Myra Hiatt Kraft Memorial Bridge Restoration in Historic Elm Park

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A Major Qualifying Project Report:

Submitted to Faculty of

WORCESTER POLYTECHNIC INSTITUTE

in partial fulfillment of the requirements for the

Degree of Bachelor of Science By

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Abstract

In 2013, the City of Worcester announced its intentions to work with students from Worcester Polytechnic Institute to refurbish the historic Elm Park Red Wooden Footbridge. Along with apparent accessibility concerns, the bridge contained many structural defects. After extensive structural evaluation and site assessment, the WPI project team was able to design an optimized and accessible footbridge and landscape that retain the historical integrity inherent to the park. The design will be implemented in the park in the coming months.

Authorship

Alexandra Jaeger:

Alexandra's initial role in this project was to conduct research as well as act as the liaison between the project group and the City of Worcester and the Worcester Technical High School. She also helped conduct a preliminary assessment of the bridge as a basis to work from in creating the new design. As the project progressed, Alexandra worked with Matthew Steeves and Courtney Verdel-Ogden on the bridge design. Working first in SolidWorks CAD software to create a model of the bridge, Alexandra took the original architectural design and analyzed the members. She continued Matthew Steeves' work in the STAAD finite element analysis software to model the final bridge design after consultation with the Professional Engineer. Using this information she compiled data to be input into Matthew Steeves' spreadsheets. Alexandra also had a significant role in the writing of the sections mentioned above, as well as in editing of the final document.

Nicole Martucci

Nicole's contribution to this MQP revolved around research, writing and onsite examination of the bridge. Nicole initially researched Elm Park and the Hiatt-Kraft family history. Nicole also conducted an initial assessment of the existing bridge conditions with Lamyae by taking measurements of the members and investigating the connections and foundation. Once the bridge was moved, Nicole did a condition assessment of the entire bridge, which involved checking each member individually for rot and decay. Nicole obtained samples for wood testing to determine the strength of the wood that was in the original design. In addition, Nicole also conducted research on alternative materials and other aesthetic properties of the new bridge design. Once a wood type was chosen, Nicole completed a materials cost analysis for the new bridge based on data provided by the associated lumber company. In the final stages of the project, Nicole focused on editing the final document and writing the first draft of the executive summary and the capstone design

Lamyae Reklaoui

Lamyae worked primarily on the condition assessment and conducted research on the aesthetic, architectural, and material options for the new footbridge design. She visited the bridge on site in Elm Park as well as at the WTHS to perform a pick test and sound test on each individual timber member for determining where points of failure had occurred. These results were used to help improve and optimize the new design. Additionally, Lamyae studied the physical and chemical properties of the wood specimen used for the Red Wooden Footbridge in order to obtain values that would be necessary to complete Matthew Steeves' design calculations. Furthermore, she coordinated with Koopman Lumber, Co. Inc. to acquire information such as cost and availability of the lumber pieces that would be used for construction. Like the rest of the team, Lamyae contributed to the writing and editing of certain sections of the paper.

Erik Snodgrass

Erik worked chiefly on the site design and abutment design. Erik took the lead on the site design, creating conceptual models of varying site plans with the help of Matt Valcourt. He relied heavily on his skills with different CAD software to accomplish creating site plans for the

Worcester Parks Department. Erik not only generated detailed abutment design drawings, but also created a conceptual drawing of the Myra Hiatt Kraft Memorial Footbridge. Erik was one of the key representatives relaying information between the project group and the Parks Department regarding critical site design constraints. Erik had a significant role in the writing of the sections mentioned above, and also served as one of the lead editors of the document. In addition to this, Erik worked with Matt Valcourt to complete a significant amount of historical background research on Elm Park which served as a foundation for the project.

Matthew Steeves:

Mathew's initial role in the project involved researching modern accessibility requirements for the bridge. Using this understanding of ADA and AAB compliancy codes, he assisted in the accessible conceptual design of the bridge. The remainder of Matthew's work focused on the structural design of the bridge. He began by modeling the iterations of the bridge in design software. He then set up numerous spreadsheets to interpret the results of the analysis and tabulate design values for the bridge and railing members as well as the connections. Matt played a significant role in ensuring that the bridge design and connections were adequate. He also wrote on all of sections the aforementioned material.

Matthew Valcourt:

Matt worked predominantly on the foundation analysis, site design and abutment design aspects of the project. Matt took the lead on the foundation analysis calculations and was the lead designer for the new reinforced concrete abutments. Matt was also served as lead consultant for the site design portion of the project, and worked significantly with Erik to complete the final designs. Matt contributed heavily to the writing portions of the previously mentioned aspects, and also served as one of the lead editors for the entire document. Furthermore, Matt worked with Erik in completing the historical research portion of the project and was responsible for the historical summary of the bridge and park. Matt also worked with Erik in creating the conceptual design of the Myra Hiatt Kraft Memorial Footbridge.

Courtney Verdel-Ogden:

Courtney initially took the role of producing the CAD designs of the original bridge as well as multiple alternative designs suggested by the group. To do this, she worked with Lamyae and Alexandra to perform an initial analysis of the bridge design and used the information gathered to develop comparable designs. She also focused on meeting the guidelines and requirements of the state historical commission by preparing a multi-page project notification form. Courtney also worked with Alexandra and Matt S. to perform initial site surveys of Elm Park. Once a final design was chosen she worked with Alexandra and Matt S. to perform a structural analysis of the bridge, and make final decisions regarding connections, and railing designs. Through the final terms of the project Courtney took a significant role in combining, writing and editing multiple sections of the final document.

Acknowledgements

The Myra Hiatt Kraft Memorial Bridge team would like to express its sincere gratitude to everyone who helped us throughout the year during the MQP process.

Specifically, we would like to thank Prof. Tahar El Korchi for his helpful contributions to this project. Without his guidance and support this project assuredly would not have reached its full potential.

Thank you to Professors Nima Rahbar and Rajib Mallick for their continuous consultation throughout the year. Their help and guidance in the bridge, abutment design and foundation analysis portions of the project proved to be invaluable in the final designs.

The team would also like to extend our thanks to the many institutions that assisted during the historical research phase of the project. Representatives from the Worcester Historical Museum, Olmstead Archives, American Antiquarian Society of Worcester, Worcester Public Library, WPI Library and Archives and Preservation Worcester all provided invaluable tools and information that contributed to a very thorough historical background and a sound understanding of the colorful history of Elm Park and the Red Wooden Footbridge.

We would like to acknowledge the hard work of the Worcester Technical High School. In particular, Kyle Brenner, The Director of Vocational Education, Joseph Lonergan, Department Head of Carpentry Department, and Paul Chambers, Department Head of Computer Aided Drafting and Design Department; all of whom worked with us and provided input from the day the project was conceived to the day our design was completed.

Additionally, we would like to thank the Worcester Parks and Recreation team, specifically Rob Antonelli, Bill Richard and Cesar Valiente. Their immense support and guidance throughout the site design portion of the project was extremely helpful. Without them we would not have been able to create the same final product. They were readily available and willing to help us in any way they could throughout the entire MQP process.

Furthermore, we would like to acknowledge Francis Steven Harvey Jr. P.E. for working with us to finalize our bridge design. Mr. Harvey worked with the team for months, providing invaluable feedback and helping us follow the proper steps of a professional design. With the guidance and help of Mr. Harvey the group was able to design, analyze, and produce results for a structurally stable bridge.

Lastly, we would like to extend our thanks to the City of Worcester and all those who were involved in the design process. Everyone involved was extremely supportive and provided instrumental guidance every step of the way that lead to a final product we are all very proud of.

Capstone Design

WPI requires all students to complete a Capstone Design Experience for their Major Qualifying Project (MQP). This experience is encouraged to provide the students a chance to reach a synthesis experience in solving an open ended design problem that is multifaceted and poses a number of constraints. It is required by each department in order for students to gain real-world design or research experience within their major field. In completing this experience students learn to practice the knowledge gained through their years studying different skills in prior classes. This MQP has followed the requirements set forth by the Civil and Environmental Engineering Department. In order to succeed in meeting the requirements for this MQP, three alternative bridge designs as well as a corresponding site design were developed to find an economic, environmental, sustainable, manufacturable, ethical, social and political solution that also addresses health and safety factors.

The Red Wooden Footbridge in Elm Park was deemed structurally unsound for pedestrian traffic in 2013. As a result, this MQP was initiated through a partnership with the City of Worcester and the Worcester Technical High School (WTHS) in order to redesign and construct a new footbridge that is accessible for all users and adheres to the Americans with Disabilities Act (ADA) criteria. Three designs were developed, each consisting of its unique architectural design features and functionality. These three designs consisted of an unaltered replication design, a replication with an additional switchback, and an ADA compliant design with corresponding site grading. Utilizing multiple civil design programs, such as AutoCAD and Solidworks, the proposed designs were created and further analyzed to select one that would satisfy all the requirements for a pedestrian footbridge, while also preserving the historical integrity inherent to the park. This design was then evaluated for its structural strength based on international, national and state building codes and design requirements. The surrounding landscape was also redesigned in order to account for the new ADA compliant bridge design and meet accessibility requirements.

In the process of redesigning this bridge and the site landscape, these constraints were addressed:

Economic: Cost was taken into consideration as a key factor in the bridge design. A complete cost analysis was made for the chosen bridge and corresponding landscape design. This analysis took into consideration the associated raw materials and labor costs for each aspect of the project. Additionally, cost of work also played a vital role in the site and abutment design process. Both of these aspects of the project were designed in a manner that would yield the most cost effective design possible.

Environmental: The bridge design considered the utilization of renewable and biodegradable materials, as well as non-volatile paint without lead material hazardous to the environment. In addition, a soil analysis was completed in accordance with accepted standards to assess existing site conditions. Furthermore, the new site design relied heavily on the existing conditions in the park. Considering walkways, a proper cross-pitch needed to be implemented in order for storm water runoff and drainage to occur correctly. Aspects such as location of additional pond walls and elimination of some existing shrubbery and trees were also important as so any changes would not be detrimental the park atmosphere.

Sustainable: Not only was creating an accessible design important, but it was also essential to develop a design that would be sustainable over an extended period of time and did not need to be consistently maintained by the city Parks and Recreation department. Many different types of wood were studied in order to find a species that would be stronger and more durable than the specimen used for the current structure. Other properties were also studied such as rot resistance and vulnerability to insect attack. The site design also considered longevity of recommended solutions and avoidance of current issues such as erosion at the bridge base on both sides as well as erosion around the mere edges.

Constructability & Manufacturing: This bridge design was created with constructability in mind. During the design phase, communication with the WTHS was essential to ensure that the wood structure proposed in the design was manufacturable and constructible with the idea that one day it would be manufactured and built as a replacement in Elm Park by the students of WTHS. All calculations were performed in order for the footbridge to be safe for pedestrians to utilize. The design was also optimized to be as simple as possible so that it could easily be replicated and constructed by senior students at the WTHS and their staff.

Ethical*:* There are many fundamental canons that an engineer must follow in order to make sure that the structure is safe and built to the correct standards. The first code of ethics from the American Society of Civil Engineers (ASCE) states that "Engineers shall hold paramount the safety, health and welfare of the public and shall strive to comply with the principles of sustainable development in the performance of their professional duties" (American Society of Civil

Engineers, 2014). The design for this footbridge was made upholding these standards. Both the footbridge design and the site design adhere to the requirements set forth by the Americans with Disabilities Acts (ADA) and the Massachusetts Architectural Access Board (AAB) regarding accessible structural design.

Social: The footbridge and site designs that were chosen were developed based on the needs and input of the city and its citizens. The project itself demanded teamwork and communication at many different levels with many professionals in Worcester. Numerous meetings were held between the project group and different city organizations and entities in order to build a footbridge that the community as a whole would enjoy and benefit from. Among the groups that were involved in this project were the Worcester Historical Commission, WTHS, Worcester City Manager's Office, and Worcester Parks and Recreation Department. All of these groups, as well as other additional stakeholders were involved in the design process. The new bridge also acts as an integral part of the parks enhancement plans which will improve the quality of the park and its use by the community. By offering the community an aesthetically appealing park, the citizens who use the park will be able to enjoy outdoor recreation in a safe and beautiful place.

Political: Building codes have changed significantly since this footbridge was last renovated in 1972. One of the major political concerns for the new design was addressing the compliance of accessibility requirements for disabled persons. After much discussion with the different stakeholders involved in the project, it was decided that an accessible design should be implemented so that all the citizens of Worcester can enjoy the iconic bridge. ADA and AAB codes were studied in order to develop an accessible design that can be safely traversed by the disabled citizens of Worcester in the future, while still keeping in touch with the historical aesthetic features. Throughout the process, a delicate balance had to be met in order to satisfy ADA safety requirements and the needs of local advocates for a purely historically accurate bridge.

Health and Safety: A major factor that always plays a role in any engineering project is the idea of health and safety for the general public. In order to ensure the safety of those traversing the bridge, the International Building Code (IBC) and Load and Resistance Factor Design (LRFD) design specifications for a footbridge were taken into consideration. Using structural analysis software, the final bridge design was completely analyzed and cross-checked with stop point hand calculations to ensure that it could support maximum loading conditions. Furthermore, appropriate factors of safety were incorporated in both the footbridge and site design. Lastly, this project was then presented to a professional engineer for approval to ensure the adequacy of the design.

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Executive Summary

In 2013, the city of Worcester created a task force consisting of representatives from the Worcester Parks and Recreation Department, WPI, and WTHS to help refurbish the historic Elm Park Red Wooden Footbridge. The new bridge, which will be named the Myra Hiatt Kraft Memorial Footbridge in honor of the late Myra Kraft, a native of Worcester, will incorporate both historical and new design aspects. The design of the new bridge and site posed many challenges for the MQP group, the most important of which was meeting the current standards of accessibility requirements for all residents, while still preserving the historical integrity of the storied bridge and park. Unfortunately, the existing Elm Park Red Wooden Footbridge was not only structurally unstable, but also extremely steep which hindered the ability of some citizens to traverse the bridge safely. Ultimately, the project group utilized a multi-step approach to produce an accessible bridge and site design conforming to the historic prominence of one of the oldest public parks in the United States.

The Elm Park Red Wooden Footbridge has long been a local icon portrayed on postcards, pamphlets, and even wedding photos in Worcester. Dating back to 1877, the bridge itself is one of the oldest structures in Elm Park. It has seen multiple reconstructions since its inception, due mainly to the harsh weathering effects on the wood. The most recent reconstruction of the bridge took place in 1972 after Elm Park was named a National Historic Landmark. The bridge has since deteriorated significantly, forcing its closure from the public and triggering the new Myra Hiatt Kraft Memorial Footbridge project. As was the case in 1972, the city has made it a top priority to preserve the historical integrity of the bridge in the new design as much as possible.

As a first step for the project, it was important to conduct an initial assessment of the existing Elm Park Red Wooden Bridge as it stood. This helped the team gauge any structural issues the current bridge contained, as well as what changes would need to be made to avoid future problems. To begin, the team visited the site at Elm Park and measured each piece of the bridge for the purposes of creating a replication AutoCAD drawing. Additionally, team members conducted a preliminary site survey around the bridge to gather a rough estimate of ways in which the park might need to be altered to adhere to accessibility standards. Finally, once the existing bridge was relocated to the Worcester Technical High School, a more involved condition assessment was completed to determine which members had undergone the greatest deterioration over the years, and reasons why the structural wear had occurred. It was vitally important that the new design incorporate means to combat the issues found in the initial condition assessment.

The original bridge was constructed of Douglas fir wood in 1972. Over the past 42 years the existing bridge conditions have declined significantly due mainly to exposure to weather, water and constant use in the park. Due to the proximity of the bridge to the pond, water and rain often pooled around the connections of the bridge, which contributed substantially to the rotting of the members. Because of this, great consideration was put into the member properties and locations, as well as the connections from member to member and member to foundation, in order to increase the longevity of the bridge.

One of the most important considerations for the new Myra Hiatt Kraft Memorial Footbridge involved the issue of accessibility. For the past 136 years, the Elm Park Red Wooden Footbridge has retained a slope well above the acceptable limits set forth by ADA. After extensive discussion with the city and many other local stakeholders, it was decided in November of 2013 that rather than keeping the same design and pursuing historical variances, the new Myra Hiatt Kraft Memorial Footbridge would be made accessible for all citizens, disabled or not, to cross. Ultimately the redesign will incorporate new accessibility aspects that adhere to current accessible structural design standards, as well as many historical aesthetic features that were seen on the bridge through the years. The bridge with overall dimensions can be seen in the image below.

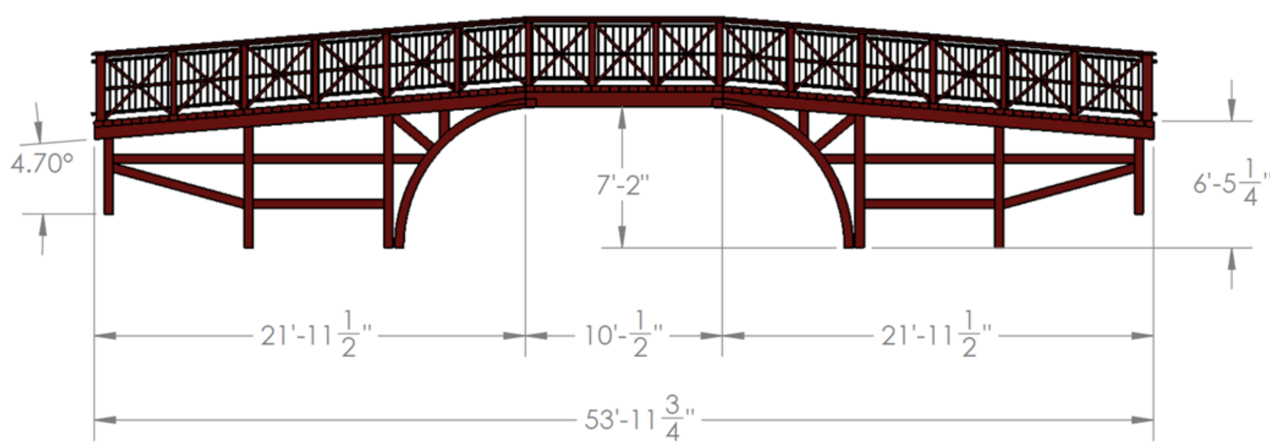


Figure 1: Final bridge design with overall dimensions noted

Utilizing finite element analysis software, STAAD, the new bridge was designed and analyzed extensively in order to determine if it was safe to implement into the park for future use by pedestrians. As part of the design process, structural building codes provided by the International Code Council and Massachusetts AAB were referenced; and the LRFD procedure for wood design was followed. In STAAD, the bridge was modeled and analyzed for maximum loading conditions that it may at one point sustain in its lifetime. The loads considered for this bridge were the dead loads of the members themselves, the pedestrian live load, and the local wind and snow loads. After all loads were applied and results were collected and finalized, a professional engineer for the city, Steve Harvey, was given all results to ensure that the bridge was structurally sound. He provided feedback regarding our design analysis and suggested some minor changes. Mr. Harvey was ultimately responsible for approving the final design of the bridge.

