Pass?
ϕP	3564.00	kip	Yes
			18.10%
SAP2000 MAX P	645.00	kip	Capacity

SAP2000 Deflection: Uy about 3.5 inches in arch



Tapered Section

Tapered Section		
b	21.00	in
d	21.00	in
t	0.50	in
Area	41	in^2
I_3	2873.4	in^4
I_2	2873.4	in^4

Designed just for axial load
Tapered section will not see moment .

Moment Check			
σ	60	ksi	
E	29000	ksi	Pass?
φM3	14777.57	kip-in	Yes
	1231.46	kip-ft	88.76%
SAP2000 MAX M3	1093.00	kip-ft	Capacity
φM2	14777.57	kip-in	Yes
	1231.46	kip-ft	98.99%
SAP2000 MAX M2	1219.00	kip-ft	Capacity

Max Moment from fully loaded case

Axial Load Check			
σ	60	ksi	Pass?
φP	2214.00	kip	Yes
			29.13%
SAP2000 MAX P	645.00	kip	Capacity

Axial Load Check			
σ	60	ksi	Pass?
φP	2214.00	kip	Yes
			24.71%
SAP2000 MAX P	547.00	kip	Capacity

Deflection not a problem at base, so can use smaller section

APPENDIX H: SIMPLE SAP2000 MODEL

South Cable Connections

Axis Along Deck		Axis Along Arch	
Detail	X	X'	Y'
foundation center	0.0	0.5	2.5
foundation end	2.5	2.9	2.0
cable 1	17.5	17.7	-1.0
cable 2	32.5	32.4	-3.9
cable 3	47.5	47.1	-6.9
cable 4	62.5	61.8	-9.8
cable 5	77.5	76.5	-12.7
cable 6	92.5	91.2	-15.7
cable 7	107.5	105.9	-18.6
foundation start	122.5	120.6	-21.6
foundation center	125.0	123.1	-22.1

Spacing of Cables on Arch:	8	ft
Downward load on Cable:	47	kips *
Length of Arch:	127.5	ft

*47 kips was calculated based on assumed deck area (actual deck unknown at this point)

Connections to Arch							
X'	Y'	Z	Cable #	Length	Theta	Alpha	Tension
39.75	0	27.36	1	35.182	0.891	0.680	60.443
47.75	0	29.87	2	33.83	1.08	0.49	53.23
55.75	0	31.37	3	33.27	1.23	0.34	49.84
63.75	0	31.88	4	33.41	1.27	0.30	49.26
71.75	0	31.37	5	34.19	1.16	0.41	51.23
79.75	0	29.87	6	35.63	0.99	0.58	56.06
87.75	0	27.36	7	37.75	0.81	0.76	64.85

				Projected	Break Tention X'' into X' and Y'			
Cable	delta X'	Alpha	Tension	Tension X''	Tension Z	Gamma	Tension X'	Tension Y'
1	22.10	0.68	60.44	38.00	47	0.04	37.97	1.68
2	15.39	0.49	53.23	24.99	47	0.25	24.22	6.17
3	8.68	0.34	49.84	16.58	47	0.67	13.01	10.28
4	1.97	0.30	49.26	14.75	47	1.37	2.91	14.46
5	-4.74	0.41	51.23	20.37	47	1.22	-7.09	19.10
6	-11.44	0.58	56.06	30.56	47	0.94	-18.01	24.69
7	-18.15	0.76	64.85	44.69	47	0.80	-31.19	32.01

South and North Cables Summary

South						
Cable	Tension Z	Arm Z	Tension X'	Arm X'	Tension Y'	Arm Y'
1	47	39.75	37.97	27.36	1.68	39.75
2	47	47.75	24.22	29.87	6.17	47.75
3	47	55.75	13.01	31.37	10.28	55.75
4	47	63.75	2.91	31.88	14.46	63.75
5	47	71.75	-7.09	31.37	19.10	71.75
6	47	79.75	-18.01	29.87	24.69	79.75
7	47	87.75	-31.19	27.36	32.01	87.75

North						
Cable	Tension Z	Arm Z	Tension X'	Arm X'	Tension Y'	Arm Y'
1	47	39.75	31.23	27.36	-32.01	39.75
2	47	47.75	18.05	29.87	-24.69	47.75
3	47	55.75	7.13	31.37	-19.10	55.75
4	47	63.75	-2.87	31.88	-14.46	63.75
5	47	71.75	-12.97	31.37	-10.28	71.75
6	47	79.75	-24.18	29.87	-6.17	79.75
7	47	87.75	-37.92	27.36	-1.68	87.75

APPENDIX I: WIND LOAD CALCULATION

v (mph)	140
K_d	0.85
exposure	c
K_{zt}	1
G	0.86
GC_{pi}	0
L	20.00
B	125.00
L/B	0.16
h	12.00
Windward	
C_p	0.80
K_z	0.85
q_z	36.25
p (psf)	24.84
Leeward	
C_p	-0.50
K_z	0.85
q_z	36.25
p (psf)	-15.52

gq	3.4
gv	3.4
z	12
c	0.2
Iz	0.236729
ϵ	0.2
l	500
Lz	408.4167
Q	0.871519

Wind Load as Distributed		
Windward	0.014	k/ft
Leeward	-0.009	k/ft
Wind Load as Point Loads		
Windward End	0.108	kip
Windward Middle	0.216	kip
Leeward End	-0.068	kip
Leeward Middle	-0.135	kip

APPENDIX J: NON-UNIFORM LOADS

Single Arch Non-Uniform Loading			
Case	Maximum Values		
	U1	U2	U3
	in	in	in
1.2D+1.0W+0.5L	0.29	1.00	1.47
1.2D+1.0W+0.5CaseA	0.29	1.00	1.47
1.2D+1.0W+0.5CaseB	0.29	1.25	1.46
1.2D+1.0W+0.5CaseC	0.31	1.39	1.61
1.2D+1.0W+0.5CaseD	0.40	1.09	1.35
1.2D+1.0W+0.5CaseE	0.21	0.78	1.15
1.2D+1.0W+0.5CaseF	0.24	0.74	1.13
1.2D+1.0CaseG+0.5CaseA	0.28	1.25	1.46
1.2D+1.0CaseG+0.5CaseB	0.31	1.39	1.61
1.2D+1.0CaseG+0.5CaseC	0.43	1.21	1.49
1.2D+1.0CaseG+0.5CaseD	0.40	1.09	1.35
1.2D+1.0CaseG+0.5CaseE	0.21	0.78	1.15
1.2D+1.0CaseG+0.5CaseF	0.24	0.75	1.13
1.2D+1.0CaseH+0.5CaseA	0.28	1.25	1.46
1.2D+1.0CaseH+0.5CaseB	0.31	1.39	1.61
1.2D+1.0CaseH+0.5CaseC	0.43	1.21	1.49
1.2D+1.0CaseH+0.5CaseD	0.40	1.09	1.35
1.2D+1.0CaseH+0.5CaseE	0.21	0.78	1.15
1.2D+1.0CaseH+0.5CaseF	0.24	0.75	1.13
1.2D+1.6L	0.59	2.05	2.99
1.2D+1.6CaseA	0.66	3.22	3.31
1.2D+1.6CaseB	0.73	3.48	3.63
1.2D+1.6CaseC	1.06	2.79	3.52
1.2D+1.6CaseD	1.01	2.54	3.20
1.2D+1.6CaseE	0.33	1.34	1.96
1.2D+1.6CaseF	0.44	1.23	1.90
Overall MAX	1.06	3.48	3.63
Allowable Deflection	4.17	4.17	4.17

Full Bridge Non-Uniform Loading

Case	Maximum Values		
	U1	U2	U3
	in	in	in
1.2D+1.6L	0.64	2.24	3.07
1.2D+1.0W+0.5L	0.31	1.07	1.49
1.2D+1.6L(middle)	0.55	1.83	2.96
1.2D+1.6L(ends)	0.56	2.04	2.99
1.2D+1.0W+0.5L(middle)	0.27	0.90	1.46
1.2D+1.0W+0.5L(ends)	0.28	1.02	1.47
Overall MAX	0.64	2.24	3.07
Allowable Deflection	4.17	4.17	4.17

APPENDIX K: SPECTRAL DISPLACEMENT

Spectral Displacement Response

Mode #	Frequency	Period	S_d	Shape Factor			Actual Displacement		
				Max ϕ_x	Max ϕ_y	Max ϕ_z	Max U_x	Max U_y	Max U_z
	Cyc/sec	Sec	in	in	in	in	in	in	in
1	2.59	0.39	0.14	2.81	12.72	9.65	0.19	1.80	1.37
2	3.71	0.27	0.10	0.71	8.03	2.24	0.03	0.79	0.22
3	4.39	0.23	0.08	1.93	7.14	7.87	0.07	0.59	0.65
4	4.85	0.21	0.07	4.64	6.24	11.37	0.16	0.46	0.84
5	6.62	0.15	0.05	1.52	9.33	10.02	0.04	0.50	0.53
6	6.85	0.15	0.05	1.38	8.06	9.83	0.03	0.41	0.50
7	8.92	0.11	0.04	0.91	7.44	3.94	0.02	0.29	0.15
8	9.32	0.11	0.04	1.32	3.64	9.56	0.02	0.13	0.35
9	9.69	0.10	0.04	1.79	15.76	5.75	0.03	0.55	0.20
10	10.14	0.10	0.03	4.75	2.19	10.67	0.07	0.07	0.36
11	12.56	0.08	0.03	1.51	1.81	9.47	0.02	0.05	0.25
12	14.50	0.07	0.02	1.71	6.33	10.92	0.02	0.14	0.24
13	15.65	0.06	0.02	1.97	9.72	3.26	0.02	0.20	0.07
14	15.79	0.06	0.02	1.61	1.27	11.84	0.02	0.03	0.24
15	16.84	0.06	0.02	3.39	12.19	7.77	0.03	0.23	0.14
16	17.78	0.06	0.02	1.76	2.08	15.79	0.01	0.04	0.28
17	18.43	0.05	0.02	1.35	0.69	9.82	0.01	0.01	0.16
18	20.27	0.05	0.01	1.96	1.69	14.85	0.01	0.03	0.22
19	22.59	0.04	0.01	2.39	1.83	17.07	0.01	0.02	0.22
20	23.52	0.04	0.01	3.79	8.54	11.78	0.02	0.11	0.14

USGS Spectral Displacement

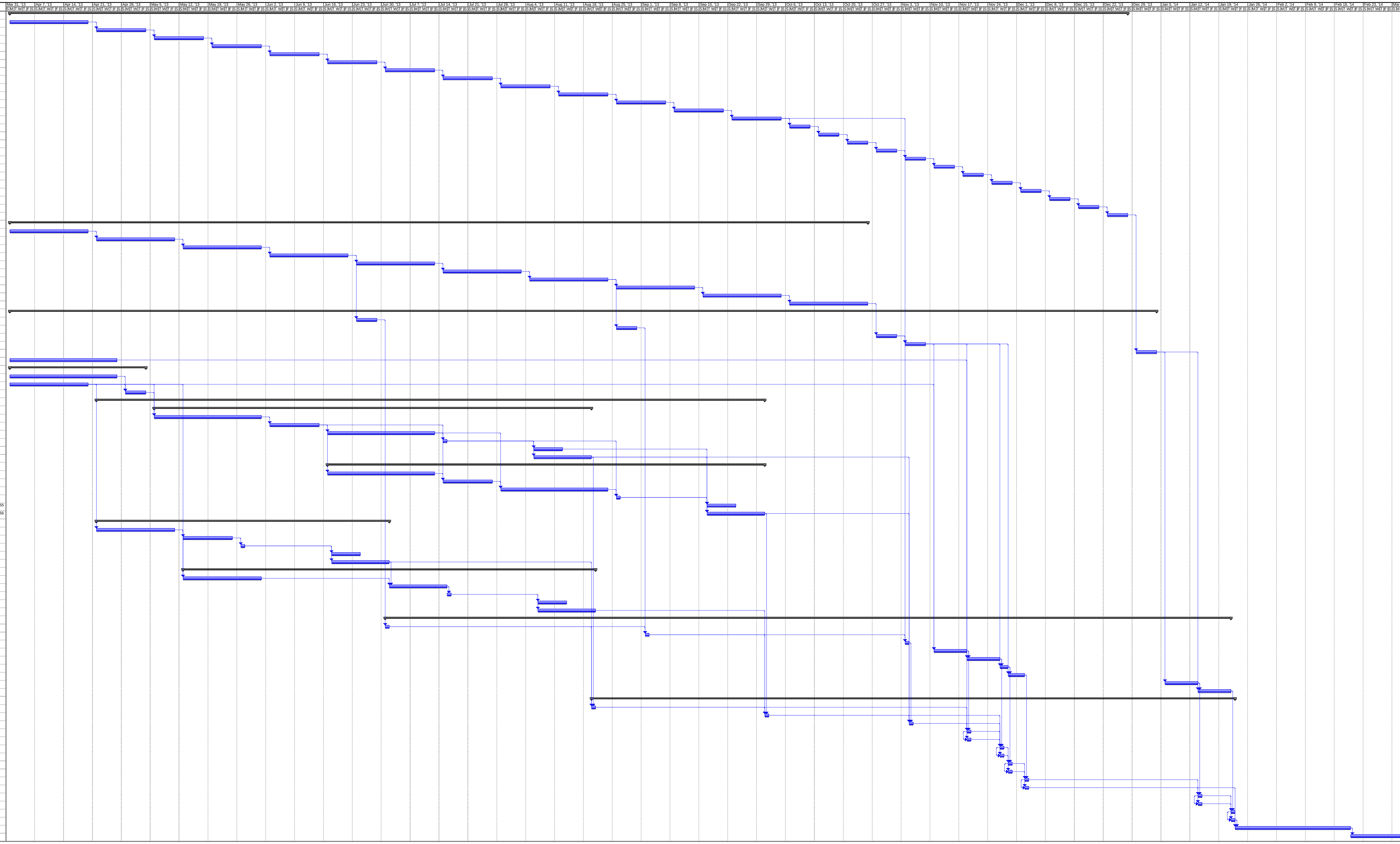
$$\boxed{S_d = 0.3771T - 0.0037}$$

Modal Participation Factors	
UX	46.7%
UY	99.9%
UZ	99.7%

**ALL DEFLECTIONS WELL BELOW DEFLECTION LIMIT

APPENDIX L: CONSTRUCTION SCHEDULES

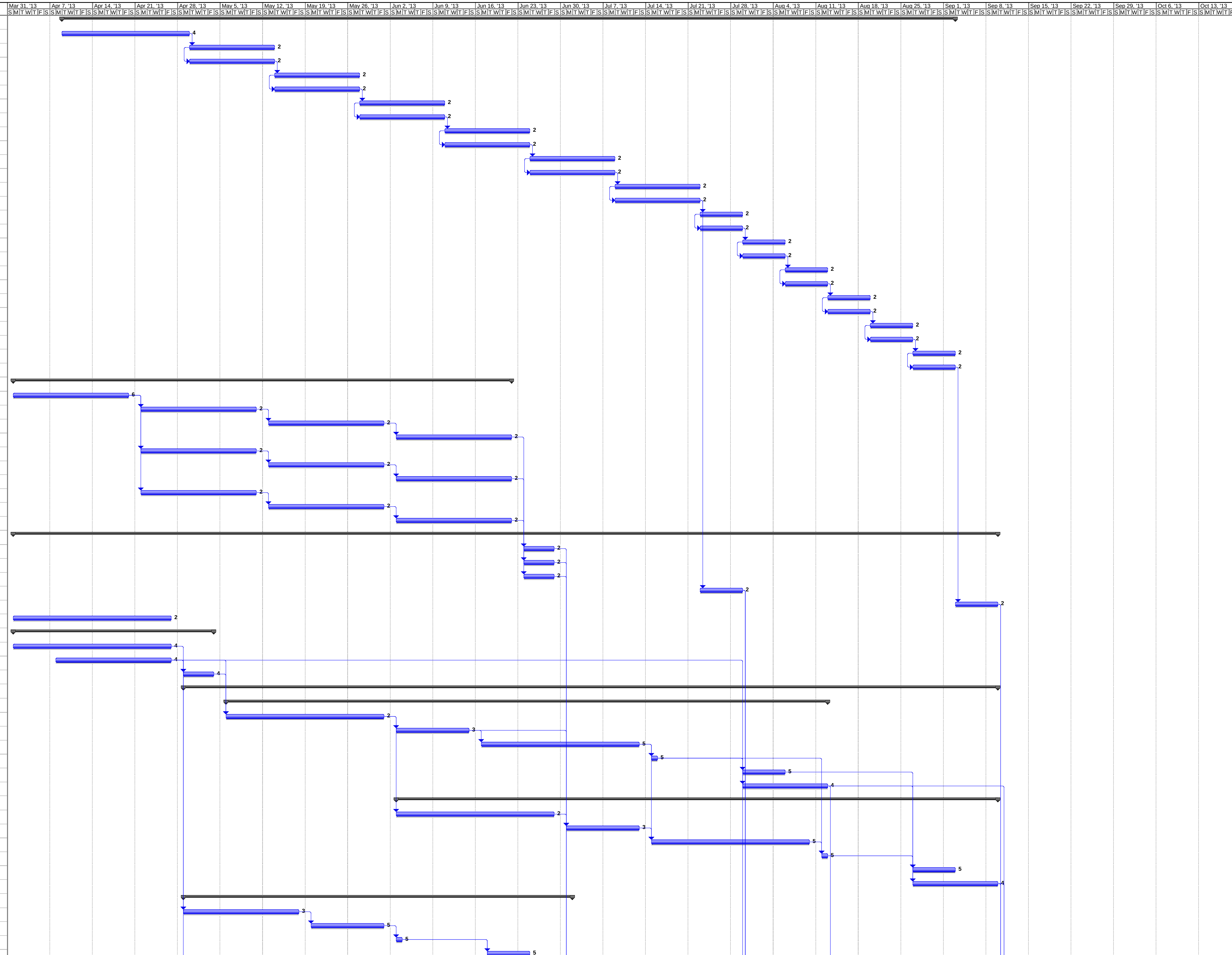
ID		Task Name	Duration	Start	Finish	Predecessors
1		**Deck Fabrication**	**195 days**	**Mon 4/1/13**	**Fri 12/27/13**	
2		Set Up Fab Lines	15 days	Mon 4/1/13	Fri 4/19/13	
3		Deck Section 1-1	10 days	Mon 4/22/13	Fri 5/3/13	2
4		Deck Section 1-2	10 days	Mon 5/6/13	Fri 5/17/13	3
5		Deck Section 1-3	10 days	Mon 5/20/13	Fri 5/31/13	4
6		Deck Section 1-4	10 days	Mon 6/3/13	Fri 6/14/13	5
7		Deck Section 1-5	10 days	Mon 6/17/13	Fri 6/28/13	6
8		Deck Section 1-6	10 days	Mon 7/1/13	Fri 7/12/13	7
9		Deck Section 1-7	10 days	Mon 7/15/13	Fri 7/26/13	8
10		Deck Section 1-8	10 days	Mon 7/29/13	Fri 8/9/13	9
11		Deck Section 2-1	10 days	Mon 8/12/13	Fri 8/23/13	10
12		Deck Section 2-2	10 days	Mon 8/26/13	Fri 9/6/13	11
13		Deck Section 2-3	10 days	Mon 9/9/13	Fri 9/20/13	12
14		Deck Section 2-4	10 days	Mon 9/23/13	Fri 10/4/13	13
15		Deck Section 2-5	5 days	Mon 10/7/13	Fri 10/11/13	14
16		Deck Section 2-6	5 days	Mon 10/14/13	Fri 10/18/13	15
17		Deck Section 2-7	5 days	Mon 10/21/13	Fri 10/25/13	16
18		Deck Section 2-8	5 days	Mon 10/28/13	Fri 11/1/13	17
19		Deck Section 3-1	5 days	Mon 11/4/13	Fri 11/8/13	18
20		Deck Section 3-2	5 days	Mon 11/11/13	Fri 11/15/13	19
21		Deck Section 3-3	5 days	Mon 11/18/13	Fri 11/22/13	20
22		Deck Section 3-4	5 days	Mon 11/25/13	Fri 11/29/13	21
23		Deck Section 3-5	5 days	Mon 12/2/13	Fri 12/6/13	22
24		Deck Section 3-6	5 days	Mon 12/9/13	Fri 12/13/13	23
25		Deck Section 3-7	5 days	Mon 12/16/13	Fri 12/20/13	24
26		Deck Section 3-8	5 days	Mon 12/23/13	Fri 12/27/13	25
27		**Arch Fabrication**	**150 days**	**Mon 4/1/13**	**Fri 10/25/13**	
28		Set Up Fab Line	15 days	Mon 4/1/13	Fri 4/19/13	
29		Arch Section 1-1	15 days	Mon 4/22/13	Fri 5/10/13	28
30		Arch Section 2-1	15 days	Mon 5/13/13	Fri 5/31/13	29
31		Arch Section 3-1	15 days	Mon 6/3/13	Fri 6/21/13	30
32		Arch Section 1-2	15 days	Mon 6/24/13	Fri 7/12/13	31
33		Arch Section 2-2	15 days	Mon 7/15/13	Fri 8/2/13	32
34		Arch Section 3-2	15 days	Mon 8/5/13	Fri 8/23/13	33
35		Arch Section 1-3	15 days	Mon 8/26/13	Fri 9/13/13	34
36		Arch Section 2-3	15 days	Mon 9/16/13	Fri 10/4/13	35
37		Arch Section 3-3	15 days	Mon 10/7/13	Fri 10/25/13	36
38		**Delivery**	**200 days**	**Mon 4/1/13**	**Fri 1/3/14**	
39		Arch 1 Delivery	5 days	Mon 6/24/13	Fri 6/28/13	31
40		Arch 2 Delivery	5 days	Mon 8/26/13	Fri 8/30/13	34
41		Arch 3 Delivery	5 days	Mon 10/28/13	Fri 11/1/13	37
42		Deck Cantilever Sections	5 days	Mon 11/4/13	Fri 11/8/13	14,41
43		Deck Mid Sections	5 days	Mon 12/30/13	Fri 1/3/14	26
44		Cable Delivery	20 days	Mon 4/1/13	Fri 4/26/13	
45		**Mobilize**	**25 days**	**Mon 4/1/13**	**Fri 5/3/13**	
46		Move Barge	20 days	Mon 4/1/13	Fri 4/26/13	
47		All Other Equipment	15 days	Mon 4/1/13	Fri 4/19/13	
48		Erect Crane	5 days	Mon 4/29/13	Fri 5/3/13	46,47
49		**Foundations**	**116 days**	**Mon 4/22/13**	**Mon 9/30/13**	
50		**Pier 1**	**76 days**	**Mon 5/6/13**	**Mon 8/19/13**	
51		Coffer Dam 1	20 days	Mon 5/6/13	Fri 5/31/13	47,48
52		Excavation Pier 1	10 days	Mon 6/3/13	Fri 6/14/13	51
53		Form Work/Rebar Pie	20 days	Mon 6/17/13	Fri 7/12/13	52
54		Pour Concrete Pier 1	1 day	Mon 7/15/13	Mon 7/15/13	53
55		Remove Form Work P	5 days	Tue 8/6/13	Mon 8/12/13	54FS+15 days
56		Prepare Bearings Pier	10 days	Tue 8/6/13	Mon 8/19/13	54FS+15 days
57		**Pier 2**	**76 days**	**Mon 6/17/13**	**Mon 9/30/13**	
58		Coffer Dam 2	20 days	Mon 6/17/13	Fri 7/12/13	52
59		Excavation Pier 2	10 days	Mon 7/15/13	Fri 7/26/13	58,52
60		Form Work/Rebar Pie	20 days	Mon 7/29/13	Fri 8/23/13	59,53
61		Pour Concrete Pier 2	1 day	Mon 8/26/13	Mon 8/26/13	54,60
62		Remove Form Work P	5 days	Tue 9/17/13	Mon 9/23/13	61FS+15 days,55
63		Prepare Bearings Pier	10 days	Tue 9/17/13	Mon 9/30/13	61FS+15 days,56
64		**North Shore Foundation**	**51 days**	**Mon 4/22/13**	**Mon 7/1/13**	
65		North Shore Foundatio	15 days	Mon 4/22/13	Fri 5/10/13	47
66		Form Work/Rebar Nor	10 days	Mon 5/13/13	Fri 5/24/13	65
67		Pour Concrete North S	1 day	Mon 5/27/13	Mon 5/27/13	66
68		Remove Form Work N	5 days	Tue 6/18/13	Mon 6/24/13	67FS+15 days
69		Prepare Bearings Nor	10 days	Tue 6/18/13	Mon 7/1/13	67FS+15 days
70		**South Shore Foundation**	**72 days**	**Mon 5/13/13**	**Tue 8/20/13**	
71		South Shore Foundati	15 days	Mon 5/13/13	Fri 5/31/13	47,65
72		Form Work/Rebar Sou	10 days	Tue 7/2/13	Mon 7/15/13	69,71
73		Pour Concrete South S	1 day	Tue 7/16/13	Tue 7/16/13	72
74		Remove Form Work S	5 days	Wed 8/7/13	Tue 8/13/13	73FS+15 days
75		Prepare Bearings Sou	10 days	Wed 8/7/13	Tue 8/20/13	73FS+15 days
76		**On Site Assembly**	**147 days**	**Mon 7/1/13**	**Tue 1/21/14**	
77		Assemble Arch 1	1 day	Mon 7/1/13	Mon 7/1/13	39
78		Assemble Arch 2	1 day	Mon 9/2/13	Mon 9/2/13	40,77
79		Assemble Arch 3	1 day	Mon 11/4/13	Mon 11/4/13	41,78
80		Assemble Cantilever Pier 1	6 days	Mon 11/11/13	Mon 11/18/13	42,47
81		Assemble Cantilever Pier 2	6 days	Tue 11/19/13	Tue 11/26/13	42,80
82		Assemble Cantilever North	2 days	Wed 11/27/13	Thu 11/28/13	42,81
83		Assemble Cantilever South	2 days	Fri 11/29/13	Mon 12/2/13	42,82
84		Assemble North Mid Span	6 days	Mon 1/6/14	Mon 1/13/14	43
85		Assemble South Mid Span	6 days	Tue 1/14/14	Tue 1/21/14	43,84
86		**Lifts**	**112 days**	**Tue 8/20/13**	**Wed 1/22/14**	
87		Lift Arch 1	1 day	Tue 8/20/13	Tue 8/20/13	77,56,69
88		Lift Arch 2	1 day	Tue 10/1/13	Tue 10/1/13	78,63,75,87
89		Lift Arch 3	1 day	Tue 11/5/13	Tue 11/5/13	79,56,63,88
90		Lift Deck Sections Pier 1	1 day	Tue 11/19/13	Tue 11/19/13	80,87,89
91		Connect Cables Deck Sect	1 day	Tue 11/19/13	Tue 11/19/13	44,90SS
92		Lift Deck Sections Pier 2	1 day	Wed 11/27/13	Wed 11/27/13	81,88,89,90
93		Connect Cables Deck Sect	1 day	Wed 11/27/13	Wed 11/27/13	92SS,91
94		Lift Deck Sections North Sh	1 day	Fri 11/29/13	Fri 11/29/13	82,92
95		Connect Cables Deck Sect	1 day	Fri 11/29/13	Fri 11/29/13	94SS,93
96		Lift Deck Sections South Sl	1 day	Tue 12/3/13	Tue 12/3/13	83,94
97		Connect Cables Deck Sect	1 day	Tue 12/3/13	Tue 12/3/13	96SS,95
98		Lift Deck Sections North Mi	1 day	Tue 1/14/14	Tue 1/14/14	84,96
99		Connect Cables Deck Sect	1 day	Tue 1/14/14	Tue 1/14/14	98SS,97
100		Lift Deck Sections South M	1 day	Wed 1/22/14	Wed 1/22/14	85,98
101		Connect Cables Deck Sect	1 day	Wed 1/22/14	Wed 1/22/14	100SS,99
102		Finish Arches	20 days	Thu 1/23/14	Wed 2/19/14	97,101
103		Hand Rails	10 days	Thu 2/20/14	Wed 3/5/14	102



Project: Arch Bridge slow
Date: Thu 4/5/12

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

ID	Task Name	Duration	Start	Finish	Predecessors
1	**Deck Fabrication**	**105 days**	**Tue 4/9/13**	**Mon 9/2/13**	
2	Set Up Fab Lines	15 days	Tue 4/9/13	Mon 4/29/13	
3	Deck Section 1-1	10 days	Tue 4/30/13	Mon 5/13/13	2
4	Deck Section 1-2	10 days	Tue 4/30/13	Mon 5/13/13	3SS
5	Deck Section 1-3	10 days	Tue 5/14/13	Mon 5/27/13	4
6	Deck Section 1-4	10 days	Tue 5/14/13	Mon 5/27/13	5SS
7	Deck Section 1-5	10 days	Tue 5/28/13	Mon 6/10/13	6
8	Deck Section 1-6	10 days	Tue 5/28/13	Mon 6/10/13	7SS
9	Deck Section 1-7	10 days	Tue 6/11/13	Mon 6/24/13	8
10	Deck Section 1-8	10 days	Tue 6/11/13	Mon 6/24/13	9SS
11	Deck Section 2-1	10 days	Tue 6/25/13	Mon 7/8/13	10
12	Deck Section 2-2	10 days	Tue 6/25/13	Mon 7/8/13	11SS
13	Deck Section 2-3	10 days	Tue 7/9/13	Mon 7/22/13	12
14	Deck Section 2-4	10 days	Tue 7/9/13	Mon 7/22/13	13SS
15	Deck Section 2-5	5 days	Tue 7/23/13	Mon 7/29/13	14
16	Deck Section 2-6	5 days	Tue 7/23/13	Mon 7/29/13	15SS
17	Deck Section 2-7	5 days	Tue 7/30/13	Mon 8/5/13	16
18	Deck Section 2-8	5 days	Tue 7/30/13	Mon 8/5/13	17SS
19	Deck Section 3-1	5 days	Tue 8/6/13	Mon 8/12/13	18
20	Deck Section 3-2	5 days	Tue 8/6/13	Mon 8/12/13	19SS
21	Deck Section 3-3	5 days	Tue 8/13/13	Mon 8/19/13	20
22	Deck Section 3-4	5 days	Tue 8/13/13	Mon 8/19/13	21SS
23	Deck Section 3-5	5 days	Tue 8/20/13	Mon 8/26/13	22
24	Deck Section 3-6	5 days	Tue 8/20/13	Mon 8/26/13	23SS
25	Deck Section 3-7	5 days	Tue 8/27/13	Mon 9/2/13	24
26	Deck Section 3-8	5 days	Tue 8/27/13	Mon 9/2/13	25SS
27	**Arch Fabrication**	**60 days**	**Mon 4/1/13**	**Fri 6/21/13**	
28	Set Up Fab Line	15 days	Mon 4/1/13	Fri 4/19/13	
29	Arch Section 1-1	15 days	Mon 4/22/13	Fri 5/10/13	28
30	Arch Section 2-1	15 days	Mon 5/13/13	Fri 5/31/13	29
31	Arch Section 3-1	15 days	Mon 6/3/13	Fri 6/21/13	30
32	Arch Section 1-2	15 days	Mon 4/22/13	Fri 5/10/13	28
33	Arch Section 2-2	15 days	Mon 5/13/13	Fri 5/31/13	32
34	Arch Section 3-2	15 days	Mon 6/3/13	Fri 6/21/13	33
35	Arch Section 1-3	15 days	Mon 4/22/13	Fri 5/10/13	28
36	Arch Section 2-3	15 days	Mon 5/13/13	Fri 5/31/13	35
37	Arch Section 3-3	15 days	Mon 6/3/13	Fri 6/21/13	36
38	**Delivery**	**116 days**	**Mon 4/1/13**	**Mon 9/9/13**	
39	Arch 1 Delivery	5 days	Mon 6/24/13	Fri 6/28/13	31
40	Arch 2 Delivery	5 days	Mon 6/24/13	Fri 6/28/13	34
41	Arch 3 Delivery	5 days	Mon 6/24/13	Fri 6/28/13	37
42	Deck Cantilever Sections	5 days	Tue 7/23/13	Mon 7/29/13	14
43	Deck Mid Sections	5 days	Tue 9/3/13	Mon 9/9/13	26
44	Cable Delivery	20 days	Mon 4/1/13	Fri 4/26/13	
45	**Mobilize**	**25 days**	**Mon 4/1/13**	**Fri 5/3/13**	
46	Move Barge	20 days	Mon 4/1/13	Fri 4/26/13	
47	All Other Equipment	15 days	Mon 4/8/13	Fri 4/26/13	
48	Erect Crane	5 days	Mon 4/29/13	Fri 5/3/13	46,47
49	**Foundations**	**96 days**	**Mon 4/29/13**	**Mon 9/9/13**	
50	**Pier 1**	**71 days**	**Mon 5/6/13**	**Mon 8/12/13**	
51	Coffer Dam 1	20 days	Mon 5/6/13	Fri 5/31/13	47,48
52	Excavation Pier 1	10 days	Mon 6/3/13	Fri 6/14/13	51
53	Form Work/Rebar Pier 1	20 days	Mon 6/17/13	Fri 7/12/13	52
54	Pour Concrete Pier 1	1 day	Mon 7/15/13	Mon 7/15/13	53
55	Remove Form Work Pier 1	5 days	Tue 7/30/13	Mon 8/5/13	54FS+10 days
56	Prepare Bearings Pier 1	10 days	Tue 7/30/13	Mon 8/12/13	54FS+10 days
57	**Pier 2**	**71 days**	**Mon 6/3/13**	**Mon 9/9/13**	
58	Coffer Dam 2	20 days	Mon 6/3/13	Fri 6/28/13	51
59	Excavation Pier 2	10 days	Mon 7/1/13	Fri 7/12/13	58,52
60	Form Work/Rebar Pier 2	20 days	Mon 7/15/13	Fri 8/9/13	59,53
61	Pour Concrete Pier 2	1 day	Mon 8/12/13	Mon 8/12/13	54,60
62	Remove Form Work Pier 2	5 days	Tue 8/27/13	Mon 9/2/13	61FS+10 days,55
63	Prepare Bearings Pier 2	10 days	Tue 8/27/13	Mon 9/9/13	61FS+10 days,56
64	**North Shore Foundation**	**46 days**	**Mon 4/29/13**	**Mon 7/1/13**	
65	North Shore Foundation Prep	15 days	Mon 4/29/13	Fri 5/17/13	47
66	Form Work/Rebar North Shore	10 days	Mon 5/20/13	Fri 5/31/13	65
67	Pour Concrete North Shore	1 day	Mon 6/3/13	Mon 6/3/13	66
68	Remove Form Work North Shore	5 days	Tue 6/18/13	Mon 6/24/13	67FS+10 days