In order to accommodate for the new elevation of the Myra Hiatt Kraft Memorial Footbridge, changes need to be made to the site adjacent to the bridge. The new design called for a change in elevation of about six feet at the landing of the bridge. As a result, a new abutment needed to be designed and added to the existing foundations in order to make up for the height difference. The additional abutment piece was not considered as part of the original project, and so it was important to design a practical and cost effective piece. Utilizing current best practices for design of reinforced concrete structures, the group was able to produce a design that adequately addresses the maximum soil pressures and live loads that would be experienced at the landings on both sides of the bridge. Shop drawings and a cost analysis were developed as part of the design process. The abutment design can be seen in the figure below.

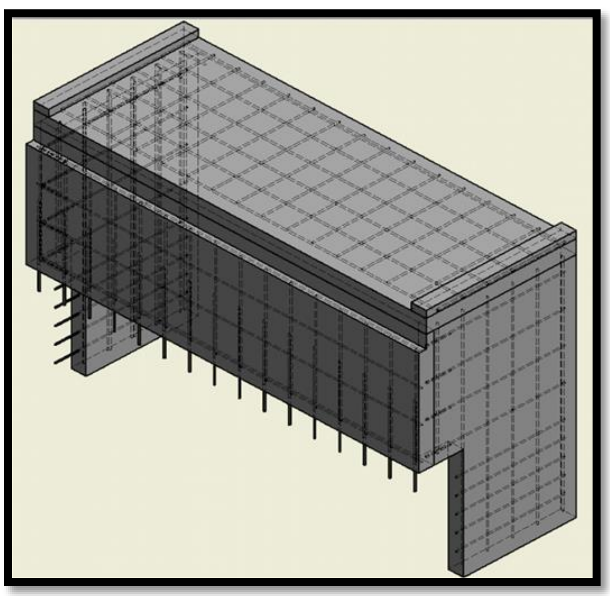


Figure 2: Final abutment design drawing

In addition to the new abutment, extensive site grading was also required as a result of the changes made to the bridge design. Like the bridge, the site needed to be graded in a manner that would meet the accessibility requirements set forth by ADA. The site design itself also posed additional challenges, including integrating the new site design with existing landscape features, and addressing the ambiguous boundary line of the South Mere pond. The project group worked diligently with the Worcester Parks and Recreation Department to develop multiple design options that explored different ways to address the aforementioned challenges. Ultimately, a final design was chosen that meets the accessibility requirements set forth by ADA and reinstates pond walls in the South Mere that were seen in the original park design in the late 1800's. Upon finalizing the new site design for the Myra Hiatt Kraft Memorial Footbridge, the project group then worked with representatives from Beal's and Thomas, a local engineering firm, to produce final engineering plans and a cost analysis for the total cut and fill that would be required in the park.

The final design, which can be seen in the rendering below, optimizes the previous design in many ways. The columns were changed from four double 4 x 6 columns to three single 6 x 6 columns. This decreased not only the required materials for the bridge, but the number of connections, thus minimizing the overall cost of the project. Additionally, the concealed connections were designed to increase the longevity of the bridge by decreasing the likelihood of pooling at the connection points, a reoccurring issue with previous historical designs. Furthermore, the railings were redesigned to more accurately reflect the style seen in the early years of the bridge, and still provide adequate safety measures for the public. Lastly, the most prominent change in the new bridge can be seen in the slope, which is now ADA compliant and much less steep then the previous design.

Figure 3: Final bridge and site design rendering

Over the course of the next year, the WTHS and the Worcester Parks and Recreation department will work together in order to implement the new bridge into the park. The first phase will include the pouring of the new abutments at the base of the bridge wings, as well as construction related to the new site grading that will be necessary for the new design. Concurrently, the lumber and connections will be sent to bid, purchased, and delivered to the WTHS. There, the pieces will be cut to the correct dimensions, and small assemblies will be created. Once the first phase of the project is completed, the WTHS will bring the partially assembled pieces to the park and construct the rest of the bridge on site to integrate with the new abutments. The city estimates that the entire project will be completed by the fall of 2014.

1.0 Introduction

Elm Park has long been an icon of the Worcester scenery, from its humble beginnings in 1856 to it being named a national historic landmark in 1982. The Elm Park Red Wooden Footbridge is one of many aspects of the park that create its picturesque landscape seen on postcards and frequented by local and out of state visitors. The bridge itself is one of the only structures in the park to remain in the same location since its inception, a testament to its historical significance and importance.

The bridge, as well as the park, have undergone extensive renovations throughout a nearly century and a half lifespan. Most recently, the city initiated a renovation plan in 2013 to completely refurbish the park and the Elm Park Red Wooden Footbridge. It was the responsibility of the WPI project team to provide design recommendations and options for the new design of the bridge as well as any site design alterations that could be needed to accommodate a new bridge structure.

In 2013, the Elm Park Red Wooden Footbridge was closed to the public due to structural concerns and fear of safe travel for the public. As a result, WPI was asked by the City of Worcester to help with the design of the footbridge and a group of seven students were assigned the task of designing the bridge and the park landscape. The MQP group worked in partnership with representatives from the Worcester Parks and Recreation and the WTHS to accomplish the task of redesigning the bridge. In order to complete the process of designing the bridge, a number of alternative designs were suggested, and a final design was selected by the City with input from all stakeholders including WTHS, the Worcester Historical Commission and Robert Kraft. The bridge was designed to preserve historical significance and ADA compliance, and update the current materials to ensure sustainability of the structure.

As a first step, it was important that the group developed a thorough understanding of the history of the bridge within the park. In this way, the historical integrity of the bridge and park could be retained and incorporated in the final design. Therefore, the history of the bridge was examined and documented from its earliest stages in the late 1800s to the present day when the bridge was closed and disassembled for complete reconstruction. With this historical information, the proper considerations were made during the design phase of the project.

In order to design a bridge that adhered to today’s structural standards, certain design regulations were considered. One of the greatest challenges in designing the bridge was ensuring the accessibility for all citizens of Worcester while still retaining its historical integrity. This required designing for all accessibility standards, outlined by the ADA and AAB. These regulations ensure that any structure is safe and accessible for disabled persons to access, including those in wheelchairs or crutches. During the feasibility and initial design stage, it was important to decide if ADA compliance or historical architectural preservation was more critical and must be retained. If it were decided that the bridge would be fully ADA compliant, the new design would be significantly different than the existing bridge design, including a much smaller overall slope and a different geometrical appearance. Consequently, the safety regulations described in the report played a significant role in the design process.

During the project, three options for the redesign of the Elm Park Red Wooden Footbridge were proposed to the project stakeholders. The three options included: an exact replica design of the existing bridge, an exact replica design with a switchback option including viewing platforms, and a new design of the bridge with a gentler slope to be in compliance with ADA requirements. All three design options address particular existing issues and were created to allow for a variety of choices for relevant stakeholders to consider.

A footbridge carries social implications because it is built for pedestrians to traverse, both safely and efficiently, as well as utilize for personal pleasure. Without this bridge, it would take the citizens of Worcester more than ten minutes to walk from one side of the park to the other. Ultimately, it is paramount to the fluidity of movement through the park and needs to be designed in such a manner. Furthermore, we will design a bridge that is structurally sound, aesthetically pleasing, and environmentally friendly, with sustainable materials and features. Developing a stable bridge that is able to withstand weathering and varying loading conditions was extremely important both for the safety of pedestrians and the longevity of the bridge itself. Color, lighting, and alternative construction materials were all aspects that contributed to the aesthetic appeal of the footbridge. Careful planning of these details enhanced the visual appearance of the footbridge, and allowed for it to be structurally sound while maintaining the historical significance of the park. This paper will examine the methods and approaches that were taken throughout the design process for both the bridge and surrounding site, and will provide final optimized designs that will be implemented in the park in the coming months.

2.0 Background

As part of the design process, initial research in areas related to the project was conducted. To begin, the history of Elm Park, code requirements, and accessibility were all explored allowing the group to utilize up-to-date procedures in designing applicable bridge and site designs. To ensure appropriate designs were developed, relevant project stakeholders were identified and consulted to ensure their goals for the project were met. Initial bridge and site assessments were also conducted so that the project group could establish the current failings of the structure and address them in the design process.

2.1 History of Elm Park

Elm Park has a long and storied history in the city of Worcester, spanning nearly a century and a half. Dating back to the 1870's, Elm Park has become an iconic feature in Worcester and one of the most heavily used parks in the city. It is estimated that over 40,000 patrons frequent the park each year, enjoying features such as the parks winding paths and picturesque views (Krueger, 1989). Its historical significance paved the way for its recognition as a national historic landmark in 1982 (City of Worcester Parks & Recreation Department, 2013). Over its nearly 160-year history, the park has undergone extensive renovations as well as acquisitions and compromises of land. Recent years, however, have seen the park fall into a state of disrepair. As a result, the Mayor and the City of Worcester made it a priority to restore the park to its original grandeur as part of its extensive community outreach agenda. In doing so, many of the structures will be refurbished within the park, including the Elm Park Red Wooden Footbridge, in a manner that contends with their historic predecessors. In order to most accurately portray structures like the bridge in their proper historic light, it is important to understand the history of the park and the structures that reside within the landscape. This history can be divided into four distinct time periods that encompass all of the major events that have occurred over the past 160 years (Favretti & Alexopoulous, 1985).

2.1.1 Period 1: 1850-1872

The first years of the park were in the second half of the nineteenth century. Today, many associate Elm Park with being the first purchase of land for public use in the United States. This is most likely due in part to the memorial plaque that sits at the main entrance of the park, which proclaims this title (City of Worcester Parks & Recreation Department, 2013); however, this is not entirely accurate. After a call for new “public grounds” by the mayor and other advocates in the city in 1854, Worcester followed the actions of other cities like Hartford and New York City who had already purchased land for parks earlier that same year (Rice, 1899). On March 15th, 1854 Worcester purchased 27 acres of land now known as Elm Park (previously referred to as the “New Common”) for a total of $11,250.00 (Rice, 1899). The land that was purchased was positioned in the center of the streets known today as Park Avenue, Highland Street, Russell Street and Elm Street. Worcester eventually became a pioneer in this new concept of purchasing spaces for the use of the general public, and by 1891 the city owned nine additional parks (Favretti & Alexopoulous, 1985).

The purchase of Elm Park initially failed to impress the general population of Worcester. Many of its citizens believed the city paid too much money for a dull, swampy area that is just outside of the city (Rice, 1899). They blamed Levi Lincoln, the governor at the time who sold the land to Worcester, for selling an inferior piece of land for the purposes of his own personal gain. Strong opposition to the purchase favored heavily over the course of the next year, and many tried to influence the city council to reverse its vote and give the land back to its original (Rice, 1899). The disapproval of the voters and lack of funds prevented any improvements being made on the land for the next twenty years. Instead, the land became a dumping ground for the city highway department and was used for different exhibitions such as circuses and growing apples (Favretti & Alexopoulous, 1985).

Fortunately, Edward Winslow Lincoln was voted chairman of The Commissioners for Shade Trees and Public Grounds (Parks Commission) in the year 1870, a position which he would hold until 1896 (Favretti & Alexopoulous, 1985). Lincoln made it a priority to realize the park’s potential. As the leading architect of the site, it was his “genius and vision that would help guide the park to becoming one of the most attractive pleasure-grounds in Worcester by 1880” (Rice, 1899). Almost immediately upon his arrival as commissioner, the park began to develop into its iconic form.

2.1.2 Period 2: 1873-1909

In February of 1873, The Worcester City Council approved an expenditure of $2,000.00 to begin work on Elm Park, after relentless lobbying from Lincoln. Wasting no time, they began work on the park in early 1873, utilizing an English, Victorian pleasure ground as a model to work from. By 1877, two ornamental pools had been constructed (see Figure 4), today known as “Elm Mere” at the North end and “South Mere” at the South end (Favretti & Alexopoulous, 1985). The joining of these pools by a narrow channel necessitated that a bridge be built that could be the link between the East and West sides of the park. The first version of the Elm Park Wooden Footbridge was built in 1877 over the narrow channel, in the same place it stands today. The bridge was built of cedar wood and Lincoln described the plan for the bridge in an English Horticultural Magazine as being a slight change from a rustic design that had been used in the past (Favretti & Alexopoulous, 1985). Lincoln also wanted to ensure there was a high enough arch in the bridge so citizens would be able to skate and boat underneath and experience the two meres conjunctively.

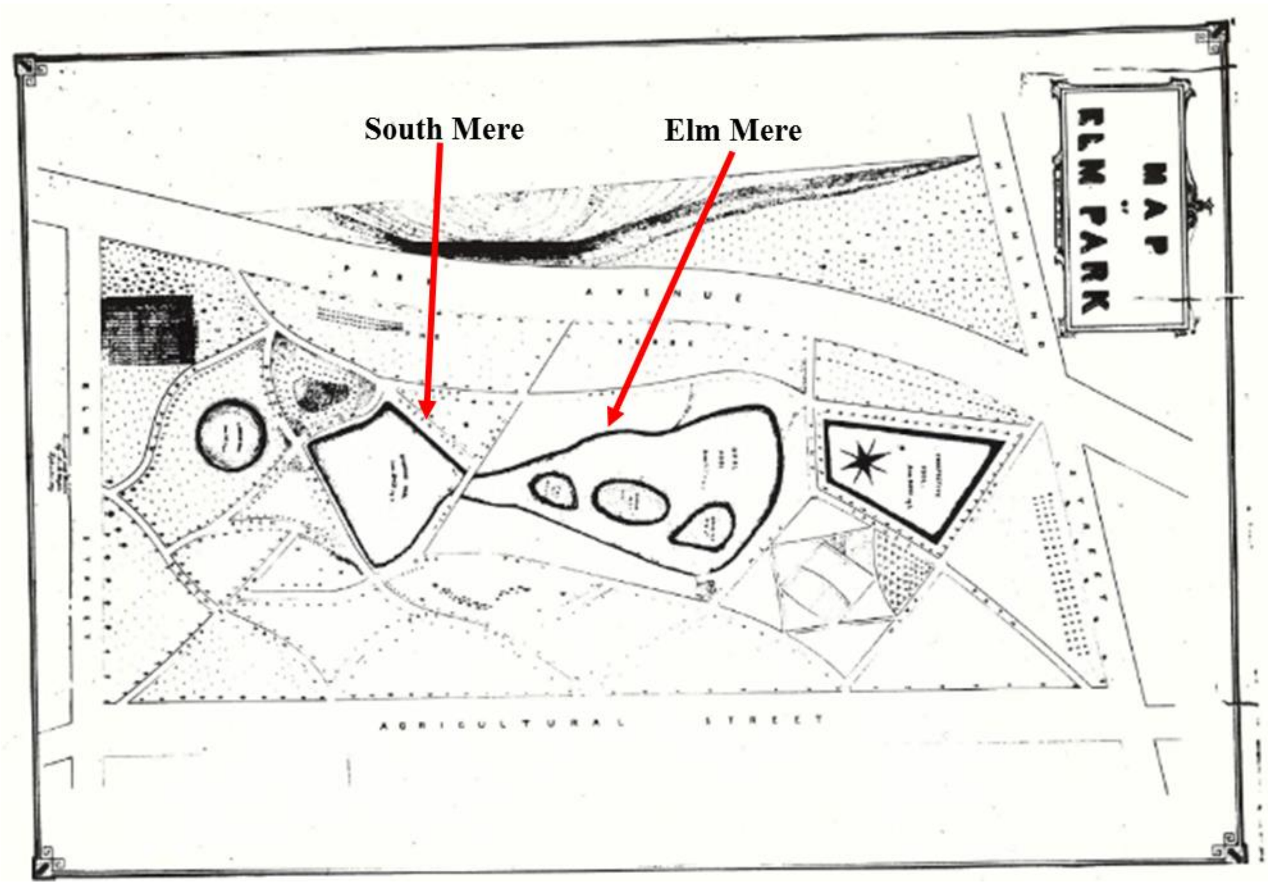


Figure 4: Map of Elm Park in 1879. (Favretti & Alexopoulous, 1985).

In addition to the bridge, Lincoln made further alterations to the park. He cleared and graded the landscape of the park and added walking paths, benches, shrubs and trees. Between 1878 and 1884, Elm Park was virtually completed and it began to take on its classic picturesque appearance. In the spring and summer many people would go to the park to enjoy the flowers and wildlife. Even the bitter cold winters did not keep people from crowding in and playing hockey, ice skating and just walking around the grounds (Yeulinski, 1994). In its early years, it was also regarded as a favorite place for carnivals, circuses, and other traveling menageries, much to the dismay of Lincoln, as he was opposed to the commotion that these events caused in what was supposed to be a relaxing atmosphere (City of Worcester Parks & Recreation Department, 2013).Unfortunately, the addition of all the attractive elements in the park also lured vandals. In only a years' time, the cedar bridge, along with other elements within the park had become defaced and needed replacement (Favretti & Alexopoulous, 1985).

By 1881, the cedar Elm Park Wooden Footbridge had been replaced again, likely due to weathering and vandalism. This time however, it was replaced with pine, a change from the original material (Favretti & Alexopoulous, 1985). It was during this same year that the original iron bridge was constructed as well, the other bridge located in the park. Lincoln wanted to promote a "light and airy" feeling of the water through the creation of these bridges and the approaches that lead up to them. He was against incorporating features such as huge piles of hammered stone in the bridges, as he felt it would take away from the ornamental ponds, which they spanned (Favretti & Alexopoulous, 1985). (See Figure 5)

Figure 5: Pine Bridge in 1885, Original Structural Design (American Antiquarian Society, Worcester).

In 1888, the city acquired an additional 60 acres of land (known today as Newton Hill), for the sum of $50,000. Lincoln was now in charge of over 88 acres of land that comprised the park, and it took a few more years to fashion the newer parts. The Park was officially completed in 1892, at which time attendance began to increase. As mentioned previously, with greater attendance also came increased vandalism. As a result, details of the park continually changed throughout the late 1800s and early 1900s (Favretti & Alexopoulous, 1985).