Project: Arch Bridge
Date: Thu 4/5/12

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

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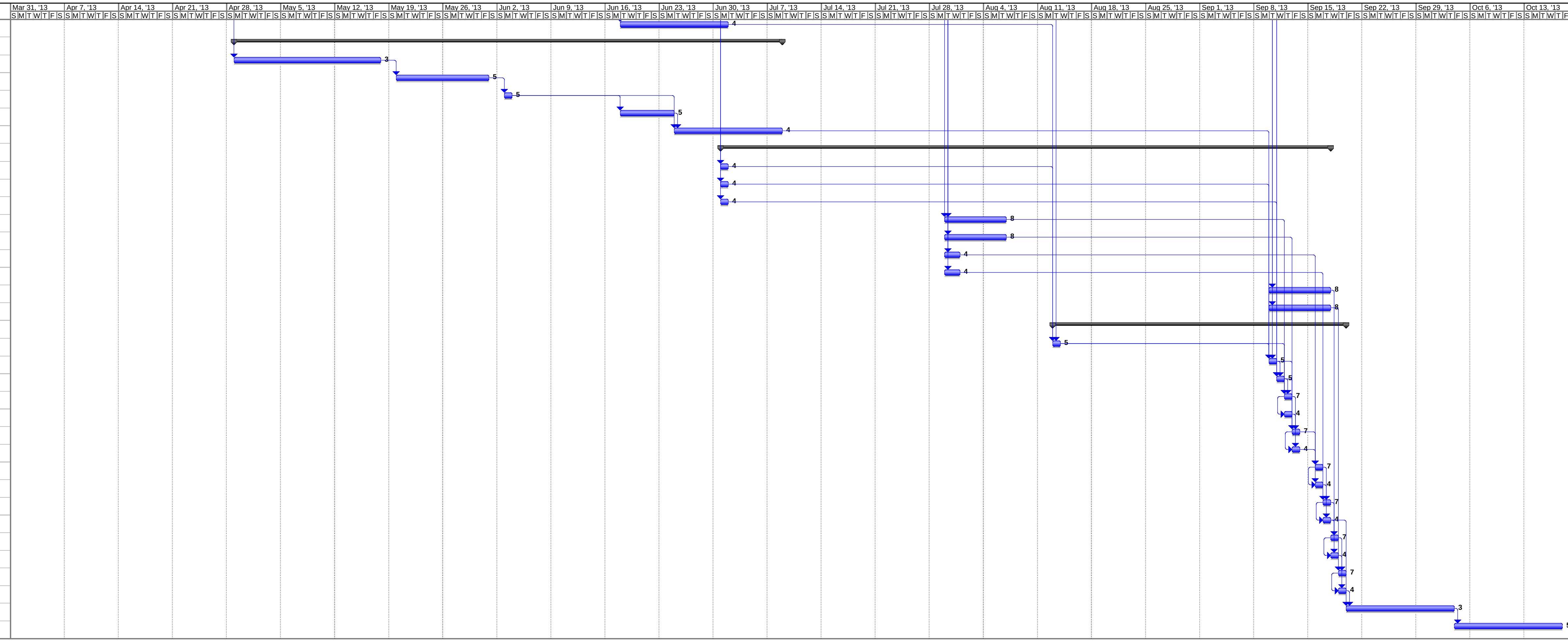
ID		Task Name	Duration	Start	Finish	Predecessors
69		Prepare Bearings North Shore	10 days	Tue 6/18/13	Mon 7/1/13	67FS+10 days
70		**South Shore Foundation**	**51 days**	**Mon 4/29/13**	**Mon 7/8/13**	
71		South Shore Foundation Prep	15 days	Mon 4/29/13	Fri 5/17/13	47
72		Form Work/Rebar South Shore	10 days	Mon 5/20/13	Fri 5/31/13	71
73		Pour Concrete South Shore	1 day	Mon 6/3/13	Mon 6/3/13	72
74		Remove Form Work South Shore	5 days	Tue 6/18/13	Mon 6/24/13	73FS+10 days
75		Prepare Bearings South Shore	10 days	Tue 6/25/13	Mon 7/8/13	73FS+10 days,74
76		**On Site Assembly**	**57 days**	**Mon 7/1/13**	**Tue 9/17/13**	
77		Assemble Arch 1	1 day	Mon 7/1/13	Mon 7/1/13	39
78		Assemble Arch 2	1 day	Mon 7/1/13	Mon 7/1/13	40
79		Assemble Arch 3	1 day	Mon 7/1/13	Mon 7/1/13	41
80		Assemble Cantilever Pier 1	6 days	Tue 7/30/13	Tue 8/6/13	42,47
81		Assemble Cantilever Pier 2	6 days	Tue 7/30/13	Tue 8/6/13	42
82		Assemble Cantilever North Shore	2 days	Tue 7/30/13	Wed 7/31/13	42
83		Assemble Cantilever South Shore	2 days	Tue 7/30/13	Wed 7/31/13	42
84		Assemble North Mid Span	6 days	Tue 9/10/13	Tue 9/17/13	43
85		Assemble South Mid Span	6 days	Tue 9/10/13	Tue 9/17/13	43
86		**Lifts**	**28 days**	**Tue 8/13/13**	**Thu 9/19/13**	
87		Lift Arch 1	1 day	Tue 8/13/13	Tue 8/13/13	77,56,69
88		Lift Arch 2	1 day	Tue 9/10/13	Tue 9/10/13	78,63,75,87
89		Lift Arch 3	1 day	Wed 9/11/13	Wed 9/11/13	79,56,63,88
90		Lift Deck Sections Pier 1	1 day	Thu 9/12/13	Thu 9/12/13	80,87,89
91		Connect Cables Deck Sections Pier 1	1 day	Thu 9/12/13	Thu 9/12/13	90SS
92		Lift Deck Sections Pier 2	1 day	Fri 9/13/13	Fri 9/13/13	81,88,89,90
93		Connect Cables Deck Sections Pier 2	1 day	Fri 9/13/13	Fri 9/13/13	92SS,91
94		Lift Deck Sections North Shore	1 day	Mon 9/16/13	Mon 9/16/13	82,92
95		Connect Cables Deck Sections North Shore	1 day	Mon 9/16/13	Mon 9/16/13	94SS,93
96		Lift Deck Sections South Shore	1 day	Tue 9/17/13	Tue 9/17/13	83,94
97		Connect Cables Deck Sections South Shore	1 day	Tue 9/17/13	Tue 9/17/13	96SS,95
98		Lift Deck Sections North Mid Span	1 day	Wed 9/18/13	Wed 9/18/13	84,96
99		Connect Cables Deck Sections North Mid Span	1 day	Wed 9/18/13	Wed 9/18/13	98SS,97
100		Lift Deck Sections South Mid Span	1 day	Thu 9/19/13	Thu 9/19/13	85,98
101		Connect Cables Deck Sections South Mid Span	1 day	Thu 9/19/13	Thu 9/19/13	100SS,99
102		Finish Arches	10 days	Fri 9/20/13	Thu 10/3/13	97,101
103		Hand Rails	10 days	Fri 10/4/13	Thu 10/17/13	102



Project: Arch Bridge
Date: Thu 4/5/12

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

APPENDIX M: BRIDGE COST CALCULATIONS

Labor Cost Estimate	
Union Worker	75 $/hr
Hours/Day	10 hrs

Workers	Service	Total Days	Total Man Days	Total Man Hours
8	Deck Assembly	120	960	9600
8	Cantilevered Assembly	2	16	160
4	Cantilevered Assembly	4	16	160
2	Coffer Dam	40	80	800
3	Excavator	20	60	600
5	Rebar Men	40	200	2000
5	Concrete Pour	2	10	100
5	Remove formwork	10	50	500
4	Bearings	20	80	800
3	Side Excavation	30	90	900
5	Side Rebar	20	100	1000
5	Side Concrete Pour	2	10	100
5	Side Remove formwork	10	50	500
4	Side Bearings	20	80	800
5	Arch Lifts	3	15	150
7	Deck Lifts	7	49	490
3	Arch Swoops	20	60	600
5	Hand Rails	10	50	500

Total Man Days	1976	19760
Total Labor Cost		**$ 1,482,000.00**

NOTE: Cost same for both schedules

Steel Cost Construction

SteelPrice per tonne	1250 $/tonne
	568.18 $/kip
Steel Price Fabricated	994.32 $/kip
	0.99 $/lb
Density of Steel	0.4838 kip/ft^3

	Area (ft^2)	Total Quantity	Length (ft)	Total Volume	Weight (kip)	Price
Plate I Beam	0.322	126	15	608.6	294.4	$ 167,290
	0.322	42	17.5	236.7	114.5	$ 65,057
End Plate Channel	0.1671	36	15	90.2	43.7	$ 24,804
	0.1671	12	17.5	35.1	17.0	$ 9,646
Cross Beam Stiffeners	0.0052	25	20	2.6	1.3	$ 715
Girder	0.1875	25	20	93.8	45.4	$ 25,771
Arch (Tapered)	0.1598	6	19.1372	18.3	8.9	$ 5,044
Arch	0.4583	6	9.7205	26.7	12.9	$ 7,348
	0.4583	6	9.1874	25.3	12.2	$ 6,945
	0.4583	6	8.7376	24.0	11.6	$ 6,605
	0.4583	6	8.3845	23.1	11.2	$ 6,338
	0.4583	6	8.1405	22.4	10.8	$ 6,153
	0.4583	6	8.0157	22.0	10.7	$ 6,059
Cables	0.0218	6	11.1768	1.5	0.7	$ 402
	0.0218	6	19.0662	2.5	1.2	$ 686
	0.0218	6	26.9708	3.5	1.7	$ 970
	0.0218	6	34.488	4.5	2.2	$ 1,240
	0.0218	6	41.5734	5.4	2.6	$ 1,495
	0.0218	6	48.2606	6.3	3.1	$ 1,735
	0.0218	6	54.6128	7.1	3.5	$ 1,964
				Raw Total	**609**	**$ 346,264**
				Fabricated Tota	**609**	**$ 605,963**

Concrete Cost for Foundations

	Volume (ft^3)
Concrete	9600
Cost per yd^3	250
Cost per ft^3	9.26
Total Cost	$ 177,778
Total Cost (including mix)	$ 266,667
Total Material Cost	**$ 872,629**

Equipment Cost				
	$/truck	# Trucks	$	
Delivery	1000	21	$ 21,000.00	
	$/ft	Length	$	
Handrail	100	750	$ 75,000.00	
Fast Track				
	$/hr	Days	Hours	$
Crane	75	141	3384	$ 253,800.00
	$/day	Days		
Barge	1,000	141		$ 141,000.00
Sequential				
	$/hr	Days	Hours	$
Crane	75	206	4944	$ 370,800.00
	$/day	Days		
Barge	1,000	206		$ 206,000.00

Total Equipment		
Fast Track	**$**	**490,800.00**
Sequential	**$**	**672,800.00**

Total Costs

Fast Track	
Material	$ 872,629
Labor	$ 1,482,000
Equipment	$ 490,800
	$ 2,845,429
Sequential	
Material	$ 872,629
Labor	$ 1,482,000
Equipment	$ 672,800
	$ 3,027,429

Appendix N: Underpass Calculations

GEOMETRIC AND MECHANICAL PROPERTIES FOR UNDERPASS

Columns

Design compressive strength for flexural buckling (from Chapter E section E2 in the Manual of Steel Construction):

$$P_u \leq \varphi P_n = \varphi A_g F_{cr}$$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \quad and \quad \lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}$$

The maximum allowable load for the columns is 391kips.

SAP analysis with the entire bridge loaded at the service load (See figure 30) yielded a maximum base reaction of 284kips. (73% of allowable)

Box Girder

B = 12ft

H = 0.5ft

T1= 0.1042ft

T2 = 0.046ft

Area = (12*0.5) – [(12-(2*0.46))*(0.5-(2*0.1042))] = 2.52ft^2

I = $\frac{12*(0.5)^3}{12}$ - $\frac{(12-(2*0.46))*(0.5-(2*0.1042))^3}{12}$ = 0.1004ft^4

Truss

B = 0.0833ft

H = 3.5ft

Area = (0.0833*3.5) = 0.292ft^2

I = $\frac{0.0833*3.5^3}{12}$ = 0.2977in^4

Handrail

R = 0.292ft

Thickness = 0.0125ft

Area = $\pi(0.292)^2$= 0.000491ft^2

I = $\frac{\pi(r^4-(r-t)^4}{4}$ = 0.000913ft^4

ΣAy =(2.53*(0.5/2))+(2*0.292*(0.5+(3.5/2)))+(2*0.000491*(0.5+3.5+0.292))

= 1.95ft

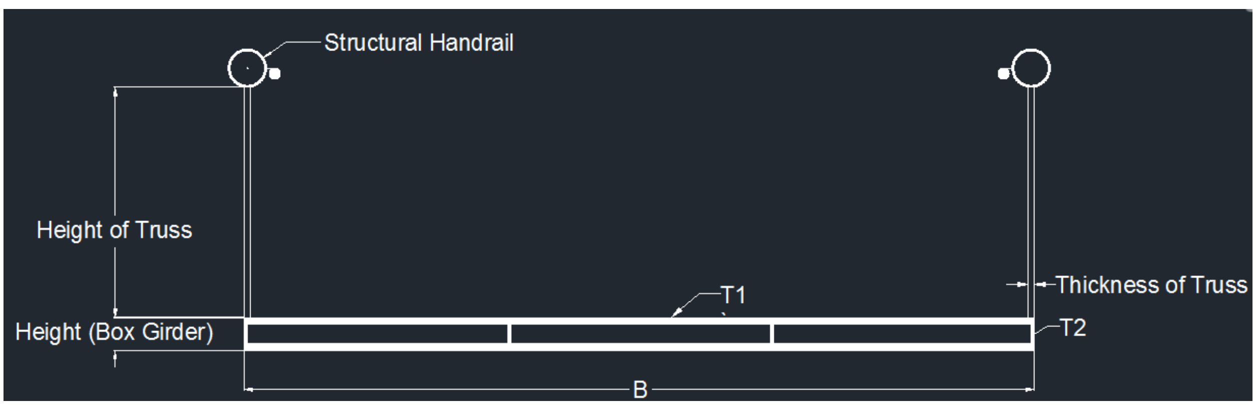
Centroid = $\frac{\sum Ay}{\sum A}$ = 0.69ft *from base of the modified deck*

$I_{modified}$ = $\sum(I + Ad\text{^}2)$ = 1.605ft^4

where d is distance from section centroid to modified deck centroid

Modified Deck Moment Capacity = (I*σ) /C = (8640*1.605)/0.69 = 20074.5K-ft

where σ = 60ksi *144 = 8640ksf, I = 1.605ft^4, C = 0.69ft



GLOBAL TRANSVERSE DEFLECTION

Factored DL, kips/linear ft = (1.2*ρ)+(ΣA) = 0.783kips/ft

ρ is density of steel (0.484 kips/ft^3)

Factored LL, kips/linear ft = 1.6*0.158*12 = 3.034kips/ft

E = 29000ksi

I = $2(I_0 + Ad^2) = 0.00835\ ft^4$ where I_0 is the inertia of the upper and lower flange of the deck with a thickness of 1.25".

L = 12

Deflection Limit = L/360 = 12/360=0.0333ft

Actual Deflection = $\frac{(DL+LL)*(L^4)}{384*E*I}$, *assuming fixed supports for the box girder*

$= \frac{(0.78+3.034)*(12^4)}{384*0.00835*29000*144} = 0.006ft$ < 0.033ft, *OK*

LOCAL TRANSVERSE DEFLECTION OF THE UPPER PLATE OF THE DECK

The same procedure is used but with the moment of inertia is taken as the top flange with a thickness of 1.25".