In 1896, Edward Winslow Lincoln, the visionary mind that made Elm Park a reality, passed away. After his death, the ideals of the park began to change as well, witnessing a shift from an emphasis of horticultural aspects to more recreational purposes. Lincoln's successors consistently made Elm Park a priority for maintenance and modifications. By 1900, gas-powered lights were added, additional plants and shrubbery had accumulated, the ornamental pools had been adjusted, and walkways and security were added to the park (Favretti & Alexopoulous, 1985).

It was also in 1905 that a change in the original design of the bridge could be observed. The supporting structure for the bridge was altered from its original form, with fewer supports, as noted in pictures that were taken during that year (See Figure 6). In 1906, both the bridges in the park were restored and painted a neutral color (Favretti & Alexopoulous, 1985). This color, a blackish/brown, differed from the original red color from 1877.

Figure 6: Different Structural Support System, c.1905 (AASW).

2.1.3 Period 3: 1910-1944

Common historical understanding has caused many to associate the brilliant Frederick Olmsted with the design of Elm Park. According to historical analysis, this is actually not the case. It is true that the Olmsted Brothers Firm, Landscape Architects of Brookline, Massachusetts, and legacy of Frederick Olmsted were part of two periods of active consulting on Elm Park. The first period was from 1910 to 1918. During this period, representatives from the firm dealt with changing the shape and size of South Mere and also consulted on landscape details and the wooden bridge design (Favretti & Alexopoulous, 1985).

In their initial report, they made comments on the horticultural aspects of the park, including the Elm Park Red Wooden Footbridge. The following is an excerpt from their initial findings in 1910:

> "The wooden bridge at the Southerly end of Elm Mere has become so old as to be unsafe and will soon have to be rebuilt. We question the advisability of retaining the present high arch of this bridge because it offers such an impediment to the walk crossing it, which is an important artery of travel. The only purpose of the high arch appears to be for the sake of the vista from the boathouse but this seems to be less important than the advantages of a low arch bridge. If the bridge is rebuilt a detailed plan of the conditions should be prepared" (Favretti & Alexopoulous, 1985).

The Olmsted Firm's renovation plan for the park and bridge was approved in 1911 and work began almost immediately. The entire park was overhauled at a cost of $6,920 (Favretti & Alexopoulous, 1985). After the construction of the meres was completed, the firm began considering an appropriate bridge to span the channel where the Elm Park Red Wooden Footbridge once was. Representatives from the Olmsted Firm exchanged notes with the park superintendent about the dimensions of the bridge and the size of the rowboats being used on the ponds. These specific details were necessary so the firm could design an appropriate stone bridge, "one that did not have to be replaced every few years like its predecessor cedar and pine bridges" (Favretti & Alexopoulous, 1985). A gentleman named R.F. Jackson of the Olmsted Firm designed a three-arched stone bridge in 1912 as a replacement for the wooden bridge; however it was never built due to its high projected cost (See Figure 7). As a cost saving measure, the city requested plans for a flat-floored concrete bridge from the firm as well; however, that bridge was never built either (Favretti & Alexopoulous, 1985).

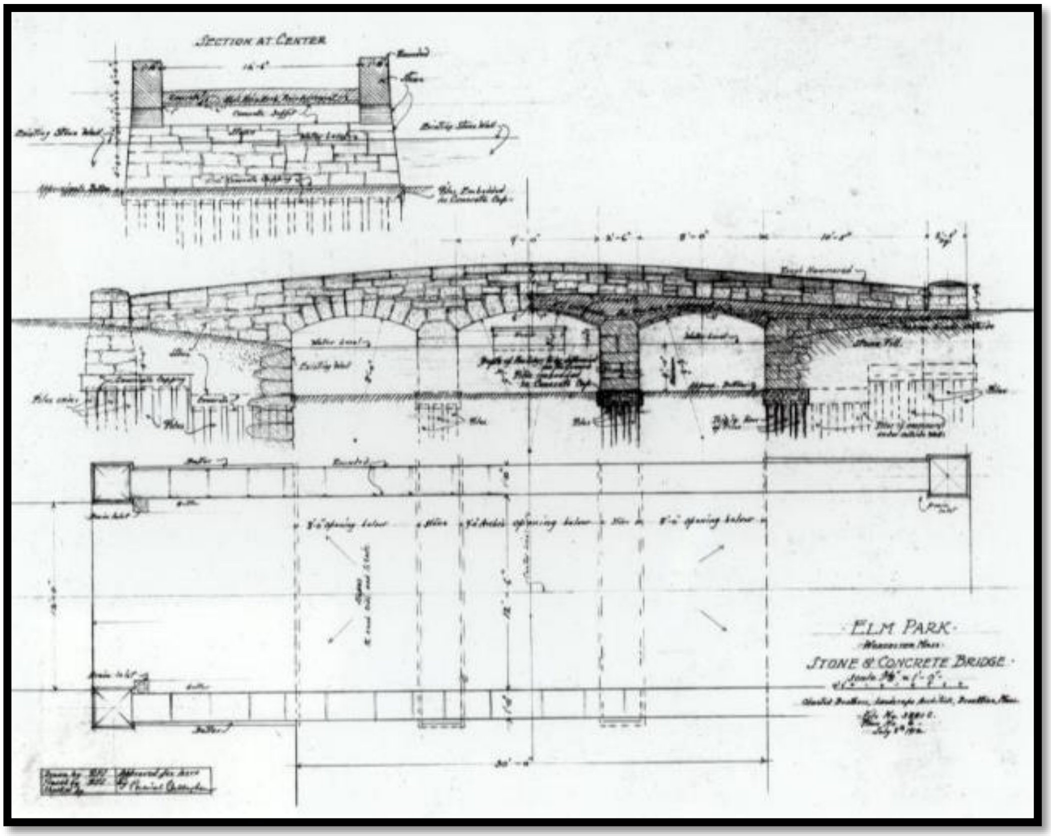


Figure 7: Alternative Stone Bridge Design Proposed By Olmsted Firm, c. 1911 (OlmstedArchives).

Few improvements were made or added to the park through the 1920s and 1930s. The meres were completely furbished with flood lighting in 1917, but other than that, the only routine maintenance that occurred was the rebuilding and repairing of bridges, benches, and walkways The lack of projects and activity in the park during this time period can be attributed to multiple reasons. Due to the Great Depression, funds that were available for parks were restricted, especially in major cities. Furthermore, Elm Park itself was essentially complete following the Olmsted Firm Renovation of 1912 (Favretti & Alexopoulous, 1985).

It was evident that despite the lack of work being completed in the park, a new bridge was built in 1924 at the cost of $305.03. It is not clear whether or not this was the Elm Park Red Wooden Footbridge, the Elm Park Iron Bridge, or maybe a different bridge entirely (Favretti & Alexopoulous, 1985). Despite the decline in work, recreational activities remained prevalent during this time period, especially skating in the winter and boat rides in the summer.

In addition to economic issues, Elm Park faced further problems throughout the early 1900s. Problems such as an ice storm in 1921 and a major hurricane in 1938 proved to be prolific impediments to the landscape of the park and the bridge as well. The park required another overhaul, and the Olmsted Landscape Architects firm was called on again to consult for the City. During this second period of consultation, they were brought in specifically to deal with grading, drainage circulation, planting around monuments, and altering footpaths. The Olmsted firm did not consult on the Elm Park Red Wooden Footbridge during this period. The city took most of the recommendations into consideration from the firm and by 1942 the majority of the endorsed work was completed. By the middle of the 1940s, “it had become one of the more beautiful places in the city” according to the Parks Commissioner at the time (Favretti & Alexopoulous, 1985).

2.1.4 Period 4: 1942-1985

Nearing the end of World War II, Elm Park began showing signs of neglect. Gasoline rationing during the war years led to increased use of the park on the weekends, which wore down the aesthetic landscape that had been renovated only a few years earlier by the Olmsted firm (Favretti & Alexopoulous, 1985). Slashed budgets and shortage of manpower also proved to be detrimental factors for the state of the park.

In 1945, a bridge in Elm Park was destroyed by a fire and rebuilt; however, it is not known as to which bridge it was that burned down. Additionally, in 1954 structural work on the Elm Park Red Wooden Footbridge was completed (Favretti & Alexopoulous, 1985). Pictures of a flat wooden bridge in 1970 in the same place as the Elm Park Red Wooden Footbridge today indicate that sometime between 1954 and 1970 the entire structure of the bridge changed, however exact dates are not known (See Figure 8).

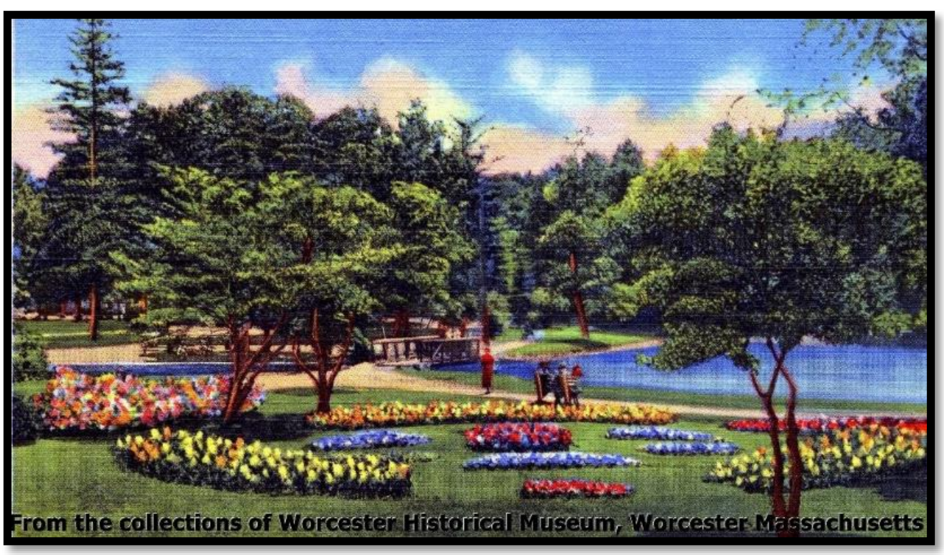


Figure 8: Postcard image of the Flat Bridge in Elm Park, 1945-1970, where the wooden bridge once stood (WHM).

By the late 1960's, attendance in Elm Park had decreased dramatically to slightly over 2,000 people per year. Elm Park also ceded 28 acres of land by 1965 for the construction of Doherty Memorial High School and street widening. Realizing the historical significance of the park and the current decline the park was experiencing, John Herron, Executive Director of the Worcester Historical Society, wrote a letter to Richard Hale, Director of the Massachusetts Historical Commission, lobbying for Elm Park to be designated as an historic landmark (Society, 1969).

After the recognition of Elm Park the city decided that the park would receive a major overhaul. The city and The Department of Parks and Recreation wanted to restore the park in a manner that would emulate its former historic glory. Arello Construction Inc. of Holden, MA was hired for the renovation of the park in what would turn out to be a nearly $320,000 project (Rayner, 1973). The project began in 1972, and encompassed landscape work, drainage, and historic replication of the two bridges in the park (Favretti & Alexopoulous, 1985).

The importance of recreating the bridges to be historically accurate was emphasized as a major focal point for the project. Harvey Rayner, a writer for the Worcester Gazette in the 1970's, gave his opinion in a 1973 newspaper article: "The arched bridge at South mere is handsome, much more so than the flat bridge it replaced (Fitzpatrick, 1973) (See Figure 9). However, public opinion was not unanimous. There were multiple articles published in 1973 criticizing the steepness of the new Elm Park Red Wooden Footbridge. Harvey and Tracy Associates, Inc. were the engineers that built the historical replicas of the Elm Park footbridges, and they were well aware beforehand of how steep the bridges would be. According to Francis S. Harvey, President of the design company:

> "The city wanted the new bridges to look exactly like the original bridges. There were no original drawings to work from, so I had to work from the old pictures. I worked mostly from these, designing the new bridges to look exactly as the original ones did, as close as humanly possible. There was no direction as to the steepness of the bridge. I did the best I could to make them exactly the steepness of the originals" (Fitzpatrick, 1973).

The original bridges were made steep in order to accommodate the skaters and boats that frequented the passage underneath them. Cartoon drawings poking fun at the steepness of the bridge were frequently observed in the Worcester newspaper in the following years (Favretti & Alexopoulous, 1985). In order to relieve some of the public dissatisfaction with the steepness, the Parks and Recreation Superintendent planned to put cleats on the bridges to improve the traction

(Rayner, 1973). For this project, the steepness of the ramps is not only a safety issue, but also an accessibility issue.

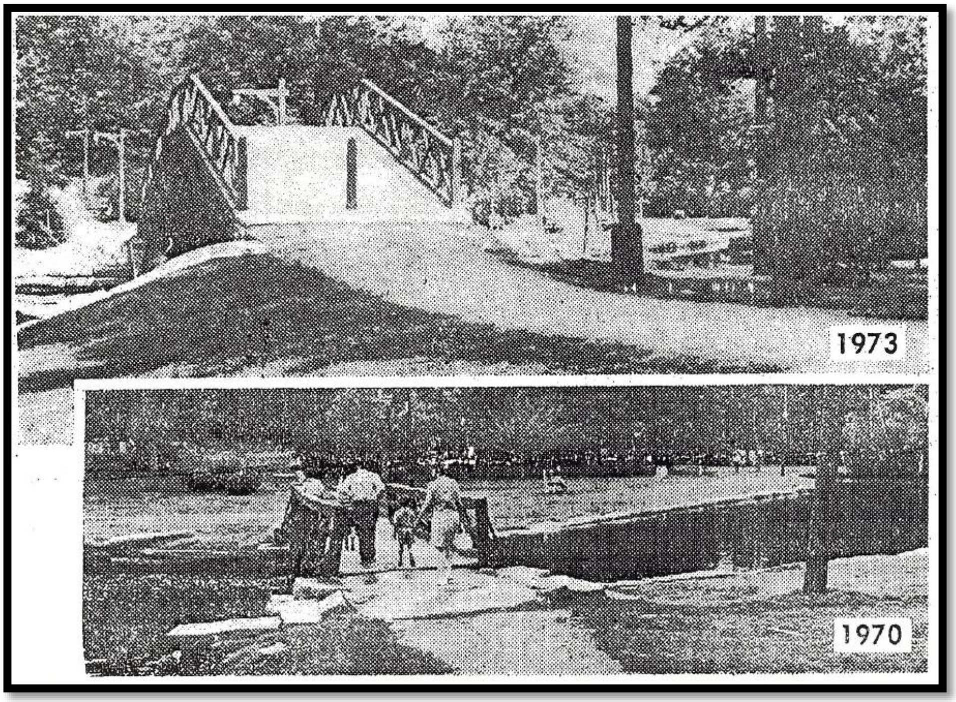


Figure 9: Photograph showing the differences in the Flat Bridge of 1970 and the Remake of the 1885 Bridge in 1973 (WHM).

Arello Construction failed to finish the project on time in 1973 as stipulated in the agreed upon contract, which they blamed on "unavoidable circumstances". Lawsuits were filed against The City on behalf of Arello in order to recover unpaid funds, which delayed the opening of the park until 1974. Upon completion, public sentiment was one of general satisfaction regarding the "New Elm Park" (Favretti & Alexopoulous, 1985).

A notable year for the Elm Park Wooden Footbridge was 1975, for two major reasons. First, photos suggest a change from the classic X-style railings to the straight board railings there are today. This more than likely was a measure taken to reduce the vandalism of the bridge, as the X-style planks were being kicked out of place frequently by vandals. Second, on May 25th, 1975 the bridge was dedicated to Harriet M. Horgan (Favretti & Alexopoulous, 1985) (See Figure 10). Multiple sources indicate it was in fact the Elm Park Red Wooden Footbridge that was dedicated to the late Ms. Horgan and not the Iron Bridge. Harriet Horgan was a member of The Daughters of Revolution and started the annual Memorial Day water ceremony at the Elm Park Red Wooden Footbridge. Ms. Horgan organized this water ceremony for many years during which "individuals stood atop of the bridge and recall loved ones who were lost at sea, missing in action, or buried overseas and then tossed flowers or wreaths into the water below" (Bridge Honors Prime Mover of Memorial, 1975).

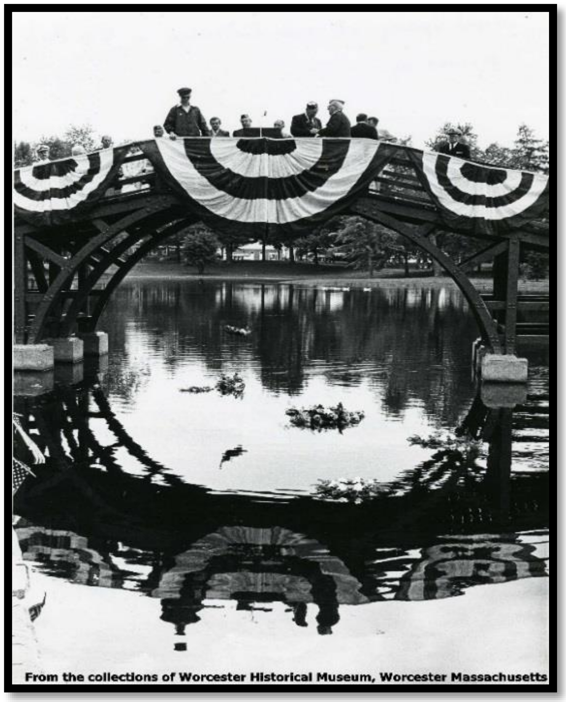


Figure 10: A photo of the bridge during the dedication ceremony in honor of Harriet Horgan in 1975 (WHM).

By the beginning of 1976, the park took on a nearly finished look. It once again became a highlight of the Worcester landscape and attendance increased over the next few years. In 1982, the park was placed on the National Register of Historic places, a designation made by the Department of the Interior, giving it "due credit as one of the earliest 19th century parks in the nation" (Favretti & Alexopoulous, 1985). Another important milestone for the park occurred in 1985, when it was selected as one of twelve parks in Massachusetts to receive a million dollar grant under the Olmsted Historic Landscape Preservation Program. This money went towards improvements of the park in the 1987 renovation. The Elm Park Red Wooden Footbridge only received routine maintenance during this park renovation, and was not rebuilt (Favretti & Alexopoulous, 1985).

The state of Elm Park has sharply declined since 1987. Lack of upkeep and routine maintenance has negatively influenced many aspects of the park. As late as early 2013, Elm Park was witnessed as a run-down landscape, with crumbled pond walls, broken lights, deteriorated walking paths, and unsafe bridges. The Elm Park Red Wooden Footbridge has not changed structurally since its 1970s renovation, and as a result is currently closed due to structural instability (See Figure 11). The Mayor and City Manager realized the condition of the park and fought hard for its refurbishment. On July 29th, 2013, the city of Worcester allocated $1 million for park renovations, and an additional $900,000 in funding was provided by the Commonwealth of Massachusetts to make repairs to the playground, install light poles, replace benches and tables, and refurbish pond walls (Worcester Breaks Ground on Elm Park Renovation, 2013).

Figure 11: Elm Park Red Wooden Bridge (2013).

Today, the Elm Park Red Wooden Footbridge is a vital asset to the makeup of the Elm Park landscape and needs to be renovated from its current condition. This bridge is the link between the East and West side of the park. It provides this link without impeding ice skaters or boater's route. This bridge has remained a "sculptural ornament" of the park for over a century and is one of the only structures that have stood in its exact location from the very beginning of Elm Park (Favretti & Alexopoulous, 1985). Its colorful history and sheer longevity is a testament to its importance in Elm Park both historically and moving forward into the future.

2.2 Stakeholders

As this project is being sponsored by the City of Worcester, there are many stakeholders that are involved. These stakeholders include various commissions and agencies that are formed to preserve the historical integrity of the structure, the other organizations involved in the design and construction of the bridge, and financiers. This project involved collaboration between the Worcester Polytechnic Institute students, the city of Worcester and the City Manager's office, the

Worcester Technical High School, the Worcester Park's Department and the professional engineer Steve Harvey. Helping to finance the project in memory of his late wife Myra Hiatt Kraft is Robert Kraft. And always a consideration throughout the project was the Worcester Community who will be the ones ultimately utilizing the new bridge.

2.2.1 City of Worcester

The Worcester City Manager's office were the ones to reach out to the school to get the group involved in this project. They brought this project to the attention of WPI in order to get the community involved in this project that will affect the entire Worcester Community. The two major contacts in the office were Eric Batista, assistant to the city manager, and Mr. Francis Steven Harvey, the professional engineer working on this project. Eric Batista was the contact for all information regarding the architectural design approval, while Steve Harvey was the main contact for checking calculations and analysis.

2.2.2 Worcester Parks and Recreations Department

The Worcester Parks and Recreation Department was an integral aspect of the development of this project. The main point of contact was Mr. Robert Antonelli, Assistant Commissioner of the Worcester Parks and Recreation Department. Another member of the parks department that worked primarily with the group was Bill Richards, a project manager for the construction currently taking place at Elm Park. Both Mr. Antonelli and Mr. Richards were interviewed by team members to help with the renovations of the park. A local engineering firm, Beal's & Thomas, was also contracted by the Worcester Parks and Recreations Department to assist with the site design portion of the project. David LaPointe and Regan Harrold of Beal's & Thomas were the main points of contacts with the firm and they were vital in establishing the final site design.

2.2.3 Worcester Technical High School

As part of this project, the WPI MQP team will be working in conjunction with the local Worcester Technical High School to construct the bridge. The major contacts at the Worcester Technical High School were Kyle Brenner, director of vocational/technical education, Joseph Lonergan, Carpentry Department Head, and Paul Chambers, Computer Aided Design and Drafting Department Head. The team met with the contacts at the Worcester Technical High School to ensure that the final design would be properly implemented when the construction was completed. Joseph Lonergan and the carpentry department students will be assembling the bridge and his input was highly considered in constructability because the students have not previously been exposed to as complex of structures as this.

2.2.4 Worcester Historical Commission

The Worcester Historical Commission (WHC) is a panel of six members and two alternatives who are responsible for determining the appropriate actions in regards to historical structures (City of Worcester, 2013). Therefore it was required that the commission be made aware of any changes that would be made to the park. If consultation with the WHC was neglected issues could arise in the future in regards to issues with the historical integrity of the design. By informing the WHC of any changes or alterations being made the bridge and asking for any feedback regarding the historical integrity of the bridge future issues were avoided.

2.2.5 Hiatt-Kraft Family

During the second presentation to the Worcester Historical Commission it was verified that the Elm Park Red Wooden Footbridge would be dedicated to the late Myra Hiatt Kraft, and be renamed the Myra Hiatt Kraft Footbridge. This dedication is in honor of the incredible work both her and her extended family have done throughout her entire life. Wife of Robert Kraft and daughter of Jacob and Frances Hiatt, Myra was a well-known philanthropist and adored by many people especially in the Worcester community (Brandeis University, 2012). She was born December 27, 1942 and raised in Worcester by her father Jacob and mother Frances Hiatt (New England Patriots News, 2011).

Both her and her father's accomplishments and input into The City set them apart from ordinary citizens. They contributed to both the Worcester Art Museum and Clark University because of their love of education and art. In 1979 the Hiatt's established two wings at the Dinand Library at Clark University in memory of Mr. Hiatt's parents and holocaust victims. In addition the Hiatt family also gave funds to student scholarships and educational programs at Clark University. The Hiatt family also contributed one million dollars towards the construction of the Frances L. Hiatt wing at the Worcester Art Museum (Kush, 2011). These contributions and Mr. Hiatt being a trustee of the Worcester community are what makes the Hiatt family such an asset to the Worcester community.

Mr. Hiatt, Myra's father, was a well-known philanthropist, trustee and friend of the Worcester community. Originally from Lithuania Mr. Hiatt came to the states in 1935. His love of education and desire to help people made him a strong supporter of schools, which he donated millions of dollars to in order to help students and children. He set up many merit-based scholarships for colleges to help seniors in Worcester who had outstanding credentials as they enrolled in college (College of the Holy Cross, 2001). Not only did he contribute to the community of Worcester but his legacy continued on through his daughter Myra Hiatt.

When Myra married Robert Kraft her contribution to Worcester and Boston grew even more. Together they contributed over $100 million dollars to charity; especially ones intended to improve the lives of children through education. Being the first female Chair of the Boys & Girls Clubs of Boston Myra helped support continuous and significant growth of tens of thousands of children in the city (Brandeis University, 2012). Her dedication did not stop there; together she and her husband did many more philanthropic acts to help better the lives of the people around them (English, 2013).

After Myra's passing in 2011 she left a legacy that will not be soon forgotten. She was considered by many to be a compassionate and hardworking philanthropist, and a loving wife, mother and grandmother (New England Patriots News, 2011). In the words of Worcester City Manager, Michael V. O'Brien, Myra Hiatt Kraft was "an incredible woman who built bridges of understanding, compassion and kindness throughout her lifetime" (O'Brien, M., 2013). Due to her hard work and dedication she has left many families and children with hope and the stepping-stones to a better future. Her husband Robert Kraft, Boston business leader and New England Patriots owner, is now helping to fund the restoration of the Elm Park Red Wooden Footbridge under Myra's name (New England Patriots News, 2011). This restoration of this bridge is being led by The City of Worcester and will be one of the main focuses of the project.

2.3 History of Building Codes

Building codes can be traced back as far back as 2000 B.C. with the Code of Hammurabi. Unlike the coded of today, codes such as this one lacked specific details on how a building should be built, and instead contained what the punishment would be if the structure were to fail. Since then, building codes have improved drastically, and in recent years have become much more detailed codes and stringent standards. (Jain & Leiva, 2013). The codes are updated periodically, according to weather and other hazards, by societies such as the American Society of Civil Engineers (ASCE) and other similar organizations (Jain & Leiva, 2013).

In 1994 the US had adopted its final building codes which are now used today. Over the past 75 years the United States adopted three different building codes: the Uniform Building Code (UBC), Standard Building Code (SBC) and the National Building Code (NBC) until finally in 1994 the International Code Council (ICC) was formed and developed a national standard (Jain & Leiva, 2013).

Although most states have adopted the International Building Code (IBC), it is not implemented uniformly across the country. In some locations enforcement of the international code is mandatory and no changes are permitted. Other locations may allow changes only if the accepted rule is more stringent than that of the national code. Ultimately local control takes precedent and respective municipals can choose whether or not to adopt the national code (Jain & Leiva, 2013).

The variation in code enforcement can be seen in areas of natural disaster such as Louisiana, and Florida after Hurricane Andrew. While enforcement and regulations have improved significantly since the disaster in 1992, code enforcement requires a more proactive approach, as in planning for the worst. Today most states do have statewide mandated codes, however a number are only based off the International Building Code and are therefore not as stringent (Jain & Leiva, 2013).

Because of the variation and complexity of building codes it is important to follow the most stringent code set forth to help ensure the structural stability of buildings and other structures. This is why the state of Massachusetts adopted the National Building Code, with a few amendments specific to the state (International Code Council, 2014).

2.4 Existing Bridge Condition

To be able to understand how to design a new bridge, the existing bridge conditions needed to be found such that they could be optimized in the new design. This assessment is an in-depth survey of individual bridge members conducted both on site and at the Worcester Technical High School, where the bridge was moved. A conditions assessment was performed to evaluate the existing conditions of the bridge as well as to identify where failure occurred in the structure, such as wood decay and insect attack.

2.4.1 Condition of the Bridge

Throughout history the Elm Park Red Wooden Footbridge has been renovated, and restored numerous times due to deterioration and vandalism. To understand and evaluate the cause of degradation due to loading, aging, and the environment, a visual conditions assessment was conducted. This assessment was done by a complete survey of the bridge and each of its members. The bridge was then analyzed further though a series of tests including a pick test, sound test and wood test. By performing these tests, weak sections of both the superstructures and substructures of the bridge were determined and analyzed. These areas of failure were found as follows:

On the Park Avenue side of the bridge the major cross-bracing beam between one support and the next was missing, one beam was only slightly connected on one end with the other end completely unattached. The fallen beams' vertical support columns were also rotting at the exposed connection point. On the Russell Street side of the bridge two beams were missing, exposing the internal connections – which are also rotted out – and one beam was separating from the vertical column it was connected to. In order to determine the absence of some members, the bridge was assumed to be symmetrical. Additionally, a few bolts at the connections were missing from both sides of the bridge. Several of these failures can be seen in Figure 12, Figure 13, Figure 14, below.

Figure 12: Example of previous bridge structure deficiencies - missing bolts.

Figure 13: Example of previous bridge structure deficiencies - missing beams.

Figure 14: Example of previous bridge structure deficiencies - rotting connections.

All of these damages reveal that the bridge's materials were deteriorating due to the age of the bridge as well as exposure to the elements. Additionally, much of the wood had been stripped of its paint due to weathering and human activity. The exterior layer of the wood had softened so much that when tested with a sharp object –in this case, a screwdriver—the object sank into the wood and formed an indentation with little effort. Splinters were also very easily formed by the wood, endangering anyone who walks barefooted or holds onto the handrails. Graffiti and trash left underneath the bridge also revealed that the bridges supports have been subject to abuse from park patrons. Hundreds of visitors come to the park each day, with a significant number of these visitors passing over the bridge when it is open. Though the bridge was deemed structurally safe for a single individual to pass over it, the heavy foot traffic from a larger number of pedestrians could cause localized structural failure and was deemed a safety hazard. The City of Worcester closed the bridge to traffic to mitigate any safety hazards to users and pedestrians.

2.4.2 Current Violations of ADA and AAB

The current bridge was surveyed and a condition assessment was completed in order to gain an understanding of the current violations of the Elm Park Red Wooden Footbridge before beginning to develop alternative designs. There were many issues with the current design of the bridge. While the existing bridge appeared to be structurally sound, the bridge was not safe in terms of its accessibility. As the bridge was built in the 1970s certain guidelines did not exist yet, such as ADA, which are now required in the new design. Therefore during the renovations many issues regarding safety and today's design standards must be considered.

Today, the most significant set of safety regulations is the ADA and the regulations set forth by the AAB. One of the most obvious violations of the current design was the slope of the bridge. The slope was unsafe for many able-bodied individuals to ascend, and even worse under certain circumstances caused by weather, such as rain, snow, and freezing. The slope must be reduced to no more than 8.3% if the bridge is to comply with ADA and AAB regulations (U. S. Department of Justice, 2010). Additionally, the slope of the path leading to the bridge was also too steep to comply with the safety regulations. While it was not as dangerous as the bridge itself, it still was too steep for a wheelchair to climb. Overall, the slopes of the bridge were the clearest violations of today's safety standards.

Figure 15: Image of Elm Park Red Wooden Bridge (September 2013).

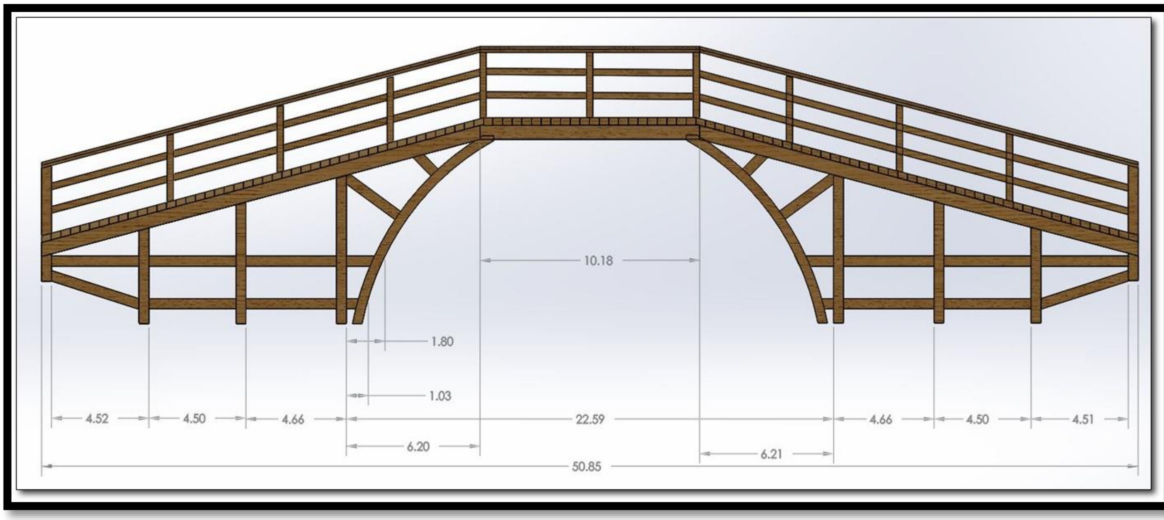


Figure 16: CAD image of Elm Park Red Wooden Bridge.

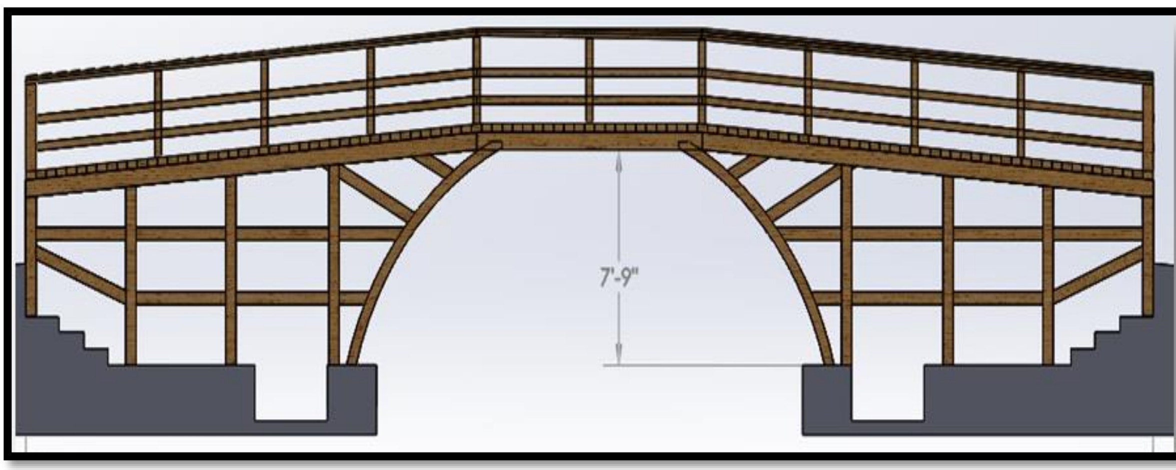


Figure 17: CAD image of first ADA-compliant iteration to show how the slopes will be changed.

In addition to how steep the bridge was, there were further accessibility issues with slopes of the bridge. The current design offered little transition from the beginning of the sloped path to the peak of the bridge, as the path is a continuous slope. This slope must be modified to provide some level sections between slopes in order to comply with regulations (U. S. Department of Justice, 2010). The peak of the bridge may be considered as one of these sections, but level sections must also be integrated into the site grading. In addition to reducing the running slope, the cross slope due to slopes perpendicular to the walking path must also be significantly reduced to no more than 2%, particularly if the access routes are not graded evenly (Massachusetts Office of Public

Safety, 2006). If the site is graded properly and the bridge's height is reduced, most of the violations of today's ADA safety regulations would be rectified.

While the steep slopes provide the major issues with respect to ADA compliance, there are also finer details of the bridge design that were not compliant with ADA and AAB. According to ADA, there must also be continuous handrails along the sloped sections (U. S. Department of Justice, 2010). The current bridge had handrails along its edges, but the sloped pathway did not. The height of the current handrails are 34 to 38 inches and are within the tolerance levels of ADA (U. S. Department of Justice, 2010), however they do not feature a second tier of continuous functional handrail required at a height of 18 to 20 inches as specified by AAB (Massachusetts Office of Public Safety, 2006). All sloped surfaces must have handrails along both edges in order to be compliant with safety regulations. Another issue was the concrete platform on both ends of the bridge. This was an exposed part of the foundation where the earthen cover has been eroded away by years of weather and human activity, and now presents an extra step that must be climbed in order to use the bridge. This step was more exposed on the Park Avenue end of the bridge than on the Russell Street end. As there was no alternative path around these steps, they must be covered with well-packed soil again so that they are integrated back into the sloped path and not exposed (Massachusetts Office of Public Safety, 2006). These issues with the handrails and path continuity must also be addressed if the renovated bridge is to be compliant with ADA and AAB.

In conclusion, in order to comply with today's standards on accessibility, the bridge must undergo many design changes. The compliant designs will focus on the slope of the bridge and pathways, as well as accessibility features such as handrails, in order to be accessible to all members of the community.

2.5 Bridge Design Considerations

In order to design and construct a bridge that would hold up over time many factors needed to be considered. The two major ones are the loading conditions and the connections. The loading conditions involve additional factors as member size, member properties, and allowable strengths. These loading conditions are based off of building codes and design standards. The connections are also an integral part of the bridge that must retain adequate strength and durability over time under harsh weather and environment.