	Acual Deflection	Limit	Length
	ft	ft	ft
Δ(local)	0,524	0,033	12
	0,033	0,017	6
	0,006	0,011	4

CONNECTIONS

Shear Connection

Using Bolts

Bolt Diameter = 1.25in

Bolt Area = 1.23in^2

Bolt Type = A490

Thread Condition = N

Loading = single

Φ rn = 62.7kips *(ASCE table 7-1)*

Bolt Spacing = 3in

# of bolts in horizontal row = 4

Using table 7-12

Min Edge Distance for 1.25" Diameter bolt = 2.5in

Eccentricity = 6in

# of bolts in vertical row = 2

Coefficient from table = 3.37

Pu, Max shear in structure = 174kips

Rn, nominal design strength of bolts group = 62.7*3.37 = 211.3kips > max shear

Can use 2 vertical rows of 1.25" bolts with 4 @3in

Try Welds, Height of shear plate will need to be 8in to accommodate this configuration while deck height is just 6in

Using Welds

Electrode = E70XX

Fexx, tensile strength of weld = 70 kips/in^2

Fw, ult shearing stress = 42kips

Using "C" weld connections (table 8.8)

L = 5in

B = 1.5in

Y, center of gravity = $\frac{1.5(5)+5(2.5)}{8}$ = 2.5in

X, Center of gravity = $\frac{1.5(0.75)(2)}{8}$ = 0.28in

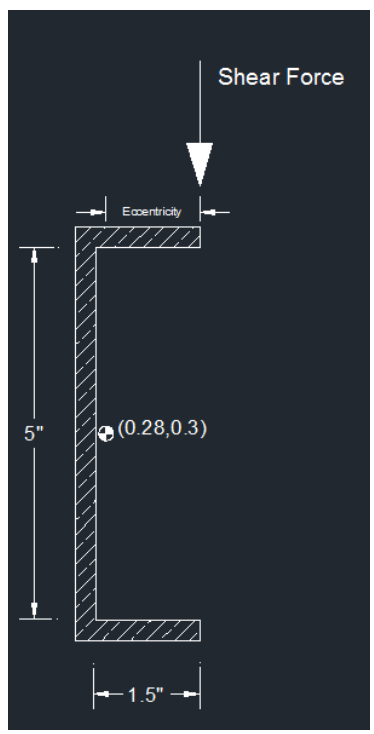
Eccentricity = 1.5-0.28 = 1.22in

a = 1.22/5 = 0.244in

Coefficient from table = 2.98in

Dmin, (weld size) = $\left[\frac{174}{(0.75*1*2.98*6)}\right] * \left(\frac{1}{16}\right)$ = 0.822in

Use 13/16" weld size



Min thickness of Shear plate = (13/16)+(1/8) =15/16"

Use 1" thick shear plate

Design Shear stress of welds = 0.75*0.707*(13/16)*42=18.1kips/in

Design Strength for each “C” weld = 18.1(2B+L) = 144.75kips

Total design of strength of shear connection = 289.52kips > max shear

TENSILE CONNECTION

Tensile Force = 2312.2kips

Electrode = E70XX

Fexx, tensile strength of weld = 70 kips/in^2

Fw,ult shearing stress = 0.6*Fexx=42kips

W, weld size = 6/16in

> ***Table J2.4***: *for base material with a thickness over 0.75in, the fillet weld size may not be less than 5/16in*

Φ rn, design weld shear strength = 0.75*0.707*(6/16)*42=8.35kips/in ***(controls)***

Check Base metal

Thickness of base metal = 0.5in

Φ rn, Shear yield strength = 1*0.6*36*0.5 = 10.8kips

Φ rn, Rupture Strength = 0.75*0.6*58*0.5=13.1kips

L = 140in (transverse weld)

B = 20in

> Fw for transverse = 0.6*Fexx*(1+0.5sin1.5θ), where θ = 90degrees; Fw = 0.6* Fexx*1.5

Design Strength of the weld = 8.35*((1.5*140*2)+(2*20))=3841kips > max tension force

BUCKLING OF THE HANDRAIL

Buckling load was calculated as:

$P_{cr} = \frac{2\pi^2 EI}{L^2} = \frac{2\pi^2(29000)(19.40)}{280^2}$ = 141.63 psi.

$\sigma_{cr} = \frac{P_{cr}}{A} = 42.80\ ksi.$

Then the compressive stress due to the moment in the deck was calculated as :

$\sigma = \frac{Mz}{I} = \frac{(1115)(2.25)}{0.47}$= 5390 ksf = 37.44 ksi.

$\sigma < \sigma_{cr}$. Therefore the handrail will not buckle.

APPENDIX O: FIRST SEMESTER REPORT

Charles River Crossing

By

The Anchorage Group

December 22, 2011

Nnamani Nnabuihe, Erika Yaroni, Timothy James, Pierre Dumas, Stephen Pendrigh

1.562 MEng Project-High Performance Structures

EXECUTIVE SUMMARY

The Anchorage Group (referred to herein as "the Group") will provide engineering, architectural, construction and financial consulting services for the renovation and design of the new Charles River Crossing in Cambridge. In this report the Group provides the client with five state of the art solutions that aim at providing a safe crossing of the river and bypasses of major roads for non-vehicular traffic. The Group has also taken into account the need to renovate the existing bridges at this location. Three of the concepts will look to provide a safe and architecturally interesting crossing over the Charles River between the Western Avenue and River Street Bridges. The other two concepts, which can be combined with any of the river crossings, eliminate the use of crosswalks at the four intersections that bound the site. Since the Charles River is used heavily for sailing and rowing, The Group has made an effort to limit the interference of these concepts with the river and to aesthetically integrate all designs into their environments.

The first concept, which is designed to minimize visual impact, is a self-supporting narrow bridge that has the appearance of a cantilevered addition of the existing bridges. Situated close to the outside of each of the bridges, this cantilevered design is fully supported by its own columns, which are placed close to the existing piers. This design will provide pedestrian crossing across the river, therefore allowing the current bridges to be renovated and eliminate the need for the existing sidewalks.

The second design is a suspension bridge which will be built in a location between the two existing bridges. This bridge will provide temporary vehicle traffic lanes during the renovation of the other bridges and will then be retrofit with seating for the benefit of the public, in particular for special events such as the Head of the Charles.

The third concept is a modern looking cable-stayed bridge that will also be built between the two existing bridges. Like the previous concept, this design will support temporary vehicular traffic. Of the three river crossing concepts, this design will have the least impact on river traffic while also enhancing the Boston skyline with a new architectural pleasing design.

As stated above, the Group is also proposing two concepts for the pedestrian bypass of River Street and Western Avenue intersections. The first concept is an underpass, which will take the non-vehicular traffic under the arches of the current bridges. The underpass is designed so that it can be raised to allow river traffic under all arches. Alternatively, the second concept is an overpass which will simply pass over the intersections.

Given the qualification of the team described in this report, The Anchorage Group is capable of addressing all aspects of the project, from the design phase through budgeting, scheduling, and construction. The Team has provided its client with a summary highlighting the main advantages and disadvantages of each of these design options.

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THE COMPANY

ANCHORAGE Group, Ltd

Consultants in Civil Engineering

Anchorage-New York-London-Lagos

COMPANY OVERVIEW

The Anchorage Group provides engineering, architectural, construction and financial consulting services to private and institutional entities willing to change the built environment. The Group specializes in providing clients with state of the art turn-key solutions through the duration of the project: from conception to project completion.

We deliver the most economical solutions as well as signature projects that make the Group one of the most recognized and respected design-build construction firms in the world. Helping clients meet their goals and completing breathtaking projects is the Group’s daily motivation. This commitment is reflected in the company’s motto: “make it happen”.

CORE SKILLS AND OFFERINGS

Since its inception in 1948, the Anchorage Group has been able to combine its expertise in architecture, structural engineering, and project management to deliver world-class projects on time and under budget. The experience gathered over the years has given the Group expertise in the following areas:

-Environment and Sustainability: As a matter of priority, The Anchorage Group keeps up with global trends in sustainability. The Group strives to meet the most demanding standards anywhere in the world by limiting the impact of projects on the natural environment and targeting the Leadership in Energy and Environmental Design (LEED) certification.

-Cost optimization: Relying on the technical knowledge, equipment and resources at its disposal, the Anchorage Group has the capacity to deliver finished projects within budget. The best practices developed over the years executing technically intensive projects gives the Group the unique knowhow to implement the most cost effective methods to tackle any structural and construction challenge.

-Structural Engineering: The Anchorage Group has developed the reputation for specializing in and leading the development of the most complex structural projects. The company can confidently rely on its technical prowess and its international network of colleagues and associates to deliver innovative solutions in a timely manner.

PORTFOLIO

The Anchorage Group boasts a long and proud history of successfully designing iconic footbridges around the world. Several projects are highlighted below.

DNA BRIDGE, MARINA BAY SANDS, SINGAPORE

Figure 1: DNA Bridge

This modern marvel redefines the limit of artistic creativity and engineering genius. Completed in 2009, it is the world's first bridge based on the double helical structure of human DNA. The bridge spans 280 meters over the Marina Bay area and is equipped with computer-controlled lighting to improve the well-being of the pedestrians.

Although it functions as a standard beam bridge, the architectural façade highlights the Group’s ability to be creative in tackling mundane challenges. Its low profile also ensures that the current skyline around Marina Bay is not drastically altered.

LÉOPOLD SÉDAR SENGHOR BRIDGE, PARIS, FRANCE

Figure 2: Leopold Sedar Senghor Bridge

The "Passerelle" Leopold Sedar Senghor is an arch bridge situated right in the heart of Paris linking the banks of the Orsay Museum with the Tuileries garden.

The Anchorage Group successfully executed this project in a highly populated area of the city. This shows the Group's ability to work in busy parts of cities without significantly impacting the daily activities of residents and commuters. Additionally, the arch structure does not interrupt the navigational channel, which allows activities, like sailing, to proceed without obstruction

HARBOR DRIVE PEDESTRIAN BRIDGE, SAN DIEGO, USA

Figure 3: Harbor Drive Pedestrian Bridge

This innovative bridge has become one of the landmarks of San Diego. It is a cornerstone of downtown San Diego's development and an iconic gateway to the city. It is one of the longest self-anchored pedestrian suspension bridges in the world.

This design illustrates the quality of the Anchorage Groups' work and the diversity of solutions it is able to deliver in order to meet the demands of clients. It also depicts the Group's ability to develop cutting edge cable-stayed and suspension bridges that not only blend into a city's skyline but also help to increase the city's prestige.

Project Background

The Western Avenue Bridge and River Street Bridge (pictured below) are earth-filled, reinforced concrete arch bridges that cross over the Charles River. They were built in 1924 and 1925 respectively. Both bridges intersect with Memorial Drive and Soldiers' Field Road, and contain 3 lanes of traffic plus a pedestrian sidewalk on either side of the road.

Figure 4: Western Avenue Bridge (left) and River Street Bridge (right)

At present, traffic flow on the River Street Bridge is one-way, eastbound, into Boston while the Western Avenue Bridge is one-way, westbound, into Cambridge. The large volume of pedestrian traffic in the area is attributed to the local universities and local residents enjoying the beautiful river walkways. As seen in Figure 5, the bridges are surrounded by numerous universities and residential neighborhoods. Currently these trails require crosswalks and crossing lights at the foot of the bridges, which is disruptive to pedestrians, cyclists and motorists alike.

As both bridges have fairly low-lying arches, the river is navigable by small craft only. However, there is a significant amount of river traffic in the form of rowing shells.

The two bridges are in need of significant renovation, with all the components of the River Street Bridge being listed in "fair" or "poor" condition by the Massachusetts Department of Transport (MassDOT). The Western Avenue Bridge is only slightly better with nearly all components in the same condition as the River Street Bridge (only the substructure and piers listed as "satisfactory"). The MassDOT currently has plans to perform significant repairs to both bridges. The last renovation occurred in 1981 and only focused on road surface rehabilitation.

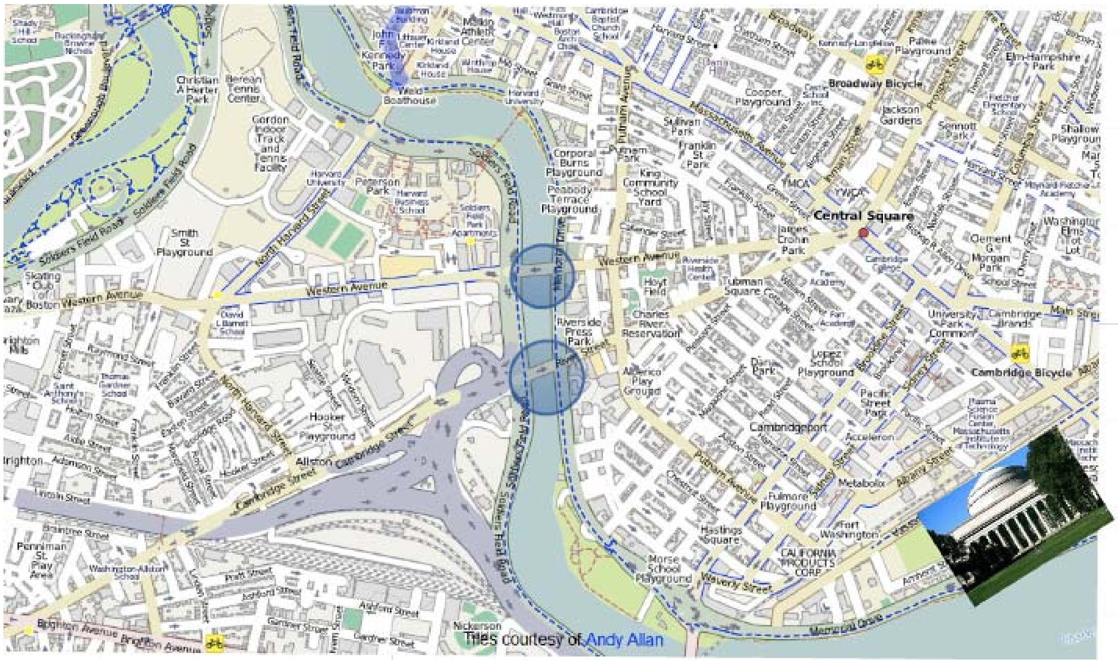


Figure 5: Aerial map of location, highlighting existing bridges

EXISTING GEOMETRY

The Western Ave Bridge consists of three arches supported by concrete piers and spread footings set into granular soils and clay found underneath the river bed settlement. It carries both vehicular (three lanes) and pedestrian (two sidewalks) traffic across the Charles River and spans a distance of 329ft. The elevations of the top and bottom of the exterior arches are 20.42ft and 8.5ft respectively and are 60ft across. The interior arch has top and bottom elevations of 24ft and 8.5ft respectively and spans 75ft. The bridge deck's maximum elevation is 28ft and is 57ft wide; 40ft for vehicular traffic with 8.5ft sidewalks on either side.

The River Street Bridge consists of three arches supported by concrete piers and spread footings set into granular soils and clay found underneath the river bed settlement. It carries both vehicular (three lanes) and pedestrian (two sidewalks) traffic across the Charles River and spans a distance of 304ft. The elevations of the top and bottom of the exterior arches are 20.42ft and 8.5ft respectively and are 60ft across. The interior arch has top and bottom elevations of 24ft and 8.5ft respectively and spans 75ft. The bridge deck's maximum elevation is 28ft and is 57ft wide; 40ft for vehicular traffic with 8.5ft sidewalks on either side.

The average water level is 8ft above gauge height, with flood level reaching 8.5ft at the two bridges (which coincides with the bottom of the arches).

DESIGN CONSTRAINTS

PEDESTRIANS/CYCLISTS

Both the river crossing and the road crossing should provide a safe, easy to use crossing for both pedestrians and cyclists. The road crossing should not interfere with vehicles at any of the four intersections of the existing bridges. Minimum width should be 10ft to allow for two way flow of foot/bike traffic.

VEHICLES

River crossing should, ideally, include provision for temporary use of vehicles. Vehicle use of the river crossing will occur during renovation of the two existing bridges, Western Avenue Bridge and River Street Bridge, to ease traffic congestion of the local area. After renovations, no vehicular access of the new river crossing is needed.

Traffic flows along Soldiers Field Road and Memorial Drive should not be permanently rerouted to accommodate the new river crossing/road crossing unless deemed absolutely necessary.

River Traffic

River traffic should remain unchanged and the Anchorage Group should limit the amount of piers placed in the river. This is especially true for reducing the effect on large scale races such as the Head of the Charles, whose route passes through the area of interest.

Minimum Requirements

To comply with the Americans with Disabilities Act, the minimum gradients of all ramps shall be 1:12 for a maximum of 200ft. If the ramp should extend further than this, resting intervals shall be included.

The minimum lane width to be used along the river crossing shall be 10ft, however if being designed for vehicular use the minimum lane width shall be 12ft. This minimum width shall allow for a single lane of vehicular traffic without pedestrian use. The clearance above the driving surface shall be at least 15ft.

To accommodate cyclists using the trail, a minimum turning radius of 100ft shall be used and a minimum clearance between piers shall be 44ft for river traffic; however, the ideal minimum should be 88ft to allow two rowing shells to pass simultaneously.

DESIGN LOADS

As the Group considered concept designs for this RFQC, only the significant loading cases were considered. Specifically, the estimated dead load of the bridge plus the live loads of the pedestrians (and traffic if applicable). During the detailed design, a more comprehensive review of the loads the structures will be subjected to will be carried out.

PEDESTRIAN LOADS

The National Cooperative Highway Research Program (NCHRP) recommends using maximum pedestrian loads of 90psf with a load factor of 1.75 equating to 158psf. This design load will be the main design load when considering the bridge as a whole.

VEHICULAR LOADS

The American Association of State Highway and Transportation Officials (AASHTO) recommends using a combination of two types of live loading from a choice of three. These three, referred as the HL-83 loading cases, are called the Design Truck, Design Tandem, and a Uniform Lane Loading. For the concept design, only one of these types was considered. The Group considered Type 2 (Design Tandem) which involves a two axle vehicle with 25kips on each axle separated by 1.2m. This loading will dominate when considering local punching shear. However, for global strength requirements, pedestrian traffic dominates.