2.5.1 Loading Conditions

Today there are two accepted methods of performing load analysis. The oldest method is referred to as Allowable Stress Design (ASD). The Allowable Stress Design method calculates the stress induced in the structure which should not exceed the allowable stress value of the model and compares it to the allowed stress (Yang, 2013). The other loading method is the Load and Resistance Factor Design (LRFD), which is used to calculate the combined factored loads and compare it to the factored capacity or resistance of the structure (Yang, 2013).

According to ASCE-7 both loading combinations must take dead loads, live loads, and environmental loads. When calculating the allowed stress or maximum load allowed (ASD and LRFD) many factors need to be looked into such as; the wet service factor, temperature factor, beam stability factor, volume factor, flat use factor, curvature factor, column stability factor and bearing area factor must be considered. The allowable stress design (ASD) must be calculated including the load duration as well, while when considering the maximum allowed load (LRFD), format conversion factor, resistance factor, and a time effect factor should be considered (American Society of Civil Engineers, 2012).

When performing the load calculations the load amounts can be found in the governing code like the IBC, whereas the loading factors can be determined using ASCE-7.

2.5.2 Connections

One way to ensure structures meet structural stability requirements is through the connections used. The issues experienced in the past while trying to join two wooden members has led to the development of multiple connection methods including bolts, split rings, nails, and plates. One of the most widely used connection methods are the metal plates because of their low cost and simple installation (Gupta, Vatovec, & Miller, 2011). Since the development of metal plates in 1952 hundreds of millions of wooden trusses have been successfully created and used in structures. Although metal plates are popular it is important to consider moisture cycles and creep when choosing a connection type. Metal plates are susceptible to moisture and creep due to the entrapment of water and the rotting of wood (Gupta, Vatovec, & Miller, 2011). Although plate connections are popular there are other options including lap joints and butt joints, both shown in Figure 18. While there are a number of possible connections it is important to take the location into consideration, because of rot and other deterioration issues.

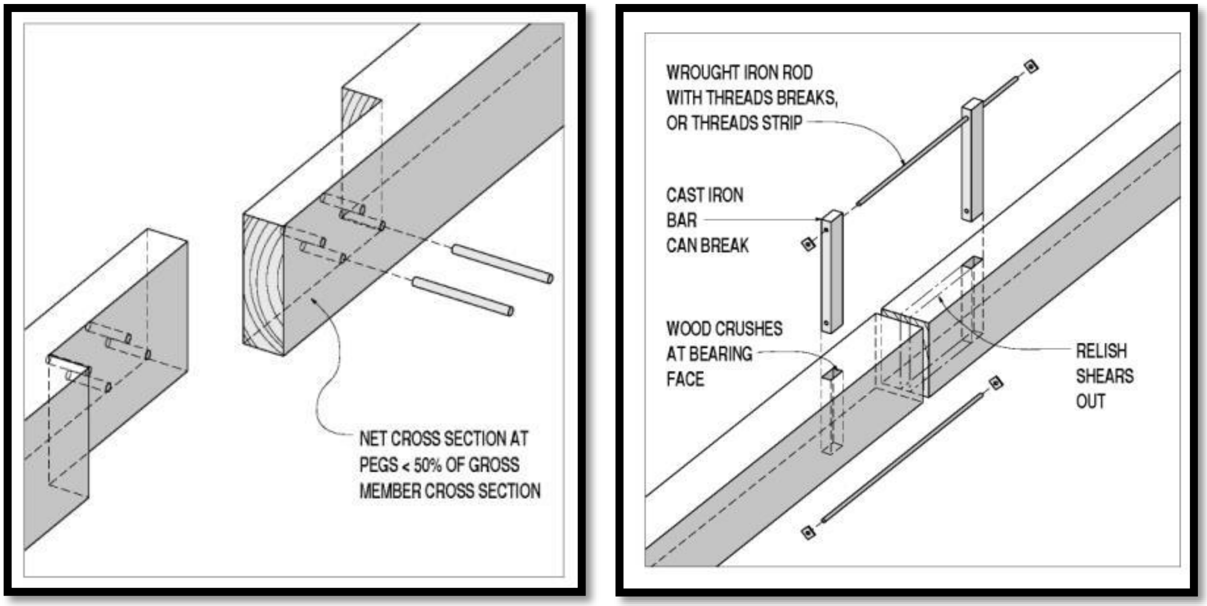


Figure 18a and b: Examples of connections: lap joint (left) and butt joint (right) (Federal Highway Administration, 2005).

2.6 Site Design Considerations

Over time, many features of Elm Park have become outdated or hazardous. As Rob Antonelli, the Assistant Commissioner of the Worcester Department of Parks and Recreations stated in an interview; "the bridge was cordoned off from the public on May 2013 due to its uncertain structural integrity" (Antonelli, 2013). A visual examination showed much deterioration of the bridge, which led to the ultimate decision to close the bridge off to the public. In addition to the concerns with the bridge itself, concerns with the rest of the park have led the City of Worcester to begin an entire renovation of the park to help make it safer and more appealing for patrons. Two stages of the renovations are currently planned, the first stage of which was completed September 2013, addressed the side of the park closest to Highland Street and included renovations to the playground, walkways, and lighting. The second stage of construction has recently commenced and focuses on the bridge renovations and the adjacent areas. Because of these expected renovations, the site conditions would be changing slightly, but in most cases the recommended bridge design would not be affected.

2.6.1 Soil Exploration

An understanding of the existing soil conditions in Elm Park was an important step in the development of this project. A sound knowledge of the soil conditions and appropriate engineering parameters was important when assessing the existing foundations for bearing capacity and settlement failure. Boring logs obtained from the vicinity of the bridge provided valuable information about the subsurface conditions. Boring logs can also provide soil types and ground resistance blow counts that allow for prediction of soil engineering parameters used in foundation design. The logs also gave a snapshot of what the soil profiles consisted of. The information obtained from these soil boring logs allowed for conservative assumptions to be made for the purposes of calculations for the foundations allowable bearing capacity.

2.6.2 Concrete Footings Assessment

An important aspect of this project that could not be overlooked was the footings upon which the bridge stand. Initially, there had been question as to whether or not new footings would be poured to accompany the new bridge, or if The City would reuse the old footings that were poured in 1972. After consultation with The City, and the Parks and Recreation Department, it was decided that the existing footings would be used for the new bridge. Not only would this be much more cost effective, but it would also be more feasible for planning purposes, especially considering the construction timeline with Worcester Technical High School and The City. If the existing footings are adequate, only minor concrete pours will be required to adjust for elevation changes with the new bridge design.

The existing footings have been in place for over 40 years and have resisted the existing loads and environmental conditions. It is estimated that the footings will last under 50 years or so if subject to similar environmental conditions. However, it was important that an initial assessment of the two foundations was completed to reaffirm the structural integrity of the footings on both the East and West side of Elm Park. Additionally, it was imperative that measurements of the foundation be taken so that an accurate representation of both foundations could be created in the Autodesk structural design programs used to create the bridge. The bridge will ultimately be placed on these footings, necessitating that its design reflect the geometrical makeup of both footings.

Prior to the walkthrough, it was vital to understand some key irregularities regarding concrete foundations and other things that should be looked for during a visual inspection. Determining whether or not foundation failure has occurred was the first step in this process. This happens when the foundation no longer performs its intended function of providing a stable support for any applied loads. If any changes or irregularities can be noticed in the foundation, they should be negligible and not change the integrity of the structure (FPA, 2013). Any form of tilt in the structure was also assessed. That is, the uniform slope from one end of the structure to another. Tilting can occur as the result of soil settlement and heaving, or if the original foundations were built in an unleveled condition. Tilting can cause induced stresses that are detrimental to the foundation if too large. Additionally, any visual pertinent damage in the concrete structures in the form of vertical and horizontal cracks, erosion, mold, unwanted deflections, and any other chips or forms of impairment were noted (FPA, 2013).

2.6.2a Shallow Foundations

It was important that a complete structural analysis be performed on the existing foundations in Elm Park to ensure the structural integrity of the foundations and be absolutely certain that they would have no problems supporting the loads imparted by the new bridge in the future. In order to accomplish this, an understanding about shallow foundations, the purpose they serve, and also what types of failures can occur with those shallow foundations, such as bearing capacity failure and settlement failure must be understood.

Shallow foundations transmit structural loads to the near surface soils they lay on. Most often, they are in the form of spread footing foundations, also known more simply as a “footing”. Spread footings are an enlargement at the bottom of a column or bearing wall that spreads the applied structural loads over a sufficiently large soil area (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). Usually, each column has its own spread footings so that a structure may have dozens of individual or combined spread footings to form one massive foundation. Spread footings are by far the most common type of foundation, primarily due to their relatively low cost and ease of construction. They are typically used on small to medium sized projects with moderate to good soil conditions, such as the Myra Hiatt Kraft Footbridge project (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

Spread footings are available in variable shapes and sizes to accommodate specific project needs. The ones utilized for this project are combined rectangular spread footings. These have plan dimensions *B x L*, where L is the longest dimension. These are very useful when on site obstructions prevent construction of a square footing with a sufficiently large base area, as well as when the design requirements for the structure it supports call for irregular shapes.

For the purposes of this project and in order to be as conservative as possible, the foundations on the East and West side of Elm Park were considered as three separate rectangular strips with perpendicular interconnections. As can be seen in Figure 19 below, the foundations as they sit now have symmetrical voids in their cores, which would reduce their overall weight and result in a decreased bearing pressure.

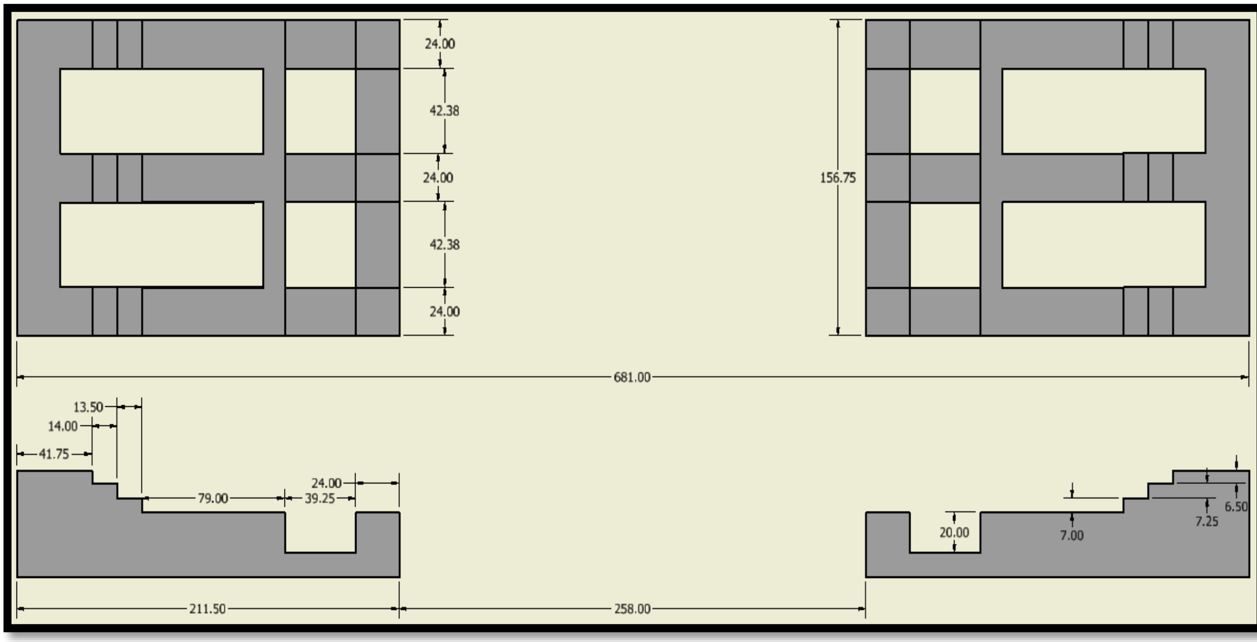


Figure 19: Existing foundation schematic

However, given their proximity to the pond and the soil they overlay, these foundations should be considered as separate strip foundations in order to obtain an estimate of the bearing pressure imparted on the underlying soils. This is a conservative approach that would ensure the structural integrity of the foundations for years to come. Furthermore, as there were no plans provided for the existing footings from 1972, it cannot be known for certain the type of concrete that was used for the project, if reinforcement was used at all, or what the depth of the footings actually are without any further extensive testing. For these reasons and given the existing conditions of the project, it would be assumed from this point forward that the concrete is normal weight, there is tensile reinforcement, and the depth of the footings is 6 ft. below the known elevation of 492 feet on the top of the footing that supports the inner most support of the bridge.

2.6.2b Settlement Analysis

Despite the fact that shallow foundations may be designed with an adequate factor of safety against bearing capacity failure, it does not mean that the foundation would not settle excessively into the earth. In reality, bearing capacity and settlement come hand-in-hand. In most cases, settlement actually controls the design of foundations, especially with larger widths (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). In evaluating the existing foundations at Elm Park, it is essential to consider any settlement that may have occurred in the foundations in the last forty years.

Settlement itself refers to how much a foundation settles or sinks in a soil after it is laid and construction is complete. Although settlement is caused mainly by the application of loads on the foundation itself, other sources of settlement are also important to consider. These include settlement caused by the weight of recently placed fill, by the rising and falling of the ground water table, and by secondary compression of underlying soils (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

Consolidation is usually the most important source of settlement and causes an immediate increase in the vertical total stress in the underlying soils of a foundation. Additionally, if the soils are saturated, an equal amount of excess pore water pressure also forms. In other words, immediately after the fill and foundation are placed in a situation, their weight is carried entirely by the pore water. The presence of these excess pore water pressures produces a hydraulic gradient which forces some of the pore water to flow out of the soil. As the water flows out, the soil consolidates, and the vertical effective stress in the soil increases. Ultimately, the consolidation settlement is the result of this increase in vertical effective stress (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

Typically, nearly all settlement analyses are based on the results of laboratory or in-situ tests. The laboratory methods are based on the results of consolidation tests and thus are applicable to soils that can be sampled and tested without excessive disturbance. Laboratory tests were not performed on the soil samples taken from the Elm Park Construction site, and only in-situ standard penetration tests were performed. From the borings taken on both sides of the East and West foundation, it was clear that at least the soil adjacent to the foundations is underlain by wet clay and peat material. Since no borings were taken in the pond directly beneath the foundations, it cannot be known for sure what soil conditions exist beneath the footings.

Peat is the accumulation of partially decayed vegetation or organic matter. Peat is extremely soft and easily compressible, even under very small loads like those that would be applied from the bridge structure (Joosten & Clarke, 2002). However, the closest boring logs available imply that the footings are predominantly underlain by peat material. This does not make sense, as any structures that support loads similar to those subjected by the Elm Park Red Wooden Footbridge would not be allowed to sit on peat material. For this reason, and the fact that no plans or documents are available from when the footings were poured, it is safe to assume that the clay and peat material under the footings were excavated before the footings were poured in 1972 and replaced with more stable sand/gravel that do not settle excessively. It is recommended however, that additional borings and ground penetrating radar or sonar methods be used to identify the exact conditions underneath the footings. These methods would not only determine the depth of the foundation but also clarify the soil strata and perhaps even identify if piles were used instead to support the two massive footings on each side of the pond.

After establishing the assumption of sandy gravel under fill beneath the footings, some additional assumptions could be made. The permeability of sand is very high. Coarse-grained soils do not undergo consolidation settlement due to relatively high hydraulic conductivity compared to clayey soils. Therefore, drainage in sandy/gravels occurs almost instantaneously and any settlement that occurs is immediate upon construction (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). This means that any settlement that did occur in the footings occurred when the footings were poured, more than 40 years ago, and has since ceased. Furthermore, visual inspection would suggest that the footings themselves have not moved noticeably since their inception in 1972, and therefore it is appropriate to assume that any settlement that has occurred is no longer occurring and the addition of the new bridge loads would play a negligible role in causing any additional settlement at the site.

Ultimately, there is no reason to calculate the settlement values for the two footings as it would not make sense given the context of the scenario and the age of the footings. However, if an analysis were completed for the footings upon being poured in 1972, it would make sense to utilize Schmertmann's Method. It was developed primarily as a means of computing settlement of spread footings on sandy soils. It is most often used with CPT results, but can be adapted to other in-situ tests such as the SPT (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

The method utilizes an equivalent modulus of elasticity, which is a linear function. The design value implicitly reflects the lateral strains in the soil. Several correlations have been developed between E_s and N_{60}, and they can be related through empirical equations. Additionally a strain influence factor, which accounts for the distribution of vertical strain at different depths below a footing, is calculated at the mid-point of each soil layer being considered in the analysis. Correction factors are then computed so that an ultimate settlement can be obtained. This process is outlined in Chapter 7 of Foundation Design: 2nd Edition and can be rapidly expedited utilizing excel spreadsheets provided by the makers of the aforementioned book. An estimate of the initial settlement that occurred in the footing in 1972 was derived using the spreadsheet, which can be seen in Appendix A: Settlement Excel Spreadsheet.

2.6.2c Bearing Capacity

Bearing pressure is the most fundamental engineering parameter that defines the interface between a shallow foundation and the soil it overlays. It is generally defined as the contact force per unit area along the bottom of the footing. The pressure exerted by the footing on the soil is not necessarily distributed evenly, and can depend on a variety of factors including eccentricity of the applied loads, magnitude of the applied moment, structural rigidity of the foundation, engineering properties of the soil, and roughness along the bottom of the footing. For the purposes of this project, it is customary to assume that the pressure beneath concentric vertical loads is uniform across the base of the footing.

Bearing pressure along the bottom of a footing is generally defined as:

$$q = \frac{P + W_f}{A} - u_d$$

where q is the bearing pressure, P is the vertical load imparted by the column on the foundation, W_f is the weight of the footing, including the weight of the soil above the footing, A is the base area of the footing, and u_d is the pore water pressure at the bottom of the footing. For normal weight reinforced concrete, the accepted unit weight is 150 lb/ft^3, and it is the value used for this project.

One of the engineering parameters a foundation must satisfy is the bearing capacity requirement, which is more or less a geotechnical strength requirement (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). When shallow foundations apply the structural loads to the near surface soil, they induce both compressive and shear stresses in the soil. The bearing pressure and size of the footings would dictate the magnitude of these stresses. In some cases, if the bearing pressure is too large or footing too small, the shear stresses may exceed the shear strength of the soil or rock, causing a bearing capacity failure. Three types of bearing capacity failures typically occur, general shear failure, local shear failure, and punching shear failure. For nearly all-practical shallow foundation problems, it is only necessary to check the general shear case. Settlement analysis typically protects against local and punching shear failures (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

There are many methods for analyzing the bearing capacity of spread footings. Various researchers have studied the relationship between bearing capacity, loading, footing dimensions, and soil properties and how they affect the bearing capacity in different projects. Two of the more common methods are Terzaghi's method and Vesic's method.

Terzaghi's method includes some inherent assumptions, which were applied for the purposes of completing the analysis. The depth of the foundation is less than or equal to its width ($D \leq B$), no sliding occurs between the foundation and the soil, the soil properties below the foundation are homogenous for a great distance below the foundation, and the general shear mode of failure governs, among others (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). Since Terzaghi neglects the shear strength of soils between the ground surface and the depth of the foundation, the shear surface stops at this depth and the overlying soil is replaced with an additional surcharge pressure σ_{zD}', which is a very conservative approach. Ultimate bearing capacity before failure then becomes:

$$q_{ult} = 1.3c'N_C + \sigma_{zD}'N_q + 0.5\gamma' BN_\gamma$$

where c' is the effective cohesion of the soil beneath the footing, γ' is the effective unit weight of the soil and depends on the groundwater location, B is the width of the footing, and N_c, N_q, and $N\gamma$ are Terzaghi's bearing capacity factors (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

Vesic's Method is an alternative to Terzaghi's Method that leads to a more accurate approach to finding bearing values (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).Vesic's method keeps the same format of the previous method except it includes additional factors. These factors produce a new ultimate bearing capacity equation, which is:

$$q_{ult} = \boldsymbol{c'N_C}s_c d_c i_c b_c g_c + \boldsymbol{\sigma_{zD}'N_q}s_q d_q i_q b_q g_q + \mathbf{0.5}\boldsymbol{\gamma' BN_\gamma}s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$$

where the bolder factors are those that are common between the two methods. Each term with *s* is a footing shape factor, terms with *d* are depth factors, those with *i* are load inclination factors, the ones with *b* are base inclination factors, and finally those with *g* are ground inclination factors. All of these factors depend on varying site condition elements; therefore unlike bearing capacity factors, there are no pre-tabulated values. Instead, there are equations to follow to obtain values for each factor (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

With Vesic's Method being more accurate, this method was utilized for bearing calculations. In order to conduct a proper bearing capacity analysis for the footings in Elm Park, the boring log reports provided by Soil Exploration Corporation of Leominster, MA, were examined, which can be found in Appendix B: Soil Boring Logs. From these reports, the proper engineering parameters of the soil underneath the footings could be derived along with educated assumptions. It is very important to not overestimate the soil strength parameters, as it can result in skewed bearing capacities. Thus, fairly conservative values were chosen. Furthermore, the boring logs also highlighted the difference in soil layers at the site. The bearing capacity analysis of soil structures that is not uniform for a great depth was also considered. This means that the engineering parameters for the different soil strata's would differ. Since Vesic's method assumes a uniform soil, it was important to determine what approach would be utilized to account for this. This could be taking the lowest values of particular soil strata, or taking the average values of the parameters. In addition, it was important to determine the ground water location at the site, particularly at the foundations.

The presence of ground water has a noticeable effect on the shear strength of the soil in two ways: the reduction of apparent cohesion and the increase in pore water pressure (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). Both of these affect bearing capacity. When conducting bearing capacity analysis of a foundation, one must determine the worst case or highest expected location of the ground water table. Since the foundations being used for the bridge have such a close proximity to the pond, the ground water elevation is inevitably high, and will assume a case where $D_w \leq D$. As a result, the effective unit weight parameter, γ', in the equation for ultimate bearing capacity above now becomes:

$$\gamma' = \gamma_b = \gamma - y_w$$

Once assumptions were finalized the equations could be used to proceed and solve the bearing capacity formula with the adjusted effective unit weight. This bearing capacity was then compared to the allowable bearing capacity, which is simply the ultimate bearing capacity divided by a factor of safety. Most often, design factors of safety are not specified, especially in a unique project such as this one. For this reason, engineers must use their own discretion when selecting a factor of safety, and consider factors such as soil type, site characterization data, soil variability, importance of structure and consequences of failure, and likelihood of design loads actually occurring (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). Figure 6.11 in Chapter 6 of Foundation Design by Daniel Coduto helps delineate this process.

Bearing capacity spreadsheets, like those in Microsoft Excel, can greatly reduce the tedious calculations necessary to perform a bearing capacity analysis, and so would be utilized in this project. Foundation Design by Daniel Coduto developed a Microsoft Excel Spreadsheet that calculates bearing capacity of foundations and can be downloaded from the Prentice Hall website. These spreadsheets were utilized, which can be found in Appendix C: Bearing Capacity Excel Spreadsheet, to complete the bearing capacity calculations for the foundations in Elm Park.

2.6.2d Visual Inspection

Prior to the walkthrough, it was important that the WPI project group understand some key irregularities regarding concrete footings and things that should be looked for during a visual inspection. First and foremost, determine whether or not footing failure has occurred. This happens when the footing no longer performs its intended function of providing a stable support for any applied loads. If any changes or irregularities can be noticed in the footing, they should be negligible and not change the integrity of the structure (FPA, 2013). We also needed to asses any forms of tilt in this structure. That is, the uniform slopes from one end of the structure to another. Tilting can occur as the result of soil settlement and heaving, or if the original footings were built in an unleveled condition. Tilting can cause induced stresses that are detrimental to the footing if too large. Additionally, the team would need to note any visual pertinent damage in the concrete structures in the form of vertical and horizontal cracks, erosion, mold, unwanted deflections, and any other chips or forms of impairment (FPA, 2013).

2.6.3 Abutment Retaining Wall Consideration

The site design process also included consideration of retaining walls and concrete foundation abutments to accompany the new bridge design. The following section provides background information on retaining wall design which was utilized in the site design process.

2.6.3a Retaining Walls

An additional aspect taken into consideration when designing the Myra Hiatt Kraft Footbridge was the implementation of a retaining wall. Retaining walls are structures used to hold back and withstand masses of earth or other loose material where existing conditions make it impossible to let those masses assume their natural slopes (Nilson, Darwin, & Dolan, 2010). Usually these kinds of conditions occur when there is some sort of restriction such as ownership or use of structure that limits the width of an excavation, cut, or embankment. There are many different types of retaining walls, of which include gravity walls, reinforced concrete cantilever walls, and reinforced counterfort walls. Gravity walls retain the earth using their own weight alone and generally contain no reinforcement. The cantilever wall consists of a vertical arm that retains earth and is held in position by a footing or base slab. In this case, the weight of the structure and the weight of the fill on top of the slab contribute to the stability of the structure. The counterfort wall is typically employed for walls that are very high, as they help reduce the bending moment in these walls. Finding out which of these is most appropriate for a given situation depends on the given case and the variety of conditions present on a given site. Typically, gravity walls are employed for heights less than 10 feet, cantilever walls for heights ranging 10 to 20 feet, and counterfort walls for heights greater than 20 feet (Nilson, Darwin, & Dolan, 2010).

2.6.3b Earth Pressure

One of the first things to consider when designing a retaining wall is the earth pressure. Soils and other granular masses occupy a position intermediate between liquids and solids. When a soil is poured from a truck, it flows, but will not form a horizontal surface. It maintains itself in a pile where its sides will form an angle of repose, the tangent of which is roughly equal to the coefficient of intergranular friction (Nilson & Darwin et al., 2010). If a vertical wall retains soil, the earth pressure will increase proportionally to the depth, with its magnitude equal to:

$$P_h = K_0 w h$$

where *w* is the unit weight of the soil, and K_0 is a constant known as the coefficient of earth pressure at rest (Nilson & Darwin et al., 2010). The value of K_0 is dependent not only on the properties of the backfill, but also the method of filling and compacting it. It has been tested and is generally accepted that for un-compacted non-cohesive soils such as sand and gravels, K_0 is between 0.4 and 0.5 while it may be as high as 0.8 for the same soils in a highly compacted state. For cohesive soils, K_0 may range from 0.7 to 1.0. Most often, clean sands and gravels are considered superior to all other soils because they are free draining and not susceptible to frost action. For this reason, non-cohesive backfills are usually preferred.

Walls move slightly under earth pressure. Due to their elastic material, they deflect under action of the pressure. If the wall moves away from the fill, a sliding plane forms in the soil mass, and the wedge sliding along that plane exerts pressure against the wall. The angle there is known as the angle of internal friction (i.e. its tangent is equal to the coefficient of intergranular friction) and is usually determined by lab tests, and defined as 45 $+\varphi/2$. The corresponding pressure is known as active earth pressure. If the wall is pushed against the fill, an ulterior sliding plane is formed and the wedge is pushed upward by the wall along that plane. The pressure that the larger wedge exerts against the wall is known as passive earth pressure, and is defined as 45 - $\varphi/2$. Many have analyzed the magnitude of active and passive pressures by soil on retaining walls. According to Rankine, for soil surfaces that make an angle δ above the horizontal surface of soil, then the coefficient of active earth pressure is equal to:

$$K_a = \cos(\delta)\ \frac{\cos(\delta) - \sqrt{\cos(\delta)^2 - \cos(\phi)^2}}{\cos(\delta) + \sqrt{\cos(\delta)^2 - \cos(\phi)^2}}$$

and the coefficient for passive pressure is:

$$K_p = \cos(\delta)\ \frac{\cos(\delta) + \sqrt{\cos(\delta)^2 - \cos(\phi)^2}}{\cos(\delta) - \sqrt{\cos(\delta)^2 - \cos(\phi)^2}}$$

In this case, K_a and K_p replace K_0 to determine soil pressure. For the frequent case of the horizontal surface, active pressure is equivalent to:

$$\mathrm{K_{ah}} = \frac{1 - sin\phi}{1 + \sin\phi}$$

and passive pressure is equivalent to:

$$\mathrm{K_{ph}} = \frac{1 + sin\phi}{1 - \sin\phi}$$

Rankine's theory is only valid for non-cohesive soils, but can be adjusted for cohesive clay soils as well. As can be seen, earth pressure at a given depth *h* depends on the inclination of the surface δ, the unit weight of the soil *w*, and the angle of friction ϕ. For ideal cases of a dry, non-cohesive fill, ϕ can be determined from lab tests.

It is important to note that fills behind retaining walls are rarely dry and uniform. In addition to rainwater increasing the pressure of the soil, frost action and other influences may also temporarily increase the value over that of the theoretical active pressure, which will cause walls to crack or even fail. Therefore, it is good practice to select conservative values for ϕ, much smaller than actual test values (Nilson, Darwin, & Dolan, 2010). Table 17.1 In *Design of Concrete Structures*, which can be seen in Appendix D: Reinforced Concrete Design Tables, gives representative values for *w* and ϕ used in engineering practice, however these do not consider additional pressures due to pore water, seepage, and frost. In addition the table includes some values for coefficient of friction between concrete and soil.

2.6.3c Common Loading Conditions

For computing earth pressures on walls, three common conditions of loading are most often met: (1) Horizontal surface of fill at the top of the wall, (2) Inclined surface of fill sloping up and back from the top of the wall, and (3) Horizontal surface of fill carrying a uniformly distributed additional load (surcharge), such as people. The increase in pressure caused by the uniform surcharge *s* is determined by converting its load into an equivalent height of soil h' above the top of the wall such that:

$$h' = \frac{s}{w}$$

and measuring the new depth from h' to a given point on the wall forming an ultimate depth of $h + h'$. Figure 17.3 in *Design of Concrete Structures* illustrates the pressure for all three cases in terms of magnitude, point of action, and direction of P.

In cases where the groundwater level is above the base of the wall, either permanently or seasonally, the pressure of the soil above the groundwater table is determined as usual, and the part of the wall below the groundwater table is subject to the sum of the water and earth pressure.

2.6.3d Cantilever Retaining Wall Design

Cantilever retaining walls are usually the most common type of earth-retaining structure because they are often the most economical, especially for wall heights that are less than 15 ft. The design of these walls must satisfy two major requirements. First and foremost, the wall must maintain adequate external stability. By this, the wall must remain fixed in a desired location, besides the small movements required to mobilize the active or passive pressures. Those walls with insufficient external stability will experience failure in the soil. Secondly, the wall must also have sufficient internal stability, which means it must have the ability to carry the necessary internal stresses without rupturing. Those walls with insufficient internal stability will experience failure in the wall itself. These requirements are independent of each other and therefore must be satisfied separately (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

A cantilever retaining wall must be externally stable in many ways. It must not slide horizontally, it must not overturn, it must not experience a bearing capacity failure, it must not undergo deep-seated shear failure, and it must not settle excessively. The external stability regarding all the aforementioned cases is dependent on the wall dimensions and on the forces between the wall and the ground (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). When evaluating the external stability of a retaining wall system, engineers will usually consider the wall and the soil above the footing of the wall as one unit, and then evaluate the external stability using static principles (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). In most cases, trial designs will be developed, upon which its external stability will be checked, and finally the design will be refined accordingly to meet the predetermined design criteria. This trial and error process will continue until an optimal design is obtained. External stability analyses can also be accomplished with professional Excel spreadsheets. Spreadsheet solutions can be useful as they reduce the potential for mistakes and expedite the trial and error process (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

Once the wall has been sized to satisfy all external stability requirements, a designer must provide adequate internal stability or structural integrity. This is accomplished by developing a structural design with sufficient integrity to resist the applied loads of the earth on the wall. Most often, cantilever retaining walls are made of reinforced concrete or reinforced masonry. Design requirements such as wall height would dictate which is used, as well as economic considerations. The design of reinforced concrete would adhere to the most recent Building Code Requirements for Structural Concrete per the American Concrete Institute.

2.7 Accessibility Design Requirements

Due to the accessibility issues with the Elm Park Red Wooden Footbridge design, it was important to take accessibility into consideration when developing the alternative designs. This section provides information about all accessibility requirements that were followed for this project. Throughout this project it was necessary to look at not only state and local codes, but also national codes due to the fact that some building codes vary in their specific restrictions compared to others. As a general rule of thumb, the code that contains the most restrictions was followed in order to provide the best accessibility and overall safety.

2.7.1 History of Accessibility Standards

Along with structural integrity issues, accessibility requirements were taken into consideration when designing the Myra Hiatt Kraft Footbridge. Long before the Americans with Disabilities Acts was signed into action on July 26, 1990 there was a movement pressuring for accessibility and equal access. The issue was disabled people were being overlooked. They were being denied easy access to buildings and venues. Their civil rights were being infringed simply because of a disability, and thousands of people congregated to lobby for the disabilities rights movement (Mayerson, 1992).

Many say that the shift that ultimately led to the disabilities rights movement was Section 504 of the 1973 Rehabilitation Act, which stated that those who received federal funding could not discriminate on the account of a disability. This act led to the realization that segregating and excluding those with a disability is discrimination (Mayerson, 1992). After the enactment of Section 504; the Department of Health, Education and Welfare (HEW) was given the task of developing regulations to ensure its implementation. Finally on May 4, 1977 after lawsuits, sit-ins, and negotiations the regulations of Section 504 were issued.

In the early 1980's President Ronald Reagan attempted to deregulate Section 504, as it was burdensome to business. However, the disability community fought back for their rights. They wrote letters to the White House, protested, and testified, and after two years the deregulation was halted. During the 1980's there were multiple Supreme Court rulings, committee formation and lawsuits that led to the American with Disabilities Act being presented on September 7, 1989 (Mayerson, 1992).

The commitment of activists, the rulings of the Supreme Court, and the support of many others ultimately led to the disability requirements that exist today. Accommodations were no longer seen as an act of charity, but rather basic civil rights.

2.7.2 AAB and ICC Relevance

As part of this projects unique process, every aspect required adherence to the standards set forth by the Massachusetts Architectural Access Board. The Architectural Access Board (AAB) is a regulatory agency within the Massachusetts Office of Public Safety. The board develops and enforces regulations designed to make public buildings accessible to, functional for, and safe for use by persons with disabilities (Executive Office of Public Safety, 2014). This board enforces mandates set forth by the "Americans with Disabilities Act" standards for accessible design. To carry out the boards mandate, the rules and regulations that appear in the code of Massachusetts Regulations as 521 CMR 1.00 have been developed and amended to incorporate disability design standards. These regulations are incorporated in the Massachusetts building code as a "specialized code" making them enforceable by all local and state building inspectors.

The purpose of these regulations are to provide full and free use of buildings and facilities for all persons with any type of disability, so that they may have the same education, employment, living, and recreational opportunities as all other persons in the state of Massachusetts, and so that they may assume full responsibilities as Massachusetts citizens.

The AAB's jurisdiction is triggered by any construction, renovation, remodeling, or alteration of a public building, facility, or a change in use of a building from private to public (Executive Office of Public Safety, 2014). Public buildings or facilities are defined as "those buildings or facilities that are constructed by the Commonwealth or any political subdivision thereof with public funds and open to public use, as well as privately financed buildings that are open to and used by the public" (Executive Office of Public Safety, 2014).

In addition to the requirements by the Architectural Access Board, Massachusetts has also adopted the standards set forth by the International Code Council, with some amendments. The International Code Council is a group dedicated to developing model codes and standards used in design, build and compliance process. Their mission is to protect the health, safety and welfare of people by creating safe buildings and communities. The international codes, or I-codes, provide a complete set of comprehensive building safety codes (International Code Council, 2014). All fifty states have adopted the International Building Code (IBC) at either the state, or jurisdictional level, including Massachusetts. On February 6th, 2011 Massachusetts adopted the 2009 International Building Code as their own personal code, with specific Massachusetts amendments (Watchorn, 2011). It is important to understand how the specific requirements in the IBC compare and differ from those specified by the AAB. These standards were assessed and addressed in the final site design.

The requirements set forth by the AAB and International Code Council would be vital throughout the design process. Elm Park is a public facility that is used frequently by pedestrians and needs to abide to the standards and restrictions specified by both the AAB and the IBC. The bridge and landscape would need to be accessible for all disabled persons along the walkways surrounding and leading up to the bridge. In order to accomplish this task, design criteria would need to be developed for the landscape portion of the project, with specific reference to the requirements for walkways, landings, and handrails. These specifications can be found in Massachusetts Building Code 521 CMR and International Building Code 2009 Edition Section 1013.