Design Calculations

The maximum allowable deflection for all spans of length L will be L/360 for a live and dead load combination and L/1000 for live load only (assuming the dead load can be cambered out).

The factored load combination 1.2*(Dead Load) +1.6*(Live Load due to Occupancy) will be used under the Load and Resistance Factor Design (LRFD) method.

All steel used in the design will have a yield stress of $f_y = 60$ksi, and concrete will have a compressive strength (cylinder test) $f'_c = 4$ksi.

River Crossing Concepts

Concept 1: Cantilever Bridge

The first concept the Group developed is a "cantilever bridge". The main focus of this bridge is to minimize the footprint and visual impact on this historic area. This concept calls for a new pedestrian bridge immediately adjacent to each existing bridge; on the south of the River Street Bridge and on the north of the Western Avenue Bridge. (See Figure below)

Figure 6: Cantilever Bridges (shown in red)

Each pedestrian bridge will veer away from the sidewalks along the river approximately 200ft from the entrance to the existing bridges. They will then slope upward and continue adjacent to the bridges above the three arches towards the other bank. The new pedestrian bridges will not be visible from the driving surface, though pedestrians will be.

There will be no structural connection between the existing bridges and the new pedestrian bridges. However, because they run immediately adjacent to each other and since the new pedestrian bridges will only be supported on one side (the side which abuts the existing bridges) there will be the appearance that the new bridges cantilever off the existing bridges. (See figure below) The supports for this bridge will encroach into the archway channels 2ft on each side of the existing bridges' piers; narrowing the channels from 75ft to 71ft (middle arch) and 60ft to 58ft (outside arches).

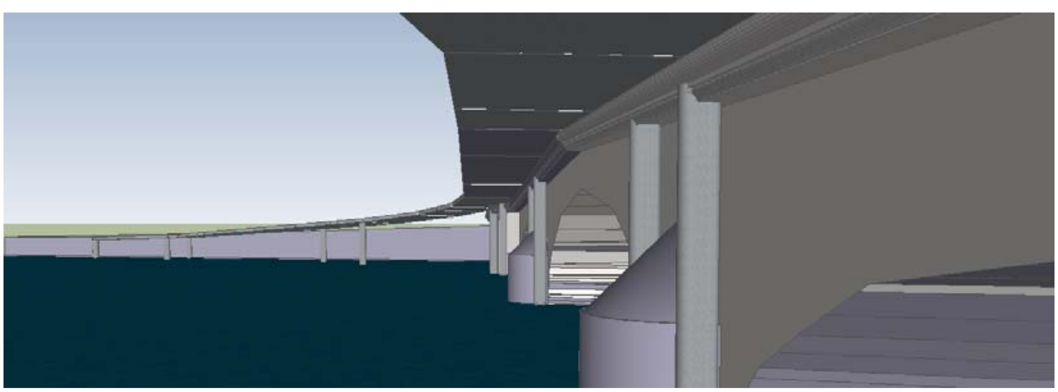


Figure 7: Cantilever as seen from below

In keeping with the cantilevered concept, columns will be staggered. i.e. The bridge will be cantilevered at each support.

Initial calculations show that deflection governs this design. The Group assumed the dead load deflection can be cambered out and considered the live load deflection only. The controlling load case for this bridge is with every other bay loaded uniformly.

The pedestrian bridges will be constructed out of 60ksi steel. Based on hand calculations and SAP analysis (See Appendix B), the deck will be a box section constructed from ¾" steel and be 12ft in width and 2ft in depth. This deck will be supported by 2ft diameter cylindrical steel columns/piles with a wall thickness of 1.5". These piles will be driven into the ground and river bed.

The maximum spans for this bridge will be 80ft and the maximum pile height will be 36.5ft.

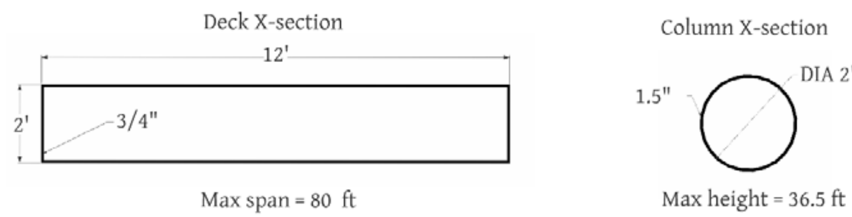


Figure 8: Cantilever bridge- typical member cross-sections

During construction, the piles will be driven first, deck sections (shown in red below) will be placed on those piles and, finally, intermediate sections (shown in yellow below) will span the gaps. The largest section will be 50ft in length and weight 20 tons. All sections will be transported by truck and lifted into place by crane.

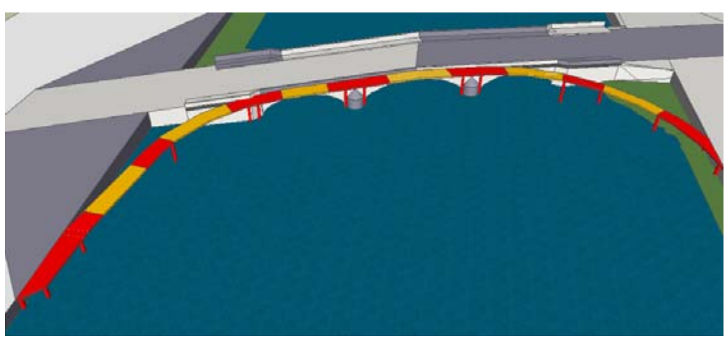


Figure 9: Construction Sequence

CONCEPT 2: SUSPENSION BRIDGE

The Anchorage Group's second proposed design for the Charles River crossing is a suspension bridge, specifically a through arch bridge. The Group chose to develop a concept for a free standing bridge that would enhance the Boston skyline without taking away from the beauty of the neighboring historic bridges. Therefore, components of the old bridges are incorporated into this new bridge design with a modern twist. This concept will provide for temporary vehicular traffic during the renovation of both River Street and Western Avenue bridges and then be converted into a primary pedestrian and bicycle crossing.

The basic idea for this concept is to create a suspension bridge, while incorporating the arch, which is a main design feature of the adjacent stone bridges. As described earlier, both the Western Ave. and River St. bridges have three arches along their spans. The arch feature therefore led the Group from a typical suspension bridge to a through arch bridge. By definition, a through arch bridge is composed of an arch, which extends above the deck, and cables in tension to suspend the deck. For this concept, the team decided to also extend the arch through, and below, the deck to a lower foundation. (See figure below)

Figure 10: Suspension bridge elevation

Another aspect of the neighboring existing bridges the Group integrated into this design is the division of the span into three segments, which the existing bridges accomplish with three arches. For this design, the total span of 390ft is divided into a central span of 190ft and two outer spans of 100ft each. The central span is supported by the through arch bridge and the outer spans are supported with extra supports that share the foundation with the arch, as well as compression rods that connect the deck to the arch below. Cross braces are added to provide laterally support to the arches, which are set at the outside of the deck width. (See figure below)

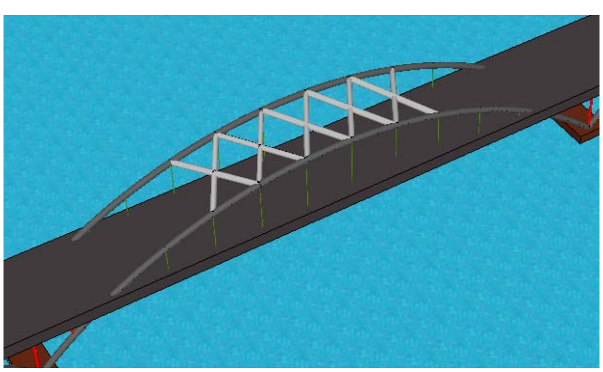


Figure 11: Suspension bridge bracing

Based on preliminary calculations, member sizes were determined for each component of the bridge using 60ksi steel and the load requirements discussed earlier. The calculations can be found in Appendix C. The following table shows the cross section for each component of the bridge. The suspension cables of the bridge are spaced 15ft on center, as are the nodes for the cross bracing. In order to maintain symmetry under the deck as well, the compression rods are spaced 13.75ft on center.

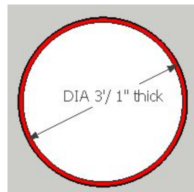
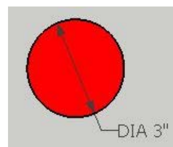
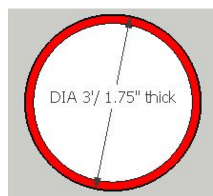
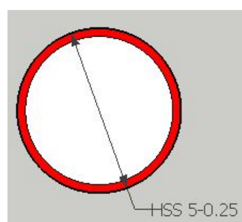
<table>
<tr><th>Item</th><th>Cross Section</th></tr>
<tr><td>Arch</td><td>
</td></tr>
<tr><td>Cable</td><td>
</td></tr>
<tr><td>Side Support</td><td>
</td></tr>
<tr><td>Compression/Tension Rod</td><td>
</td></tr>
<tr><td>Deck</td><td>
</td></tr>
</table>

Figure 12: Suspension bridge- typical member cross-section

One of the added elements incorporated into this bridge design is the addition of seating on the bridge once vehicular traffic is removed. This seating will allow for a gathering space on the river and will provide superior bleacher type seating for the many events that takes place on the Charles River such as crew racings and the “Head of the Charles”. Prefabricated off site, the seats will be installed on the bridge when traffic patterns return to normal and also be easily removed in the future if need be. Preliminary calculations were performed and the cross section determined suitable can be seen in the figure below.

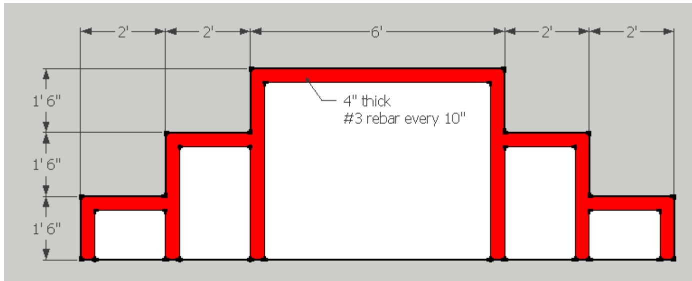


Figure 13: Seating cross section

The overall deck width of the bridge is based on accommodating the above seating. Because the requirements only call for a single lane of temporary vehicle traffic, this was not the controlling factor in the bridge width. In order to supply two lanes of pedestrian/bike traffic and seating, the minimum useable deck space is 24ft. However, 3ft was added to each side where the arch will be placed and cables connected. Therefore, the total deck width is 30ft, as detailed previously in the list of cross sections. The figure below shows the comparison of deck space as it is utilized for vehicular traffic and pedestrian traffic.

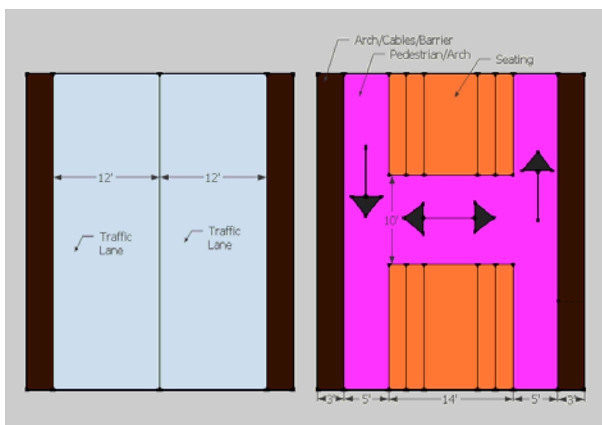


Figure 14: Bridge Uses

As shown, when the bridge is utilized for vehicles traffic there will be two 12ft lanes. Ideally, the traffic will flow in a single direction since both River St. and Western Ave. bridges have single direction traffic and will be renovated at different times. When converted for pedestrian use, the bridge will have two 5ft one-way traffic lanes separated by the seating segments which are 14ft wide. While the Group has not yet designed the specifics, the intent is for the seating segments to be spaced out along the bridge so there are breaks to allow pedestrians to turn around and travel in the opposite direction on the other lane. An example of how the seating could be configured can be seen in the figure below.

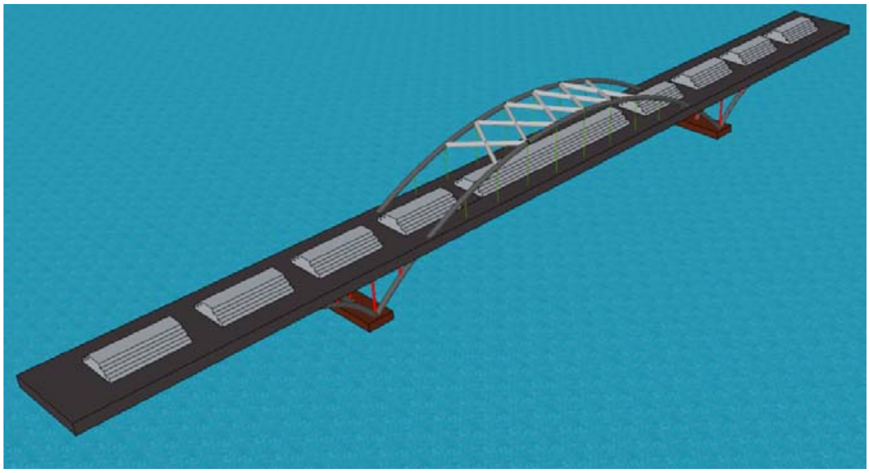


Figure 15: Possible seating scheme

Construction for this bridge will be divided into two main stages. Stage one will be off site fabrication where each element will be prefabricated in sections and then shipped to the site. The second stage will be on site assembly of the prefabricated sections. The number of sections for each element is based on the weight a barge and crane can handle. This leads to the arch being split into three segments and the deck being split into 30ft segments. Construction is assumed to be done with a 40 ton crane mounted on a barge.

The first part of construction for this bridge is to drive the piles and pour the foundation. With the foundation, the bearing plates for the arch and side supports can be put in place. Before the arches and deck can be erected, temporary support towers need to be assembled on shore to provide temporary tie back for the bridge during the assembly process.

For assembly, each arch will be delivered in three segments. First, the exterior segments will be erected on both arches and temporary cross ties will provide lateral stability as well as tie backs to the onshore towers. Next, the center segment of the arches will be erected and secured with permanent cross braces between the two arches.

Like the arch, the deck segments will be transported to the site on a barge. They will then be lifted into place and hung from the arch with the cables. To help keep loads equally distributed, the deck will be assembled starting at the center and working outward.

Fabricating the bridge elements off site will shorten the construction process. This is important since river traffic will be blocked by the barge(s) used during construction.

CONCEPT 3: CABLE STAYED BRIDGE

The Group chose a modified harp cable-stayed bridge as the third river-crossing concept. The concept provides an adequate solution to pedestrian and bike traffic and could also serve as a prestige project for both Cambridge and Boston, MA. Its slender deck allows this concept to maintain a low profile without obstructing the current skyline. The 374ft Bridge consists of two, 109ft inverted “A-shaped” pylons on either side of the river and a steel deck supported every 20ft.

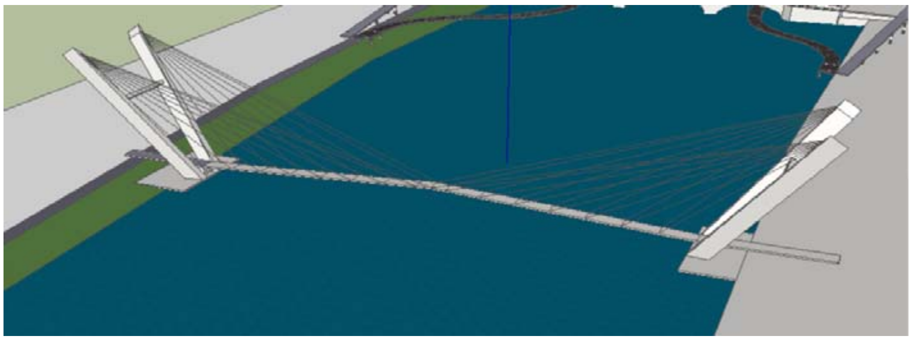


Figure 16: Cable stayed bridge

A major design consideration for this concept is to maximize the navigational channel in the Charles River. Therefore, the approach bridge, that often accompanies cable-stayed bridges, is eliminated to allow the bridge deck to converge with the existing sidewalks. Although this serves to reduce the encroachment of the piers into the river channel, it creates a challenge with respect to structural stability.

In conventional cable-stayed bridges cables are tied to the pylons from the approach bridge to balance the overturning moment created by the deck spans. However, in this concept, stability of the pylons is achieved by tilting the pylons 33 degrees from the vertical axis. The solution, similar to the Punte de la Unidad Bridge in Monterrey, Mexico, enables the weight of the pylon to create a negative moment and achieve equilibrium.

The pylons are designed as reinforced concrete elements; primarily to create a structure heavy enough to resist the imposed overturning moment. The decks are thin steel box girders; to maintain the structure’s elegance and also to reduce the dead load on the pylons. To achieve a maximum river clearance of 16ft at the center of the deck span, from an initial height of 7ft, the deck is inclined at a slope of 1:16.5, which is well within the ADA requirements.