2.7.3 *Walkways*

One of the features of the landscape design that would need to be altered within Elm Park as a result of the new bridge construction are the walkways leading up to the bridge. Therefore, the design of these walkways would need to comply with the standards for accessible walkways set forth by AAB. A walkway is defined as an interior or exterior pathway with a prepared surface intended for pedestrian use. They shall include but not be limited to all walks, sidewalks, overpasses, bridges, tunnels, underpasses, courts, and other pedestrian pathways, and shall comply with specific requirements as set forth by section 521 CMR 22: Walkways, of the Massachusetts Building Code. (AAB, 2013)

Walkways shall have a minimum width of 48 inches, excluding curbstones. An obstructed or clear path of travel shall be provided which is at least 36 inches, excluding curbstones (521 CMR 22.2). Walkways shall not have a running slope, or slope parallel to the direction of travel, greater than 1:20 (5%) Nowhere shall the cross-slope, or slope perpendicular to the direction of travel, exceed 1:50 (2%) (521 CMR 22.3.1). Walkway surfaces shall be stable and firm, and shall lie generally in a continuous plane with a minimum of surface warping. Grading and drainage shall be designed to minimize pooling of water or accumulation of ice or flow of water across walkways. Slope and cross-slope would play a significant role in eliminating this accumulation of water. Table 1 summarizes the design criteria for walkways in Elm Park:

Table 1: Summary table of walkway requirements from the Massachusetts building code

Minimum Width	48 inches (Clear Path of 36 inches)
Running Slope	Max of 1:20 (5%)
Cross-Slope (Pitch)	Max of 1:50 (2%)
Surface	Stable, firm, Clear of Obstructions
Drainage	Minimize Accumulation of Water

2.7.4 *Bridge Wings*

Similar to the slope requirements of the walkways in the park, the bridge wings have minimum slope requirements. According the ADA, the bridge wings can be classified as ramps. As a ramp with railings, the slope requirements are less stringent than those for the walkways in the park. The minimum width of the bridge wings however is greater. All values can be found in the 521 CMR 24 and are summarized in Table 2 below.

Table 2: Summary table of bridge wing requirements from the Massachusetts building code

Minimum Width	48 inches(clear from railing to railing)
Maximum Slope	1:12 (8.3%).
Maximum Rise	30 inches

2.7.5 Landings

Landings would also need to be considered as a design requirement. As a result of the slope of the bridge being made accessible for all disabled persons, it would now be considered a ramp in the eyes of the Architectural Access Board. A ramp, per AAB requirements, is required to have at a minimum, a landing at the bottom and top of each ramp run. These landings serve the purpose of allowing disabled individuals the opportunity to turn or rest. What this means is that each walkway on the West and East side of the park leading up to the bridge would now culminate in a landing at the beginning of each bridge wing before citizens traverse the bridge to the other side of the park. This landing would need to adhere to the guidelines set forth by Massachusetts Building Code 521 CMR 24.4: Landings (AAB, 2013).

Landings shall be level and unobstructed by projections or door swings (521 CMR 24.4.1). The landing width shall be at least as the wide as the ramp run leading to it (521 CMR 24.4.2). In the context of the Myra Hiatt Kraft Footbridge, this means the landing would need to be at least as wide as the bridge ramp itself. The length of the landing shall be a minimum of 60 inches of clear space (521 CMR 24.4.3). If ramps change direction at the landings, the minimum landing size shall be 60 inches by 60 inches. The landings at the Myra Hiatt Kraft Footbridge would not require a change in direction, and therefore would not need to abide to this requirement. Additionally, landings with drop-offs shall have edge curb, walls, railings, or projecting surfaces that prevent people from slipping off the landings (521 CMR 24.8). Table 3 summarizes the design criteria for landings in Elm Park:

Table 3: Summary table of landing design requirements from the Massachusetts building code

Surface	Level, Firm, Unobstructed, Minimize Water Accumulation
Minimum Width	Width of Ramp Run leading to it
Minimum Length	60 inches
Edge Protection	Edge curbs, walls, railings, or projecting surface if landing has drop-off

2.7.6 Handrails

In addition to the handrails that span the bridge itself, there is a chance that handrails would need to be implemented on the landings as well. Grading conditions and slope of the landscape from the landings to the pond edge may necessitate railings due to the height of the drop off between the edge of the landing and the ground below the landing. For this reason, it is important to be familiar with AAB specifications regarding handrails on landings and ramps (AAB, 2013).

AAB guidelines explicitly state that handrails shall be provided at all ramps, such as the bridge ramp on the East and West sides of Elm Park. Handrails shall be provided along both sides of the ramp or landing segments (521 CMR 24.5.1). Handrails shall also be provided in pairs, with the first being at a height between 34 and 38 inches from the walking surface, and the lower one at a height between 18 and 20 inches from the walking surface (521 CMR 24.5.2). Additionally, handrails shall be continuous without interruption, excluding where doorways occur, so that a hand can move from one end to another without interruption. An important consideration is that handrails need to extend at least 12 inches beyond the top and bottom of the ramp, into the landings, and be parallel with the ground surface (521 CMR 24.5.4). This would be especially important during the landing design portion of the project, as determining how to integrate railings on the landing with the railings on the bridge would be finalized.

Regarding specific handrail requirements, they need to have a circular cross section with an outside diameter of 1.25 inches minimum or 2 inches maximum. Furthermore, the shape needs to consist of a circular or oval cross section (521 CMR 24.5.6). When a handrail is mounted adjacent to a wall, the clear space between the handrail and the wall needs to be at least 1.5 inches (521 CMR 24.5.8). Lastly, the ends of the handrails shall be either rounded or return smoothly to the floor, wall, or post, and they shall not rotate within their fittings (CMR 24.5.10).

Table 4 summarizes the design criteria for any handrails that need to be installed on the bridge or landings in Elm Park:

Table 4: Summary table of handrail design criteria from the Massachusetts building code

Height of Handrail Pairs	Lower at 18-20 inches Higher at 34-38 inches
Location	All ramps and landings with drop-offs, both sides of ramp or landings, parallel to running slope or ground surface, extend 12 inches beyond top and bottom of ramp into landings.
Cross Section	Circular or Oval Minimum Outside Diameter of 1.25 inches Maximum Outside Diameter of 2 inches
Clearance from wall	Minimum of 1.5 inches
End Condition	Rounded or returned smoothly to floor

2.7.7 Guards

The standards for handrails in section 1012 of the IBC are almost the exact same as those required by the AAB. The heights, locations, cross sections, clearance from wall, and end conditions of the railings are all the same requirements as those specified by the AAB. However, the IBC uses different terminology when referring to boundaries that serve as protection from drop-offs, which they refer to as guards. Guards are what will be needed on the landings at the bottom of each bridge wing per the IBC, and these requirements are found in section 1013 of the IBC (International Code Council, 2014).

A guard is another term used instead of railing, specifically when drop-offs are present between ramps or landings and the ground adjacent to them. Guards are required along landings that are located more than 30 inches measured vertically to the floor or grade below at any point within 36 inches horizontally to the edge of the open side. They shall also be adequate in strength and attachment in accordance with section 1607.7 of the IBC. Required guards shall not be less than 42 inches high, measured vertically above the adjacent walking surfaces (IBC 1013.2). Additionally, required guards shall not have openings, which allow passage of a sphere 4 inches in diameter from the walking surface to the required guard height, the one exception applicable being from a height of 36 inches to 42 inches; guards shall not have openings, which allow passage of a 4.375-diameter ball (IBC 1013.3). Table 5 summarizes the design criteria for guards needed on the landings at both ends of the bridge in Elm Park:

Table 5: Summary table of guard design criteria from Massachusetts building code

Location	Landings located more than 30 inches to the adjacent grade below.
Height	Minimum 42 inches
Openings	Cannot allow passage of 4.375 inch diameter ball at any opening in guard

2.8 Moving the Bridge

In order to start the process of renovating the bridge and the rest of Elm Park, the bridge itself needed to be moved. There were three ways in which this could be done; taking the bridge apart piece by piece, moving it in three separate sections and moving the whole bridge at once. By moving the bridge in pieces it would be impossible to do any last minute evaluations. Using this method would also cause another issue regarding the time it would take to completely take apart the bridge. The transportation of each individual member would take a longer amount of time and would require more people to assist in the process. Moving the bridge in three sections was another possibility, but even this idea had its limitations. The bridge could never be analyzed the way it had stood again and there was a possibility that by sectioning the bridge it would collapse. The favored option that was chosen was to move the bridge in one piece, which can be seen in Figure 1. In order to do this the bridge was braced with wood members along the longitudinal axis on both sides of the railings. It was then lifted into the air by a crane and placed on a large truck flatbed. This truck then drove the entire bridge to the Worcester Technical High School, where it remained until it was taken apart over time, piece by piece and disposed of.

Figure 20: Image of the Elm Park Bridge being removed from Elm Park. Source: (Worcester Magazine, 2013)

3.0 Methodology

In order to design the new Myra Hiatt Kraft Memorial Bridge, a number of steps had be completed to ensure a successful design and construction of the bridge. Once the structure was designed and dimensions were decided, various site specific tasks needed to be completed. Among these are the site design of the park around the bridge and the soil exploration. In addition, a foundation bearing capacity analysis and the new abutment design to match the new bridge needed to be completed.

3.1 Bridge Design

The structural design of the Myra Hiatt Kraft Footbridge was comprised of many different steps. After completing a condition assessment of the bridge the group began by creating architectural designs of the bridge. These designs comprised of an exact replication and various ADA compliant designs. Once an architectural design was decided on by the stakeholders the engineered design for the bridge was developed using NDS design standards and loading conditions based on the Massachusetts Building Code. The four designs presented to the stake holders at various points in the project were an exact replication, an exact replication with switch back, an ADA compliant design modeled after the 1970s historic design, and an ADA compliant historically modified design modeled after the 1900s historic design. All of these iterations can be seen Appendix E: Architectural Bridge Design Iterations Not Chosen.

3.1.1 Design Considerations

From the analysis and conditions assessment shown in Appendix F: Conditions Assessment, a replication design was developed. Using measuring tapes and geometric calculations the member dimensions and lengths each member were determined and allowed for an exact replication. Trigonometric calculations were used to find the slope of the wings to calculate the total height of the bridge, as the measurements were not possible due to the position of the bridge. All of the measurements were used to create a 3D CAD model of the bridge using SolidWorks software. One of the biggest drawbacks of the current bridge is that it is not traversable by all park patrons, so multiple designs were created to be compliant with ADA and AAB requirements.

In order to create a compliant design various design requirements had to be met. These requirements can be found in *Section 405: Ramps* and *Section 505: Handrails* of the 2010 ADA Standards for Accessible Design publication (U. S. Department of Justice, 2010), and *Section 24: Ramps* of the AAB Rules and Regulations, 521 CMR (Massachusetts Office of Public Safety, 2006). For the initial design process, the maximum design requirement for the support structure to be handicap accessible is a slope of 1:12, or about 4.75°. In addition to the compliancy aspects, a large consideration was ensuring that the new bridge closely resembled the previous bridge due to its historical importance. Three designs were created for presentation purposes to the Worcester City Manager's Office. From these presentations, the final compliant architectural design was chosen in order to optimize the design.

Once the Worcester City Manager's Office, the Worcester Parks Department, and the Department Heads at the Worcester Technical High School decided on a final architectural design, the structural analysis portion began.

3.1.2 Software Modeling

Because of the complexity of the design, finite element analysis software was used to aid in the design of the bridge. This method required the group to first decide on an overall geometry and then assign member properties to each member. Then, using the support conditions and loading conditions, loads were applied and the analysis was run to get member stresses and deflection.

3.1.2a Geometry

The design for the Myra Hiatt Kraft Footbridge was very complex. Rather than using a truss design as is in many other typical bridges, the aesthetic constraints required the bridge be designed with a number of beams and columns. Due to this configuration, the bridge was indeterminate. Rather than calculating the axial, bending, and shear forces in each member by hand, a different approach was selected to analyze the bridge design to ensure that the selected design was safe and serviceable. The analysis was performed using STAAD finite element software.

Before the bridge could be analyzed, it had to be constructed within the software. This began by assigning coordinates to each of the bridge's nodes. These nodes represented the point of each member connection and endpoint. As the design is symmetrical about both the *x*- and *y*-axes, only one half of one support span of the bridge needed to be modeled (Figure 21). Then copies could be created and aligned for the remaining five parts of the structure, and finally connected together as necessary. The nodes for the final STAAD version of the bridge are slightly different than the intended for a few reasons, but the nodal numbering scheme shown in Appendix G: Results of STAAD Analysis, was used for presenting all member data.

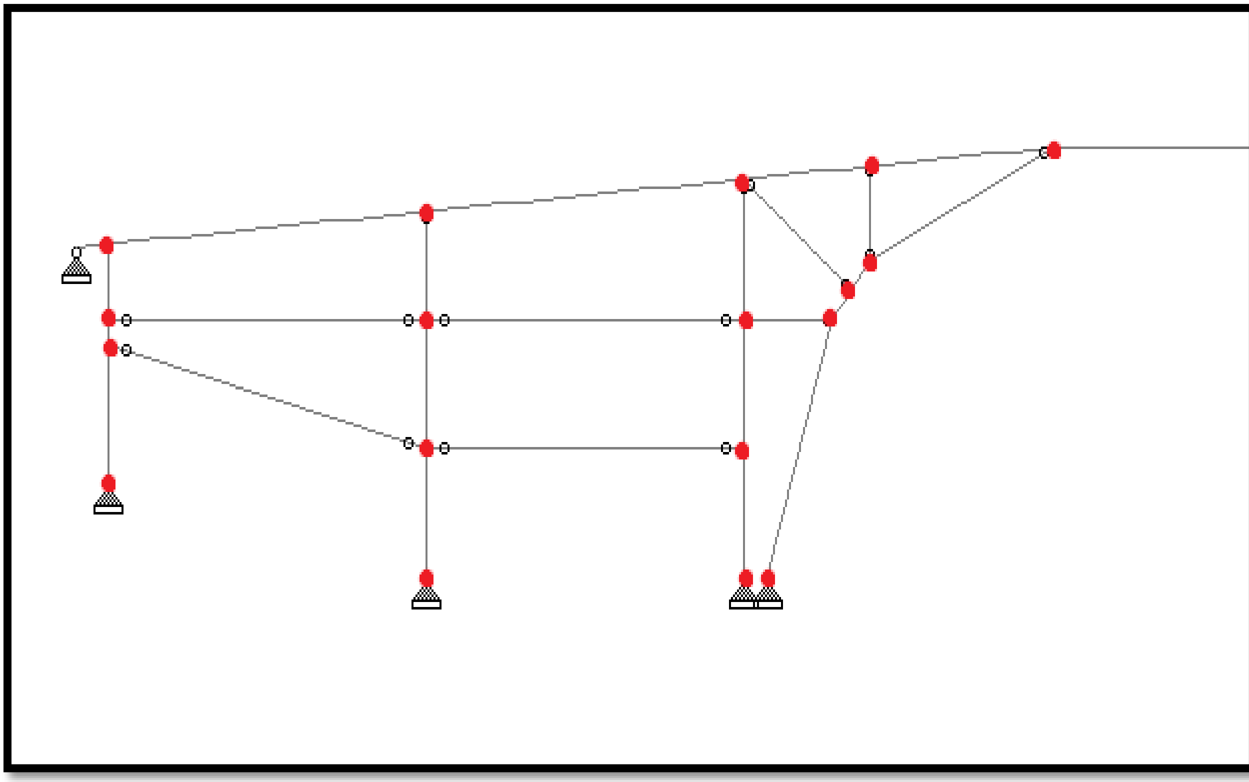


Figure 21: STAAD model with nodes highlighted

The next step in defining the bridge's geometry in STAAD was to create members. Representing each beam and column in the design, members were defined by connecting the nodes to form a wire-frame structure. In total, 13 members were defined for the primary half-span of the bridge. This method treated each "member" in the software as a separate member in the design. Unfortunately, this approach considered the columns, for example, to be three individual members connected with fixed connections at the intersection point with the beams, rather than as one member with connections along its full length. While STAAD allowed for "physical members" to be created by selecting the collinear members of the column, the rigidly connected physical members were not analyzed properly and thus not used. However, the fixed connections of the columns allowed it to be analyzed in the same way as a continuous member.

Additionally, defining the arch members proved to be quite difficult with the software. A model was initially created in which the arch was formed by connecting the two end points and defining a radius of curvature of 92.81 inches, according to the CAD model of the bridge. The locations along the arch of the beam and two chords were approximated as free ends of the members. Then they were extended to the arch to form a node at each of the three locations along the arch. While this accurately modeled the geometry of the arch and its adjacent members, the software was not able to perform the analysis of the arch members. To mitigate this issue, the best solution for the arch was to form linear members between each connection of the arch. Though slightly inaccurate, this was the only means of producing results from the arch in STAAD. This inaccuracy was one reason for checking the factor of safety of all members to ensure that it was above 1.5.