Preliminary calculations show that 2-inch cables, ranging from 22.5˚ to 49˚, are sufficient to support the deck. Additionally a deck with a depth, width and thickness of 1ft, 12ft and 0.25”, respectively, is adequate to support a live load deflection well below the limiting value of 0.12”.

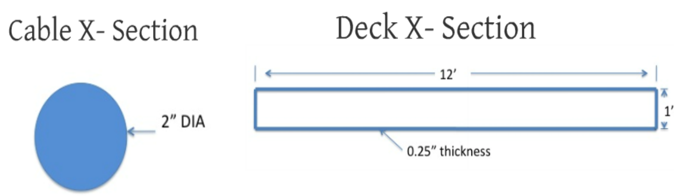


Figure 17: Cable-stayed bridge- typical member cross-sections

The following is a short description of the construction process for this bridge. A drilled piled shaft or precast concrete caissons can be used to construct the foundation. The pylons are then constructed in-situ using the slip form system. This has the added advantage of eliminating joints and the need for formwork; all if which lends itself to a stronger and more economical structure. Once the pylon is constructed, a derrick crane is used to erect the preassembled steel box girders and cables, starting from pylon moving towards the middle of the deck span. The process occurs simultaneously on both sides of the Charles River unitl the two halves are connected where they converge.

Figure 18: Deck installed by crane

PEDESTRIAN BYPASS

Regardless of which river crossing concept is implemented, there is still a need to eliminate pedestrian traffic at the intersection of Western Avenue and River Street Bridges with Memorial Drive and Soldiers' Field Road. Therefore, the Group has developed two concepts which can be implemented with any of the three river crossing concepts: an underpass and an overpass.

CONCEPT 1: UNDERPASS

The first bypass concept the Group developed that will allow uninhibited traffic flow for both vehicles and pedestrians is an underpass.

This concept reroutes pedestrians under the outer arches of both existing bridges (see figure below). Because of the required minimum height clearances for this pathway, this underpass will be close to the center of these arches, leaving just over 25ft of clearance for river traffic. The Group realized that this is prohibitive as it will impact the feasibility of events like the "Head of the Charles". To mitigate this, the design includes a cable/hinge system that will allow the underpass to be rotated out of the way to allow river traffic. (See Figure 23)

Figure 19: Underpasses (shown in red)

This pathway will be supported by steel columns near the shore and suspended from the existing bridge with a cable system.

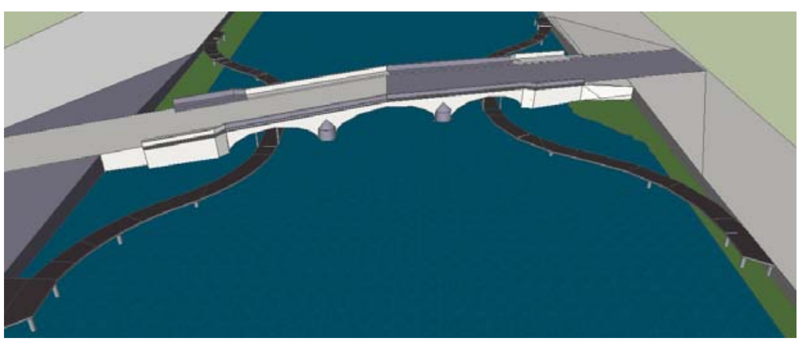


Figure 20: Underpass support system

Deflection governs this design. The Group assumed the dead load deflection can be cambered out and considered the live load deflection only. The controlling load case for this bridge is with every other bay loaded uniformly (including the span under the existing bridge).

The pedestrian bridges will be constructed out of 60ksi steel. Based on hand calculations and SAP analysis (See Appendix E), the deck will be a box section constructed from 1.5" steel and be 12ft in width and 2ft in depth. This deck will be supported by 2ft diameter cylindrical steel columns/piles with a wall thickness of ¾" and steel cables ½" in diameter under the existing bridges.

The maximum spans for this bridge will be 85ft and the maximum depth of pile will be 22ft.

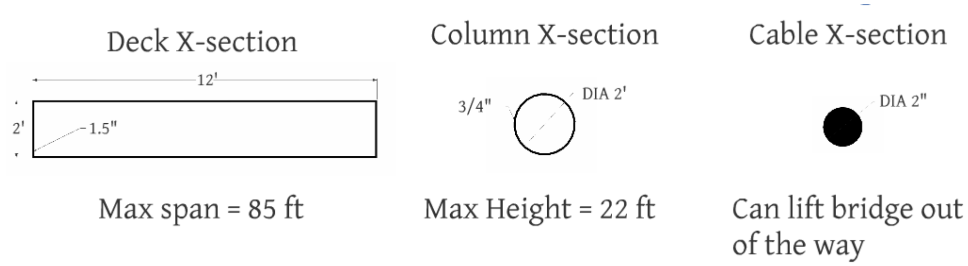


Figure 21: Dimensions of Underpass bridge

During construction, the piles will be driven first then deck sections (shown in red) will be placed on those piles. Next, the cable system will be installed. After the cable system is in place, the portion of the underpass that will be under the arch (also shown in red) will be brought in in two pieces, placed on a barge and assembled. That assembly will then be moved under the arch, connected to the cable system and lifted off the barge. Finally, intermediate sections (shown in yellow) will span the gaps. The largest section will be 50ft in length and weight 40 tons. All sections will be transported by truck and lifted into place by crane.

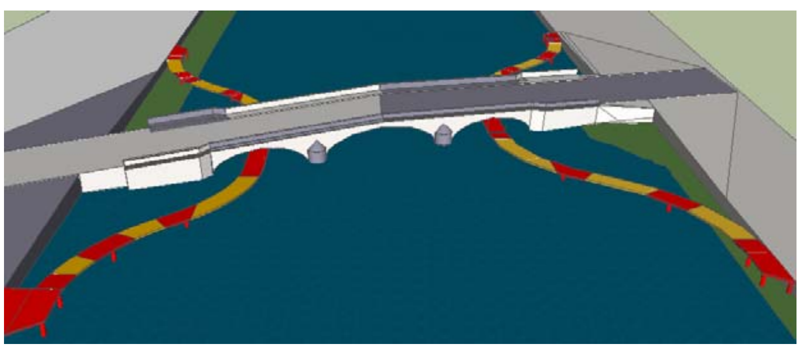


Figure 22: Underpass construction sequence

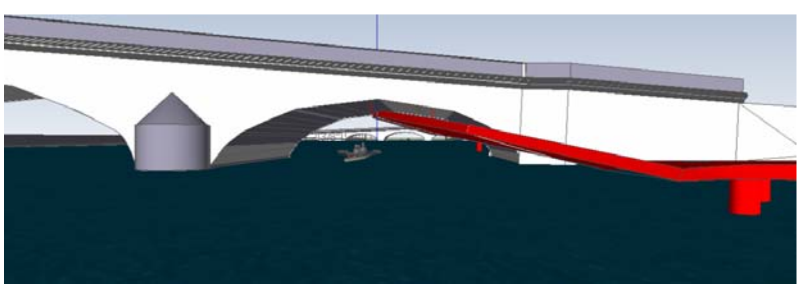


Figure 23: Bridge lifted to allow for river traffic (color added for clarity)

CONCEPT 2: OVERPASS

The Group chose an overpass as the second road-crossing concept. The concept provides an opportunity to move pedestrian and bike traffic across Western Avenue and River Street without interfering with river traffic. It consists of a span that stretches over the existing roadway and two ramps that connect the elevated span to the sidewalks.

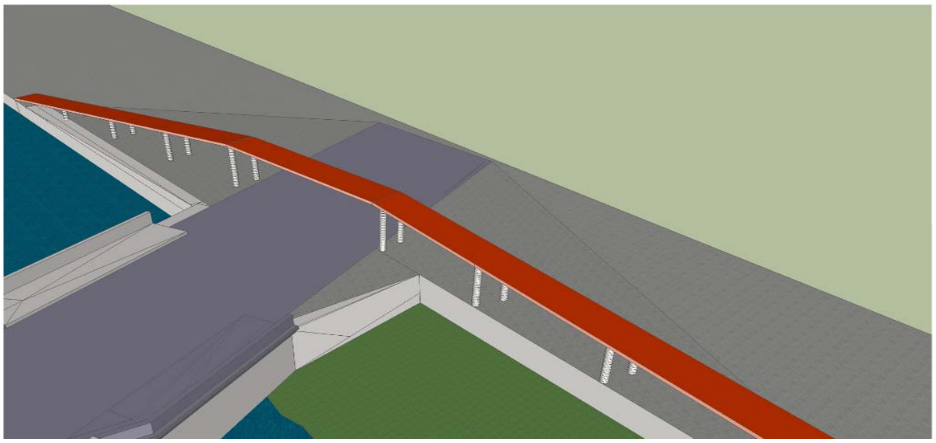


Figure 24: Overpass

A minimum height of 15ft must be maintained as the bridge spans across the existing roads. Therefore, the ramps extend out a minimum of 180ft to maintain the ADA required slope 1:12. An architectural envelope could be installed around the bridge to improve its visual appearance and make it unique to the Boston metropolitan area.

The decks are steel box girders and are supported at every 40ft while the columns are reinforced concrete members. The steel box girders are 12ft wide, 1ft deep and ½" in thick. The columns are 1ft in diameter with a ½" in thickness.

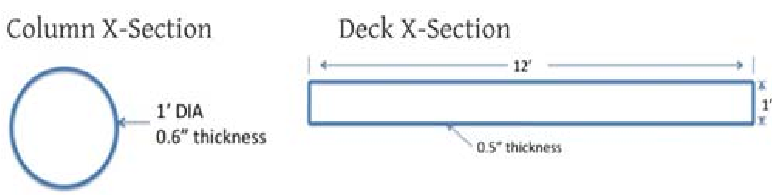


Figure 25: Geometric properties

During construction, the bridge is shut down and traffic re-routed as depicted in the traffic flow diagram. The columns are then installed before the decks are installed. As mentioned in the underpass concept, all sections are small enough to be transported to the site by truck and lifted into place by crane.

TRAFFIC FLOW

The rerouting of traffic for all three concepts and the renovation of the two existing bridges all follow similar principles. If either the suspension bridge or cable-stayed bridge concepts is selected, the renovation of River Street Bridge and Western Avenue Bridge will have the traffic flow of Figures 26 and 27 respectively. During the construction of the cantilevered bridge, and when renovating the two existing bridges with this concept design, the traffic flow will be very similar to Figures 26 and 27, however there will not be the option of routing traffic over the new river crossing and so there will only be one lane devoted to moving traffic from the renovated bridge to the other bridge which is not being worked on.

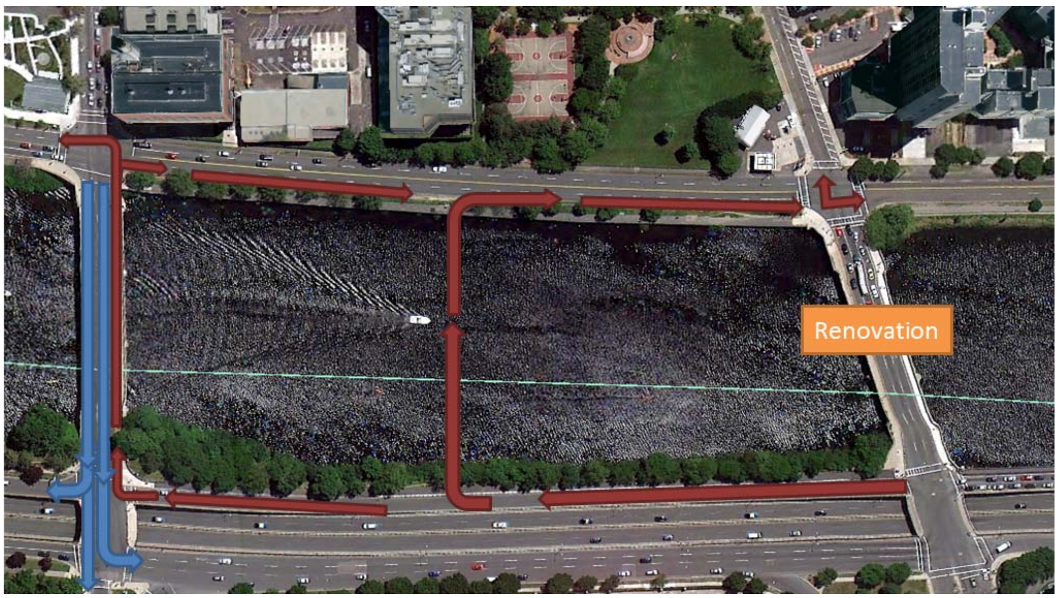


Figure 26: Traffic flow while renovating of River Street Bridge

Figure 27: Traffic flow while renovating Western Avenue Bridge

DISCUSSION OF THE CONCEPTS

The Anchorage Group has included in this report 3 concepts that address a new river crossing in accordance with specifications defined by the client. The purpose of this discussion is to provide measurable parameters to help determine which of the solutions provided best suit the client's needs.

The Cable-Stayed Bridge is a design solution that would limit interference with the river traffic, an important design constraint. Indeed, with this solution the Charles River Bridge would only need 1 span and therefore no piles in the river. This concept is a sleek and modern design that would be an aesthetic option for this crossing. It is also a solution that provides a quick and easy construction.

However, the Cable-Stayed Bridge also requires a high degree of control in regard to quality, time and budget. Given the experience of the Anchorage Group in the built environment, and particularly in the area of cable-stayed bridges, the Group is confident it would complete this project while exceeding the expectations of the client.

The Suspension Bridge is also a solution that offers a low profile design that would limit interference with river traffic. However it is the more expensive and access to the river will be limited during construction due to the use of a barge and the need for temporary supports.

The Cantilever Bridge it is clearly the most cost effective solution. It also requires the shortest construction time, which is clearly a huge advantage. Moreover, this innovative solution would have an extremely minimal footprint in the river. Also, it is a very aesthetic solution given the curve and slenderness of the structure. However, some may feel the integration of this concept with the existing bridges compromises their original look and feel. Structurally, it is also the less impressive option.

	Cable Stayed Bridge	Suspension Bridge	Cantilever Bridge
Aesthetics	+	+	++
Money	+	++	+++
Interference with river	+++	++	+++
Time	+	+	+++
Constructability	++	+	+++
Integration in surroundings	++	++	+
Sum	10	9	15

For the road crossing, the Anchorage Group has designed two concepts that perfectly meet the expectations and specifications of the client.

The first concept is the underpass. There are a two interesting features with this design: First, it takes pedestrian traffic away from the road and down to the river (a nice reprieve from running along-side vehicles. Second, the underpass is constructed to allow it to be rotated out of the way to allow river traffic during events like the "Head of the Charles".

However, this beautiful design and its perfect integration in its surroundings would require a greater investment than the overpass solution.

As stated above, the Overpass concept is cheaper easier to construct than the Underpass. The Overpass would also not interfere with the river and requires less maintenance than the Underpass solution.

That being said, it may be considered visually obtrusive. Additionally, though on-site construction time will be short, some lane closures may be necessary.

It should be noted; because all 5 concepts are steel, they will require routine maintenance.

	Underpass	Overpass
Aesthetics	+++	---
Money	-	++
Interference with river	+++	+++
Time	+	++
Constructability	+	++
Integration in surroundings	+++	-
Sum	10	5

Based on the Anchorage Group's preliminary analysis all concepts are valid designs and will offer proper solutions for the client. The above comparison can be used to help the client select the preferred concept(s).

APPENDIX A: RESUMES

Nnabuihe Nnamani

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EXPERIENCE

THE ANCHORAGE GROUP
Anchorage, AK
Construction and Engineering Project Manager June 2005 – December 2011

- **DNA Bridge**, Singapore
  Managed the construction of this masterpiece bridge. Supervised the different mechanical, light, structural engineers. Coordinated the work of the different companies.

- **Harbor Drive Pedestrian Bridge**, San Diego
  Carried out quality control inspections to ensure that recommended procedures were followed in correcting concrete defects such as cracks and honeycombs

- **Passerelle Leopold Sedar Senghor**, Paris, France.
  Managed the construction of this bridge situated in a very busy area of Paris. Supervised environmental risk assessment and the impact on the Seine river.

EXXON-MOBIL **Ras Laffan, Qatar** *Project Manager* June 2000 – April 2005

- Managed construction of a gasification plant for EXXON-MOBIL in Qatar. Completed the project under-budget and one year in advance.

ENI-SAIPEM **Port-Harcout, Nigeria**
Off-Shore Structural Engineer June 1998 - May 2000

- Assisted manager in designing off-shore structures for super major oil companies
- Supervised the finite element analysis of the team within ENI

EDUCATION

MASSACHUSETTS INSTITUTE OF TECHNOLOGY (MIT)
CAMBRIDGE, MA *CIVIL AND ENVIRONMENTAL ENGINEERING DEPARTMENT* JUNE, 1998
Master of Engineering in High Performance Structures

The George Washington University
Washington, DC
Bachelor of Science in Civil and Environmental Engineering May, 1997

AWARDS AND PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers
National Society for Black Engineers

SKILLS

Computer: Microsoft Office, STAAD Pro, RISA 3D, AutoCAD, MATHLAB
Foreign Languages: Igbo (fluent) and French (conversant)

Stephen Pendrigh

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EXPERIENCE

THE ANCHORAGE GROUP **Anchorage, AK**
Construction Engineer June 2005 – December 2011

- **Marina Bay Sands**, Singapore
  Supervised the construction on this very large-scale project. Planned and scheduled the work on the building site. Coordinated the different companies on site.

- **Hoover Dam Bridge**, Las Vegas
  Led the team during the construction of the Hoover Dam bridge. Used extremely innovative solutions to make this project a succes and a state of the art bridge.

- **Passerelle Leopold Sedar Senghor**, Paris, France.
  Managed the construction of this bridge situated in a very busy area of Paris..

AECOM **Hong-Kong**
Construction Engineer June 2000 – April 2005

- Managed renovation of Kai Tak airpot in Hong Kong. Completed the project under-budget and one year in advance.

ARUP **London, UK**
Structural Engineer June 1998 - May 2000

- Participated to the solution given to the Millenium Bridge problem in London

EDUCATION

Massachusetts Institute of Technology (MIT) **Cambridge, MA**
Civil and Environmental Engineering Department June, 1998
Master of Engineering in High Performance Structures

University of Cambridge, Queens' college **Cambridge, UK**
Bachelor of Science in Civil and Environmental Engineering May, 1997

AWARDS AND PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers.
Licensed PE in Structural Engineering in MA, AK.
Member, Boston Society of Civil Engineers.

SKILLS

Computer: Microsoft Office, STAAD Pro, RISA 3D, AutoCAD, MATHLAB
Foreign Languages: Spanish (fluent) and German (conversant)

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EXPERIENCE

THE ANCHORAGE GROUP **Anchorage, AK**
CEO and Head of Design June 2005 – December 2011

- **Zaragoza Bridge Pavilion**, Spain
  Supervised the design of this project.

- **Hoover Dam Bridge**, Las Vegas
  Responsible for the design of the bridge and its visual integration in the environment.

- **Calatrava's Bridge**, Valencia, Spain
  Managed the design of this innovative bridge.

FOSTER+PARTNERS **London, UK**
Senior Partner June 2000 – April 2005

- Managed the design of the Viaduc de Millau in France which is the higher bridge in the world and one of the most emblematic state of the realization of Foster+Partners

ZAHA HADID ARCHITECTS **London, UK**
Associate Architect June 1998 - May 2000

- Participated to the design of the CMA-CGM headquarters in Marseille.
  Was in charge of the relation with the clients and the engineers.

EDUCATION

Massachusetts Institute of Technology (MIT) **Cambridge, MA**
Department of Architecture June, 1998
Master of Architecture

Ecole Spéciale des Travaux Publics **Paris, France**
Bachelor of Science in Civil and Environmental Engineering May, 1997

Lycée Pasteur **Neuilly-sur-Seine, France**
Intensive Mathematics and Physics May 1995

AWARDS AND PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers.
Licensed Architect
Member, Boston Society of Civil Engineers.

SKILLS

Computer: Microsoft Office, SAP, AutoCAD, MATLAB

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EXPERIENCE

THE ANCHORAGE GROUP — **Anchorage, AK**

Structural Engineer — June 2005 – December 2011

- **DNA Bridge**, Singapore

  Supervised the design of this project.

- **Hoover Dam Bridge**, Las Vegas

  Responsible for the design of the bridge and its visual integration in the environment.

- **Calatrava's Bridge**, Valencia, Spain

  Managed the design of this innovative bridge.

THORNTON TOMASETTI Inc. — **NYC, USA**

Senior Partner — June 2000 – April 2005

- Responsible of design for multi-unit condominium projects. Supervised 20 structural engineers

ARUP — **NYC, USA**

Associate Structural Engineer — June 1998 - May 2000

- Participated to the design of the Lincoln Center in NYC

EDUCATION

Massachusetts Institute of Technology (MIT) — **Cambridge, MA**

Department of Civil and Environmental Engineering — June, 1998

Master of Engineering in High Performances Structures

Stevens Institute of Technology — **Hoboken, NJ**

Bachelor of Engineering in Civil and Environmental Engineering, High Honors, GPA 3.74/4 — May, 1997

AWARDS AND PROFESSIONAL AFFILIATIONS

American Society of Civil Engineers.

Professional Engineer

Member, Boston Society of Civil Engineers.

SKILLS

Computer: Microsoft Office, SAP, AutoCAD, MATLAB

Timothy P James

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EXPERIENCE

THE ANCHORAGE GROUP — **Anchorage, AK**
Senior Structural Engineer — June 2006 – December 2011

- **DNA Bridge**, Singapore
  Managed the structural design of this masterpiece bridge.

- **Gateshead Millenium Bridge**
  Executed the entire design of this spectacular bridge in the UK.
  Won the *IStructE Supreme Award*

- **Passerelle Leopold Sedar Senghor**, Paris, France.
  Applied technical expertise and common sense evaluation of new requirements to ensure the project was coordinated

NAVALE MOBILE CONSTRUCTION BATTALION 74 — **Afghanistan**
Project Manager — June 2000 – April 2005

- Managed 106-person workforce consisting of military construction and engineering personnel at 13 forward operating bases (FOBs) spread across Afghanistan.

NAVAL FACILITIES ENGINEERING COMMAND FAR EAST — **Yokosuka, Japan**
Project Manager — June 1998 - May 2000

- Managed 40+ projects valued at over $50M
- Evaluated project designs for constructability and provided technical input to Architect/Engineer

EDUCATION

Massachusetts Institute of Technology (MIT) — **Cambridge, MA**
Civil and Environmental Engineering Department — June, 2006
Master of Engineering in High Performance Structures

University of Alaska — **Anchorage, AK**
Bachelor of Science in Civil and Environmental Engineering — May, 1997

AWARDS AND QUALIFICATIONS

PE (AK)
American Society of Civil Engineers
Top Secret Clearance

SKILLS

Computer: Microsoft Office, STAAD Pro, RISA 3D, AutoCAD, MATLAB
Foreign Languages: Mandarin (fluent) and Japanese (fluent)

Appendix B: Cantilever Bridge Calculations

The below results are the result of much iteration; final dimensions are:
Columns – HSS 2ft diameter, 1.5in thickness
Deck – 12ft wide, 2ft deep, .75in thickness

Loading:
The team considered both service load and construction loads during design. The analysis showed that service loads governed and deflection was the limiting criteria.

$$\text{Service load} = 1.2\ D + 1.6\ L\ (\text{where } L = 158\ \frac{\text{lbs}}{\text{ft}^2})$$

It was assumed the dead load deflection could be cambered out. Therefore, deflection criteria were compared against live load deflections only. (L/1000 being the limit)

Material Strength:
The team used 60ksi steel in this design.

Columns:
Design compressive strength for flexural buckling (from Chapter E section E2 in the Manual of Steel Construction):

$$P_u \leq \emptyset P_n = \emptyset A_g F_{cr}$$

$$where\ F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \quad and \quad \lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}$$

and K=.65

The maximum allowable load for the columns is 142kips.

SAP analysis with the entire bridge loaded at the service load (See figure 28) yielded a maximum base reaction of 140kips. (98.5% of allowable)

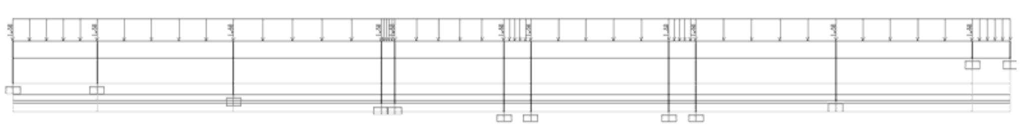


Figure 28: Cantilever Bridge model with full service load.

Deck:
In the direction of pedestrian traffic the analysis showed the following:

$$M_u \leq \emptyset M_n = \emptyset M_p = \emptyset F_y Z$$

The maximum allowable moment in the deck is 1.30 x 104 kip-ft.

SAP analysis with the bridge loaded with the service load on every other bay (See figure 29) yielded the maximum moment of 3.38 x 103 kip-ft. (26% of allowable)

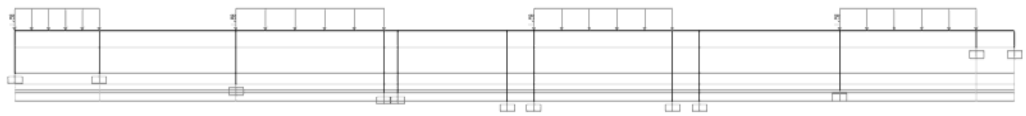


Figure 29: Cantilever Bridge model with every other bay loaded

Deflection was 68% of allowable with these dimensions. Reducing the overall depth of the deck by 3 inches or reducing the thickness of the plate by $\frac{1}{8}$ inch then exceeded the deflection criteria.

Analysis of construction loads (dead load only) showed the maximum allowable moment to be 1.22x104 kip-ft, based on the below equation:

$$M_u \leq \emptyset F_y Z$$

$$Z = \left|\frac{bh^2}{4}\right|_{outer\ dimentions} - \left|\frac{bh^2}{4}\right|_{inner\ dimentions}$$

This allows for a maximum cantilever length of 169ft. Since the longest span is 85ft in length, this is not a limiting factor.

In the direction of the cantilever action, the analysis showed the following for a representative 1ft section of deck:

$$M_u \leq \emptyset M_n = \emptyset F_y A_{gross} d$$

Where d is the distance between the centroid of the top and bottom flange

The maximum allowable moment in the deck is 50.6 kip-ft.

Analysis of service loads showed the maximum moment will be:

$$M_{max} = \frac{wl^2}{2}$$

Yielding a maximum moment of 23.5 kip-ft. (46% of allowable)

Construction:
The longest deck section during transport will be 50ft in length and weigh 19 tons.

The longest column will be 50 ft in length and weight 8 tons.

Either of these are easily transported by truck and lifted into place by crane.

APPENDIX C: SUSPENSION BRIDGE CALCULATIONS

<u>Reinforced Concrete Seating</u>

Max span = 6' = 72"

Minimum Thickness of 1-way slab: $t_{min} = \frac{L}{20} = \frac{72"}{20} = 3.6" \rightarrow assume\ 4"\ slab$

Loads:

$$DL = slab\ weig\square t = \left(\frac{4"}{12}\right)(150\ pcf) = 50psf$$

$$LL = pedestrian\ load = 158psf$$

Factored Load and moment:

$$w_u = 1.2DL + 1.6LL = 312.8psf$$

$$M_u = \frac{w_u L^2}{8} = \frac{(312.8psf)(6ft)^2}{8} = 1407.6\ lb-ft$$

Determine Reinforcement

Design for 1-ft deep segment (b=1 ft) and cover of 3 in (0.25 ft)

$$R_n = \frac{M_u}{\phi b d^2} = \frac{1407.6\ lb-ft}{0.9(1ft)(0.25ft)^2} = 25024\ psf = 173.78\ psi$$

$\rho_{min} = 0.0033$

Using table in "Design of Reinforced Concrete 8th Edition" by Jack McCormac

$$\rho = 0.0030 < \rho_{min}$$

Therefore, use $\rho_{min} = 0.0033$ $\quad A_s = \rho bd = (0.0033)(12in)(3in) = 0.1188in^2/ft$

Therefore, use #3 bars @10"

Design for Transverse Direction

$$A_s = (0.0018)b\square = (0.0018)(12in)(4in) = 0.0864in^2/ft$$

Therefore, use #3 bars @12"

How many segments for transport?

Total cross sectional area: 1440 in^2 = 10ft^2

Weight per ft: $= \rho A = (150pcf)(10ft^2) = 1500lb/ft$

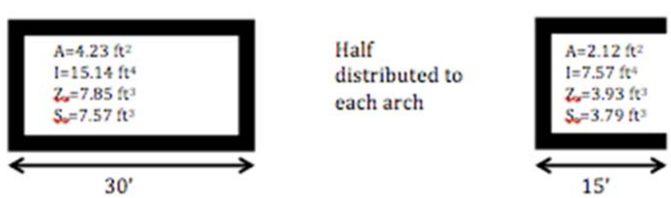
Forklift can hold 4 tons (8000lbs)

Therefore, transport in 5ft segments (7500 lbs)

Summary:

4" slab with #3@10" for longitudinal direction and #3@12" for transverse direction assembled in 5ft sections

<u>Deck Box Girder</u>



Length of girder = 390 ft

Assume height of 4' and thickness of 0.75"

Assume E=29,000ksi and Fy=60ksi

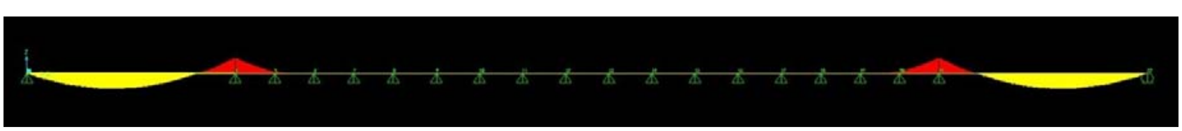
Model the half-segment in SAP2000 and replace cable connections with pin connection

Loads: LL=(158psf)(15ft)=2370lb/ft=2.37kip/ft

Dead Load: Concrete of seating=(150pcf)(5ft^2)=750lb/ft=0.75kip/ft

Dead Load: Self Weight of steel girder

Design Load: 1.2DL+1.6LL



Max Moment=2721 kip-ft

Max Shear=252.7 kips

Check Local Buckling (half section, like modeled)

$$M_p = F_y Z_x = (60ksi)\left(144\frac{in^2}{ft^2}\right)(3.93ft^3) = 33955\ kip-ft$$

$$M_n = M_p - \left(M_p - 0.7F_y S_x\right)$$

$$= (33955\ kip-ft) - \left(33955\ kip-ft - 0.7(60ksi)\left(144\frac{in^2}{ft^2}\right)(3.79ft^3)\right)$$

$$= 22922\ kip-ft$$

$$\phi_b M_n = 0.9(22922\ kip-ft) = 20630\ kip-ft$$

Check Bending

$$\sigma_y = \frac{My}{I} = \frac{(2721kip-ft)(2ft)}{7.57ft^4} = 719\ ksf = 5.0\ ksi < 60ksi\ \ GOOD$$

Check Shear

$Q=(A/2)y=(1.06ft^2)(1.84ft)=1.95ft^3$

$$\tau = \sigma_y = \frac{VQ}{It} = \frac{(252.7kip)(1.95ft^3)}{(7.57ft^4)(0.0625ft)} = 1042ksf = 7.23ksi < 60ksi\ GOOD$$

Therefore, section chosen works and is way over designed so should be modified if concept chosen.

Design for stiffeners in the direction of deck width (30ft span, 1ft segment)

*Estimating by looking at the top plate as a bending beam and stiffeners would be supports- find max deflection)

$A=0.0625\ ft^2$ for 1 ft segment $\qquad I=2.034*10^{-5}\ ft^4$

LL=(158psf)(1ft)=158lb/ft

DL(concrete)=(1500lb/ft)*(1ft)/(30ft)=50 lb/ft $\qquad$ DL(steel)=$(483.84pcf)(0.0625ft^2)$=30.24 lb/ft

$$w_u = 1.2DL + 1.6LL = 312.8psf = 1.2(50+30.24) + 1.6(158) = 349lb/ft = 0.349kip/ft$$

Max deflection allowed: $\Delta_{allow} = \frac{L}{360} = \frac{(30ft)(\frac{12in}{ft})}{360} = 1in = 0.083\ ft$

$$\Delta_{max} = \frac{5w_u L^4}{384EI} = \frac{5(0.349kip/ft)(30ft)^4}{384(29000ksi)(144)(2.034*10^{-5})} = 43.3\ ft \rightarrow NO\ GOOD$$

$$\frac{L}{360} = \frac{5w_u L^4}{384EI} \rightarrow \frac{L}{360} = \frac{5(0.349kip/ft)L^4}{384(29000ksi)(144)(2.034*10^{-5}ft^4)}$$

$L = 3.73\ ft$ $\qquad$ For symmetry, use 3ft spacing between stiffeners

Check vehicle loads

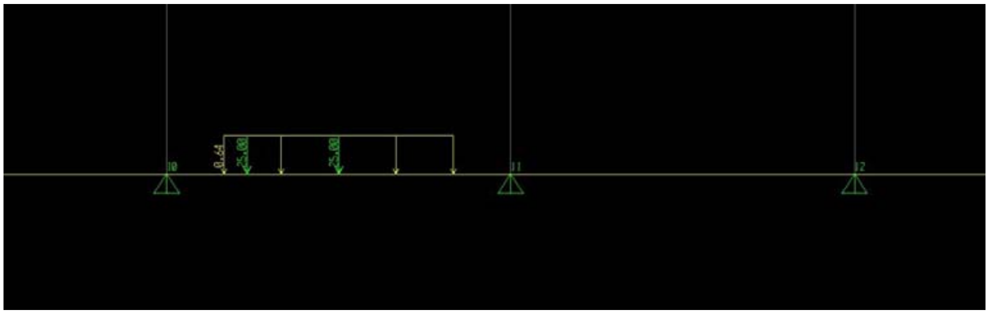
Along 30ft span of deck

Load Distribution

Max Shear=31.53 kips

$$\tau = \sigma_y = \frac{VQ}{It} = \frac{(31.53kip)(1ft*\frac{0.0625ft}{2}*\frac{0.0625ft}{4})}{(2.034*10^{-5}ft^4)(1ft)} = 678ksf = 4.7ksi < 60ksi\ GOOD$$

Along 390ft span of deck



Max Shear=35.1 kips

$$\tau = \sigma_y = \frac{VQ}{It} = \frac{(35.1kip)(1.95ft^3)}{(7.57ft^4)(0.0625ft)} = 144.7ksf = 1.0ksi < 60ksi\ GOOD$$

For construction, how much can be delivered at a time and how much can cantilever?

Weight per ft (just steel since concrete seats delivered later)

=(483.84pcf)(4.23ft^2)=2046.64lb/ft

Assume crane capacity of 60 tons (120,000lbs=120kips)

Central Span: Construct in sections that are 30 ft (61,380lbs)

*Could go larger but for symmetry and safety do this for the central span

Outer Span: Construction in sections that are 50 ft (102,332 lbs)

During construction just consider dead load

$$M_p = \phi F_y Z_x = 0.9(60ksi)(144)(7.852ft^3) = 6107\ kip - ft$$

$$M_{max} = \frac{w_u L^2}{2} \rightarrow 6170\ kip\ ft = \frac{(\frac{2.46kip}{ft})L^2}{2}$$

$$L = 223\ ft \rightarrow OKAY\ (only\ needed\ 50ft)$$

Summary:

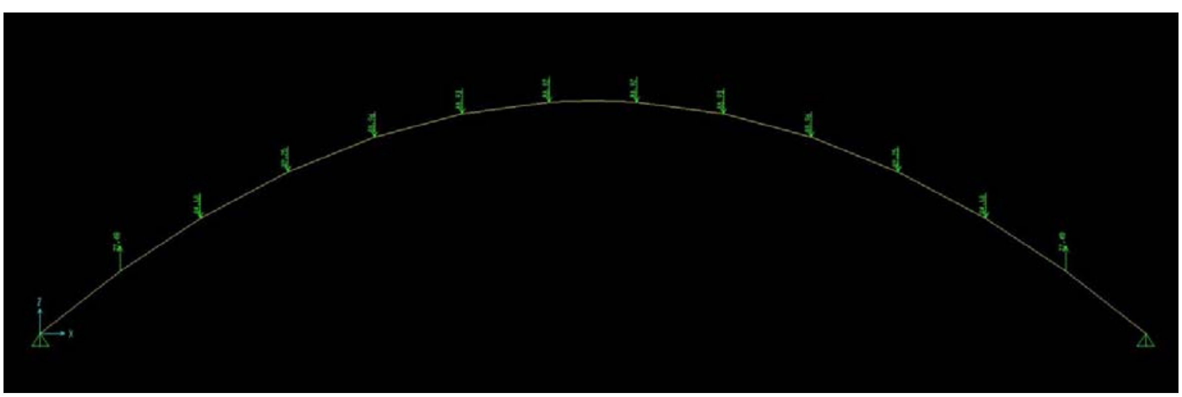
Box Section 30'x4' with 0.75" thickness and stiffeners every 3 ft

Construct central span in 30ft segments and outer spans in 50ft

Arch

Model the arch in SAP200 and applied a point load for each cable (this load was taken from the previous model of the deck and equals the vertical reaction of the support that was put in place of the cable)

3 points on the arch: (100,0) (290,0) (195,40)



From this model, the following values were taken

Vertical Reaction: 363.24 kips

Horizontal Reaction: 699.86 kips

Max Moment: 1439 kip-ft

Max Shear: 82.5 kips
Max Axial: 695.7 kips

$$\sigma_y = \frac{My}{I} = \frac{(1439\ kip - ft)(R)}{\frac{\pi}{4}(R^4 - (R-t)^4)} \rightarrow Let\ t = 1in = 0.083\ ft$$

$$60ksi = \frac{(1439\ kip - ft)(R)}{\frac{\pi}{4}(R^4 - (R - 0.083)^4)} \rightarrow R = 1.48\ ft \rightarrow round\ to\ 1.5\ ft$$

Check Axial:

$$\sigma_y = \frac{P}{A} = \frac{695.7kips}{\pi(R^2 - (R-t)^2)} = \frac{695.7kips}{\pi(18^2 - (18-1)^2)} = 6.3ksi\ GOOD$$

Summary: Use steel tube with outer radius of 1.5 ft and thickness of 1in (0.084 ft)

Cables

Max reaction for any cable (from SAP model of deck) = 89.25 kips

$$\sigma_y = \frac{P}{A} \rightarrow 60ksi = \frac{89.25kips}{\pi(R^2)} \rightarrow R = 0.69in \rightarrow round\ to\ 0.75\ in\ (d = 1.5in)$$

Check tensile strength

$0.9F_yA_g \leq 0.75F_uA_e$
$A_e = UA_n = (1.0)(\pi * 0.75^2) = 1.77$
$0.9(60ksi)(1.77in^2) \leq 0.75(75ksi)(1.77in^2)$
$95.58 \leq 99.56\ \ GOOD$

Summary: Use solid cables with diameter=1.5 in

Compression/Tension Rods

Again, from SAP Model max reaction for one of these rods is 77.21 kips (compression) or 49.5 kips (tension

Check Euler Buckling (Use Steel Manual to pick HSS Member)

K=0.65 (fixed-fixed connections)
L=17.5 ft (for longest)
KL=11.3 → 11.5 ft

Want $P \leq \phi P_n$ where P=77.21 kips
Using Table 4-5 , HSS 5x0.25 ($\phi P_n = 82.45\ kips$)

Check the Same section for tension (Table 5-6)

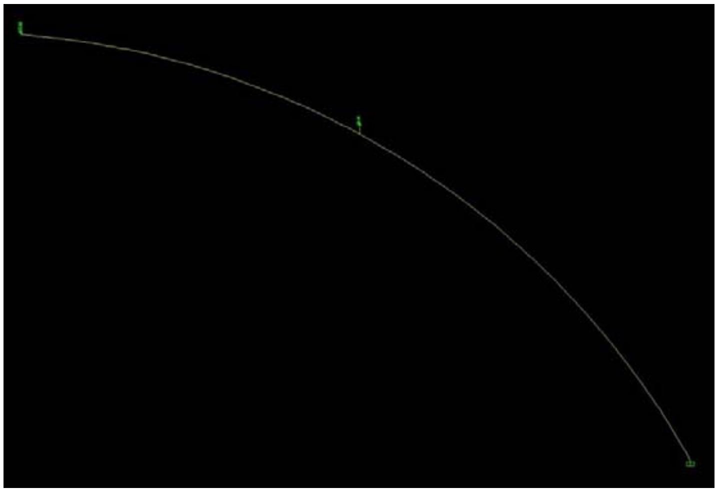
Yield: $\phi P_n = 132\ kips\ \ GOOD$
Rupture: $\phi P_n = 114\ kips\ \ GOOD$

Summary: Use HSS 5-0.25

Outer Support

Like the arch, the outer support was modeled in SAP and load with point loads where tension/compression rods are

However, there will be 3 of these supports instead of 2 so take the reaction found for the corresponding support modeled in the deck model multiply it by 2/3 (since that was for if there were 2 only). This will be the value of the point load applied here



From this model, the following values were taken

Max Moment: 7820 kip-ft
Max Shear: 300 kips
Max Axial: 233 kips

$$\sigma_y = \frac{My}{I} = \frac{(7820\ kip - ft)(R)}{\frac{\pi}{4}(R^4 - (R-t)^4)} \rightarrow Let\ t = 1in = 0.083\ ft$$

$$60ksi = \frac{(7820\ kip - ft)(R)}{\frac{\pi}{4}(R^4 - (R - 0.083)^4)} \rightarrow R = 1.9ft \rightarrow Round\ to\ 2\ ft$$

Try to get outer radius to match the arch, so increase t=1.75 in (0.125ft)

$$60ksi = \frac{(7820\ kip - ft)(R)}{\frac{\pi}{4}(R^4 - (R - .146)^4)} \rightarrow R = 1.5ft$$

Check Axial:

$$\sigma_y = \frac{P}{A} = \frac{233\ kips}{\pi(R^2 - (R-t)^2)} = \frac{233kips}{\pi(18^2 - (18-1)^2)} = 2.1ksi\ GOOD$$

Summary: Use steel tube with outer radius of 1.5 ft and thickness of 1.75in

APPENDIX D: CABLE-STAYED BRIDGE CALCULATIONS

Width (b) = 12ft
Height (h) = 1ft
Thickness (t) = 0.020833ft (.25in)
Deck spans (L) = 20ft
E_{steel}=210GPa (4.41*10^9psf)
LL = 158psf
DL = 428.5pcf
Cross sectional Area of box girder (A) = $(b*h)-((b-2t)*(h-2t))$ = 0.54ft^2
Governing load (w) = 158psf (pedestrian)

Moment imposed by load = $\frac{wl^2}{8}$= 7900lb-ft

$\Delta_{allowable} = \frac{L}{360}$= 0.667in

$I_{required}$ = $\frac{5*[(1.2*A*DL)+(1.6*LL*h)]*L^4}{384*E*\Delta}$ = 0.0281ft^4

$I_{section}$= $\frac{bh^3}{12}-\frac{(b-2t)(h-2t)^3}{12}$= 0.123ft^4 *(greater than Ireq)*

Δ_{actual} = $\frac{5*[(1.2*A*DL)+(1.6*LL*h)]*L^4}{384*E*I_{section}}$ = 0.153in *(less than allowable)*

$\Delta_{LL,allowable}$= $\frac{L}{1000}$= 0.24in

Δ_{LL}= $\frac{5*(1.6*LL*h)*L^4}{384*E*I_{section}}$= 0.14in

Tension in cable = $\frac{[DL(A)+LL(b)]*L}{2}$ = 21273.8lbs (96.4kN)

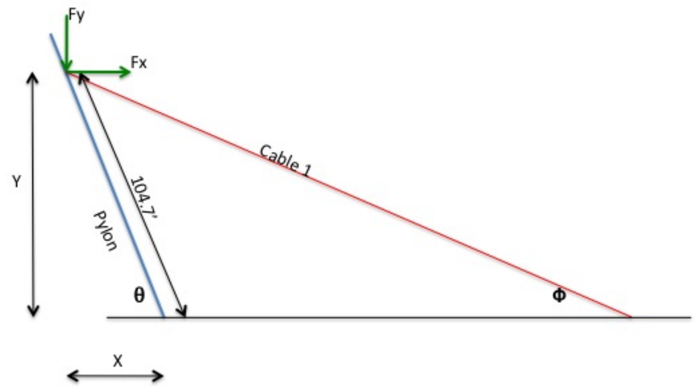
Range of cables = 22.4 – 48.9degrees

Yield Stress, σ = 5221357.5psf (250MPa)

Height of tower above river (Ht) = 90.8ft

Length of tower (Lt) = 109.1ft

Angle of tower to the horizontal, θ = $\sin^{-1}(\frac{Ht}{Lt})$ = 56.4 degrees



ϕ = Cable angles

X = Location on tower * $\cos(\theta)$
Y = Location on tower * $\sin(\theta)$

Fx = Tension in cable * $\cos(\phi)$

Fy = Tension in cable * $\sin(\phi)$

Moment imposed on tower (Mo) = Fx(Y)+Fy(X)

	Degrees	radians	Location on Tower	X	Y	Fx, lbs	Fy, lbs	M, lbs-ft
Cable 1	22.40	0.39	104.70	57.98	87.18	19670.25	8102.93	2184648.56
Cable 2	23.10	0.40	103.70	57.43	86.35	19569.84	8342.52	2168872.20
Cable 3	25.40	0.44	102.70	56.88	85.51	19219.46	9120.78	2162259.76
Cable 4	28.10	0.49	101.70	56.32	84.68	18768.71	10015.56	2153432.96
Cable 5	31.50	0.55	100.70	55.77	83.85	18142.02	11110.49	2140774.85
Cable 6	35.90	0.63	99.70	55.21	83.01	17236.65	12468.92	2119361.60
Cable 7	41.50	0.72	98.70	54.66	82.18	15938.34	14090.62	2080049.31
Cable 8	48.90	0.85	97.70	54.11	81.35	13991.83	16025.13	2005298.16
SUM								17014697.39

Weight of tower needed to negate Mo = $\dfrac{Mo}{(Ht/2)*\sin\theta}$ = 749203.2lbs (3332.62kN)

Area of cable = $\dfrac{[(1.2*A*DL)+(1.6*LL*b)]*L}{\sin\Phi*\sigma}$ = 0.033ft^2

Diameter = 0.21ft (2.47in)

APPENDIX E: UNDERPASS CALCULATIONS

The below results are the result of much iteration; final dimensions are:
Columns – HSS 2ft diameter, .75in thickness
Deck – 12ft wide, 2ft deep, 1.5in thickness
Cables – 2in diameter

Loading:
The team considered both service load and construction loads during design. The analysis showed that service loads governed and deflection was the limiting criteria.

$$\text{Service load} = 1.2\text{ D} + 1.6\text{ L (where L} = 158\,\frac{\text{lbs}}{\text{ft}^2}\text{)}$$

It was assumed the dead load deflection could be cambered out. Therefore, deflection criteria were compared against live load deflections only. (L/1000 being the limit)

Columns:
Design compressive strength for flexural buckling (from Chapter E section E2 in the Manual of Steel Construction):

$$P_u \leq \varphi P_n = \varphi A_g F_{cr}$$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \quad and \quad \lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}$$

The maximum allowable load for the columns is 391kips.

SAP analysis with the entire bridge loaded at the service load (See figure 30) yielded a maximum base reaction of 284kips. (73% of allowable)

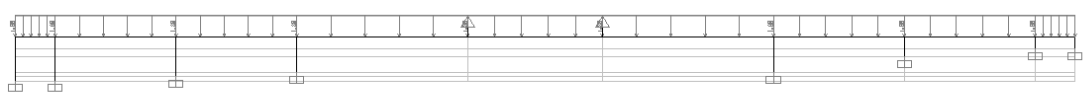


Figure 30: Underpass model with full service load.

Deck:
The analysis shows:

$$M_u \leq \varphi M_n = \varphi M_p = \varphi F_y Z$$

The maximum allowable moment in the deck is 2.34 x 104 kip-ft.

SAP analysis with the bridge loaded with the service load on every other bay (See figure 31) yielded the maximum moment of 1.20 x 103 kip-ft. (5% of allowable)

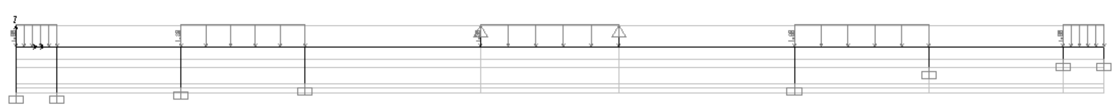


Figure 31: Underpass model with every other bay loaded

Deflection was 100% of allowable with these dimensions. This is the limiting factor.
Analysis of construction loads (dead load only) showed the maximum allowable cantilever length of 165ft. Since the longest span is 85ft in length, this is not a limiting factor.

Cables:
The maximum allowable load the cables can hold is:

$$P_u \leq \varphi F_y A$$

or 160kips.

SAP analysis with the bridge loaded with the service load on every bay yielded the maximum load in the cables to be 131kips. (82% of allowable)

Construction:
The longest deck section during transport will be 50ft in length and weigh 19 tons.

The longest column will be 50 ft in length and weight 8 tons.

Either of these are easily transported by truck and lifted into place by crane.

APPENDIX F: OVERPASS CALCULATIONS

Steel Box Girder

Width (b) = 12ft Height (h) = 1ft Thickness (t) = 0.05ft (6in) Slab length = 40ft

E_{steel}=210GPa (4.41*10^9psf)

Cross sectional area of slab (A) = $(b*h)-((b-2t)*(h-2t))$ = 1.25ft^2

Yield Stress, σ = 5221357.5psf (250MPa)

$I_{section}$= $\frac{bh^3}{12}-\frac{(b-2t)(h-2t)^3}{12}$ = 0.28ft^4

Moment capacity = $\frac{\sigma(I)}{(h/2)}$ = 2890453.5lb-ft

Mmax = $\frac{w(l\text{^}2)}{8}$ = 739538.4lb-ft

$\Delta_{allowable} = \frac{L}{360}$ = 1.33in

$\Delta_{actual} = \frac{5*[(1.2*A*DL)+(1.6*LL*h)]*L^4}{384*E*I_{section}}$ = 1.22in *(less than allowable)*

$\Delta_{LL,allowable} = \frac{L}{1000}$ = 5.67in

$\Delta_{LL} = \frac{5*(1.6*LL*h)*L^4}{384*E*I_{section}}$ = 1in

Column

Height = 16ft Diameter = 1ft Radius = 0.5ft Thickness = 0.05ft

K = 0.65

I = $(\frac{\pi}{4})*((r\text{^}4)-(r-0.5t)\text{^}4)$ = 0.017ft^4

Pcr = $\frac{\pi^2 EI}{KI}$ = 70157009.74lbs

Moment Capacity = $\frac{\sigma(I)}{r}$ = 176195.7 lb-ft