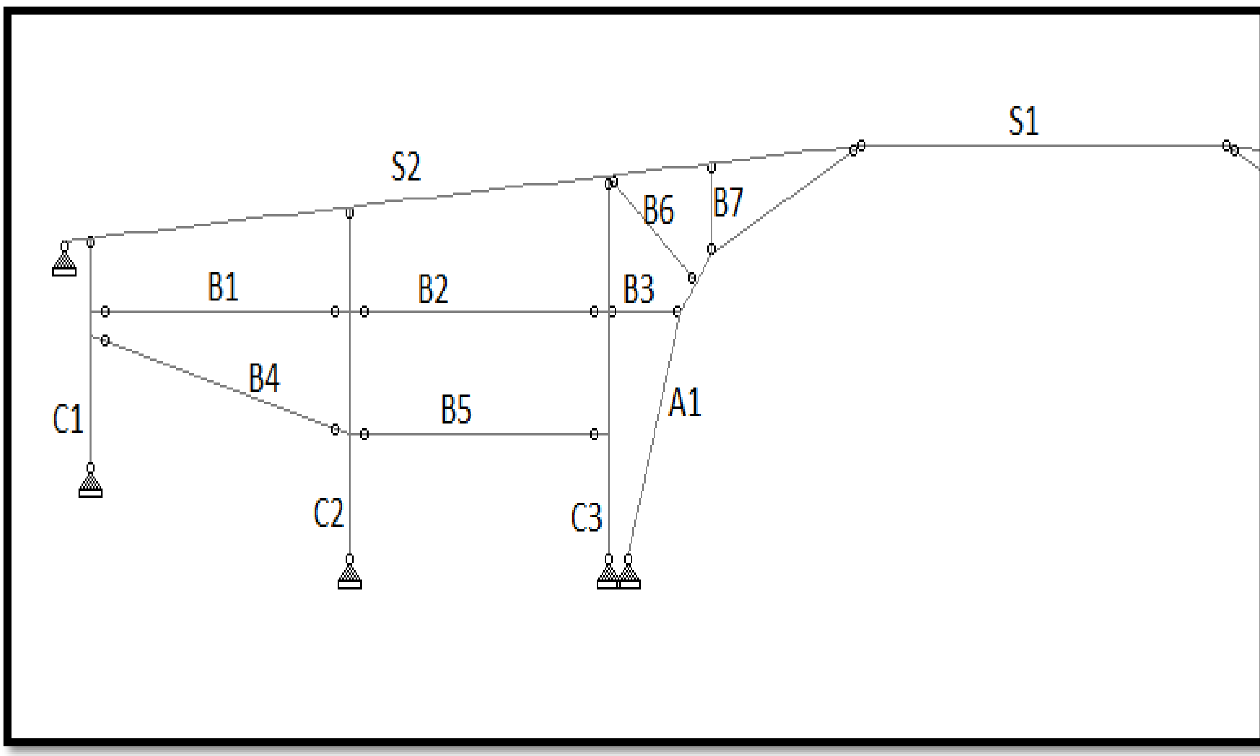


Figure 22: STAAD half bridge members labeled

The numbering in Figure 22 were applied such that C1, C2, and C3 are for column 1, column 2, and column 3. B1 through B7 are beams 1 through 7. A1 is the linear segments representing the arch and S1 and S2 are the two decking support members. Table 6 below shows the member names and their corresponding node connections:

Table 6: Member names and numbers, and their corresponding nodal connections

C1 – Column 1	6-7-8-13
C2 – Column 2	1-4-9-14
C3 – Column 3	2-5-10-15
B1 – Beam 1	8-9
B2 – Beam 2	9-10
B3 – Beam 3	10-11
B4 – Beam 4	4-7
B5 – Beam 5	4-5
B6 – Beam 6	15-18
B7 – Beam 7	16-19
A1 – Arch	3-11-17-18-19
S1 – Decking Support 1	19-38
S2 – Decking Support 2	12-13-14-15-16-17

The nodal points can also be seen in Figure 23 below.

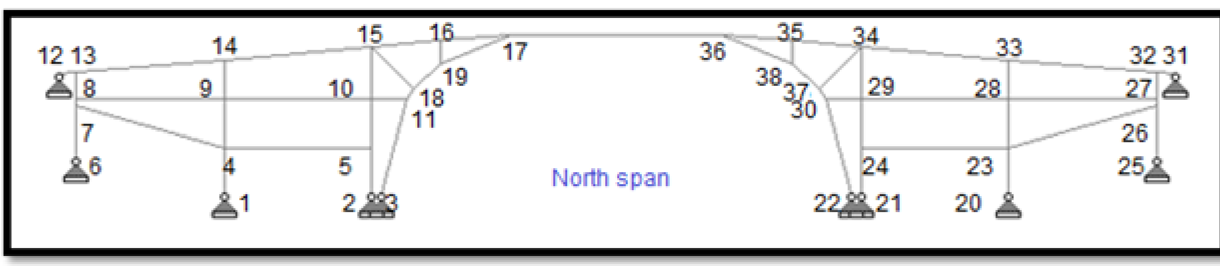


Figure 23: Nodal numbering scheme for one span of the bridge.

Once the first span's nodes and members were defined, it could be mirrored across several axes to the form the entire bridge superstructure. The global axes used in STAAD are defined as follows: the *x*-axis runs along the span of the bridge, approximately west to east; the *y*-axis is perpendicular to the ground, representing elevation; and the *z*-axis is depth into the entire bridge structure between spans, approximately south to north. To form one full span of the bridge, the primary model was copied and rotated about the *y*-axis to be put into place on the opposite end of the foundation. A horizontal member was then defined, connecting the two opposite decking support members where they met the opposite arches. This member represented the main decking support bending member of the bridge at its apex. One full span of the bridge was complete.

Finally, the full bridge could be formed. In order to do this, the full span was copied and translated about the *z*-axis then placed at equidistant intervals. The three full spans were in place parallel to each other along the span of the bridge. However, several cross-bracing members still needed to be formed to complete the geometry of the bridge. Four diagonal members were used here for the cross bracing, forming an X-shape between the middle span and its two adjacent spans. An additional horizontal member was also included below the X-shape, these can be seen in Appendix G: Results of STAAD Analysis. After these members were assigned, the final geometry of the wire-frame was assigned as seen in Figure 24 and Figure 25. Materials and dimensions could be applied next.

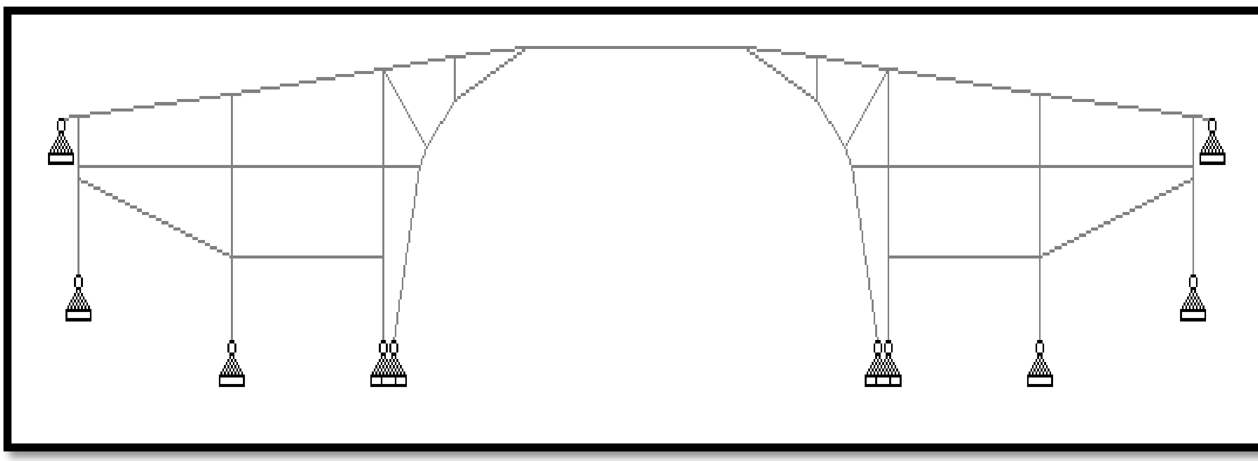


Figure 24 STAAD wireframe model, single span.

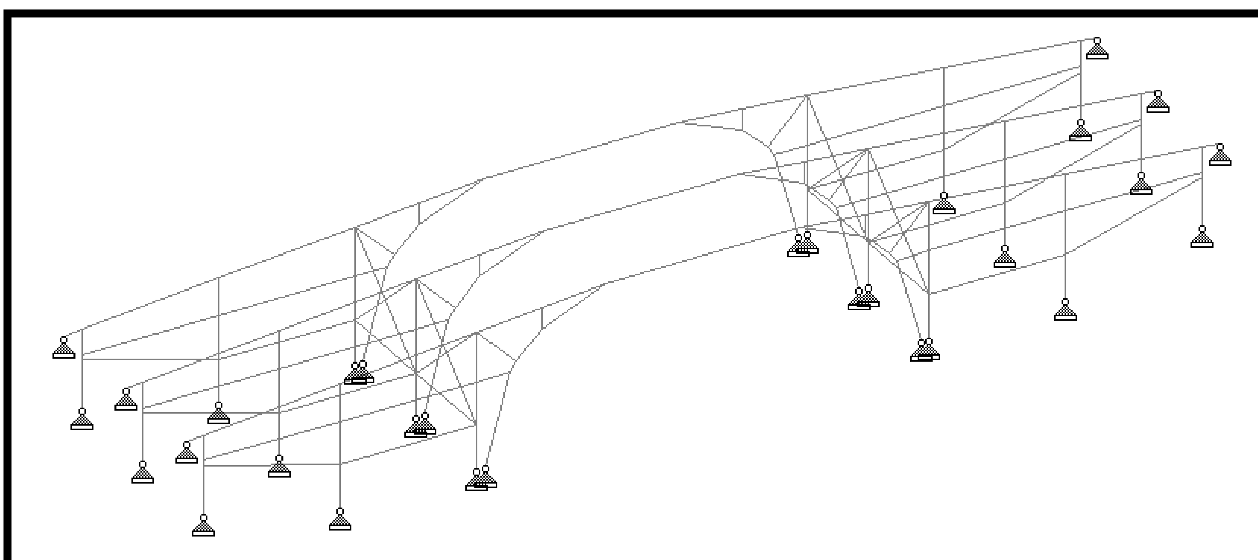


Figure 25: STAAD wireframe model, isometric view of full bridge.

3.1.2b Member Properties

Once member sizes were determined they were implemented into the analysis software. STAAD software allows for materials and member dimensions to be assigned simultaneously through the use of Properties. Selecting Properties allows one to choose between numerous materials with predefined mechanical properties as well as section dimensions and properties. Among these materials was a plethora of wood species with properties defined by the American Institute of Timber Construction (AITC). Southern pine was the preferred material for the constructed bridge and therefore southern pine was the selected material for the STAAD analysis. Predefined mechanical properties for the southern pine species can be found in the NDS, and have been copied into Figure 26 and Figure 27.

Table 4B Reference Design Values for Visually Graded Southern Pine Dimension Lumber (2" - 4" thick)[1,2,3,4,5]

(Tabulated design values are for normal load duration and dry service conditions, unless specified otherwise. See NDS 4.3 for a comprehensive description of design value adjustment factors.)

USE WITH TABLE 4B ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Grading Rules Agency
		Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	Modulus of Elasticity E_{min}	
SOUTHERN PINE									
Dense Select Structural	2" - 4" wide	3,050	1,650	175	660	2,250	1,900,000	690,000	
Select Structural		2,850	1,600	175	565	2,100	1,800,000	660,000	
Non-Dense Select Structural		2,650	1,350	175	480	1,950	1,700,000	620,000	
No.1 Dense		2,000	1,100	175	660	2,000	1,800,000	660,000	
No.1		1,850	1,050	175	565	1,850	1,700,000	620,000	
No.1 Non-Dense		1,700	900	175	480	1,700	1,600,000	580,000	
No.2 Dense		1,700	875	175	660	1,850	1,700,000	620,000	
No.2		1,500	825	175	565	1,650	1,600,000	580,000	
No.2 Non-Dense		1,350	775	175	480	1,600	1,400,000	510,000	
No.3 and Stud		850	475	175	565	975	1,400,000	510,000	
Construction	4" wide	1,100	625	175	565	1,800	1,500,000	550,000	
Standard		625	350	175	565	1,500	1,300,000	470,000	
Utility		300	175	175	565	975	1,300,000	470,000	
Dense Select Structural	5" - 6" wide	2,700	1,500	175	660	2,150	1,900,000	690,000	
Select Structural		2,550	1,400	175	565	2,000	1,800,000	660,000	
Non-Dense Select Structural		2,350	1,200	175	480	1,850	1,700,000	620,000	
No.1 Dense		1,750	950	175	660	1,900	1,800,000	660,000	
No.1		1,650	900	175	565	1,750	1,700,000	620,000	
No.1 Non-Dense		1,500	800	175	480	1,600	1,600,000	580,000	
No.2 Dense		1,450	775	175	660	1,750	1,700,000	620,000	
No.2		1,250	725	175	565	1,600	1,600,000	580,000	
No.2 Non-Dense		1,150	675	175	480	1,500	1,400,000	510,000	
No.3 and Stud		750	425	175	565	925	1,400,000	510,000	

Figure 26: Reference design values for dimension lumber southern pine (American Forest & Paper Association, 2011)

Table 4D (Cont.) Reference Design Values for Visually Graded Timbers (5" x 5" and larger)[1,3]

(Tabulated design values are for normal load duration and dry service conditions, unless specified otherwise. See NDS 4.3 for a comprehensive description of design value adjustment factors.)

USE WITH TABLE 4D ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Grading Rules Agency
		Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	Modulus of Elasticity E_{min}	
SOUTHERN PINE		**(Wet Service Conditions)**							
Dense Select Structural	5" x 5" and Larger	1,750	1,200	165	440	1,100	1,600,000	580,000	SPIB
Select Structural		1,500	1,000	165	375	950	1,500,000	550,000	
No.1 Dense		1,550	1,050	165	440	975	1,600,000	580,000	
No.1		1,350	900	165	375	825	1,500,000	550,000	
No.2 Dense		975	650	165	440	625	1,300,000	470,000	
No.2		850	550	165	375	525	1,200,000	440,000	
Dense Select Structural 86		2,100	1,400	165	440	1,300	1,600,000	580,000	
Dense Select Structural 72		1,750	1,200	165	440	1,100	1,600,000	580,000	
Dense Select Structural 65		1,600	1,050	165	440	1,000	1,600,000	580,000	

Figure 27: Reference design values for heavy timber southern pine (American Forest & Paper Association, 2011)

Next specific member sizes of the species were selected in respect to the members used in the conceptual design to ensure that they were satisfactory for the expected loadings. These sizes included cuts of nominal dimensions 4x4, 4x6, 6x6, 6x8, and 6x10. The different southern pine member sizes were assigned to each member as proposed. The support members were all oriented correctly as assigned, but the decking members had to be rotated about their local *x*-axes by an angle β. The β angles for the sloped members were 85.3° of 94.7° depending on their orientation, and 90° for the flat members. With member properties assigned, the model of the bridge was finally completed in STAAD.

3.1.2c Support Conditions

Before loadings could be applied, the support conditions of the bridge also needed to be specified. As the supports were provided by the foundation, the bottom-most nodes of each column were required to have defined support conditions. Although the connections for the wooden superstructure to the concrete foundation are intended to be rigid in construction, they likely will not be fully resistant to moment. For this reason, the pinned connection was selected at the foundation. This is a conservative method of analyzing the bridge, producing larger forces in the columns and thus adjacent members during analysis than what may actually occur in practice. In addition to the connections to the foundation, the location where the "free" end of the decking supports at the ends of each span was also given a support condition. The end should be resting on the concrete platform beyond the foundation, according to construction plans, and therefore a roller connection was assigned to this node. Overall, there were twenty four pinned supports and six rollers.

3.1.2d Loading Conditions

With the geometry, member properties, and support conditions of the bridge designed, the design loadings could then be assigned to the bridge. The following loads were considered for the Elm Park Bridge.

- Dead load
- Live load
- Snow load
- Wind load
- Seismic load

The values of these loads were determined through numerous design regulations and empirical data. First, dead loads consisted of the weight of each member of the superstructure, as well as elements affixed to the superstructure. STAAD calculated the weight of the individual members and applied it to the structure by using the density of the wood, ρ, and the dimensions of each member, b, h, and L.

$$D_{self} = \rho(bhL)$$

As a gravity load, the dead load of a member was applied to all members beneath it in the structure. These loads were concentrated on each member while all other loadings were simply distributed along the decking members because the loads are expected to only be applied to the decking surface rather than any of the support members.

The vertical loads applied on the decking included live loads and snow, in addition to the railing dead loads. The dead loads of all members were included in the analysis by the STAAD program by including a self-weight multiplier of (-1) in the y-direction. The only additional dead weight was of the railings along the external supports. This load was calculated by multiplying the density of the wood by the total area of each piece, then dividing the total weight by the total length of the bridge to get a load in pounds per linear foot.

$$D_{rail} = \frac{\sum(\rho A)}{total\ length}$$

Live and snow loads were also applied vertically to the decking. Massachusetts Building Code follows the International Building Code and specifies 100lb/ft^2 as the live load for which to design at maximum capacity of the bridge (International Code Council, 2014). Similarly, the

Massachusetts Building Code specifies that structures in the city of Worcester be able to sustain 55lb/ft^2 snow load on its roof or decking (State Board of Building Regulations and Standards, 2010, p. 82). Originally these loads were going to be applied as area loads on the decking members. However, the program will only allow an area load to be applied perpendicular to a member, and the β angles for the sloped members would not allow for a perpendicular application. Thus, the loads had to be converted into member loads per linear foot. For every foot length along the bridge span, there are a little over 2 decking members. To be conservative, the area load of 100lb/ft^2 live load and 55 lb/ft^2 snow load were converted into pound per linear foot loads of 50lb/ft and 27.5lb/ft respectively.

The most important lateral loads were due to wind effects. Wind loads were selected to act as pressures, or distributed area loads, over the perpendicular faces of each member. This conservative approach maximizes the amount of displacement that may be generated by the wind loads by avoiding any loads in counteracting directions, although it is not likely that winds would be blowing from multiple directions at once. The wind pressure used was based on the Massachusetts building code. Table 1611.4 provides design loads for wind pressure based on the zone within Massachusetts and the exposure level to wind, which are dependent on the height of the structure. Central Massachusetts is represented by Zone 2, which has lower wind speeds than the seacoast but higher wind speeds than the forested areas of western Massachusetts at 100 miles per hour (State Board of Building Regulations and Standards, 2010, p. 82). The exposure applying to parks within a city is Level B, or intermediate exposure. Finally, the height of the bridge is well within the lowest height group of 0 to 50ft. Using these criteria, Table 1611.4 provides a design wind pressure of 17lb/ft^2. This value is especially conservative because the height group for this pressure is very broad, suggesting a wind pressure that is far higher than would be typically expected. Therefore, this pressure is applied directly to one face of each member on the three spans to produce a lateral displacement.

One final consideration for loads is to use the correct factored load combinations. Each loading type may be applied individually in STAAD but results from a single loading are not useful when checking the overall integrity of the bridge; they may be more useful to check how a particular member reacts or deforms under a particular load. The loading factors used by the NDS for wood design were selected based on which generates the largest loading overall. There were two governing loading combinations, depending on the individual member, the one that considered each of the loading types is represented by the following equation:

$$1.2D + 1.6W + L + .5S$$

Because the wind load was less than the live load, the load combination that governed for many members was the following equation:

$$1.2D + 1.6L + \ .5S$$

These loading combinations were applied to the bridge to produce the results of STAAD's analysis. Each of the loading conditions has been applied to the bridge as described above. Whether the force is applied in the direction of gravity or laterally, a displacement and a reaction is going to result in the bridge members. Determining the forces in these members is the goal of the analysis to confirm that the design's members are all within acceptable limits of member stresses. In addition, serviceability of the members, or their ability to safely carry the prescribed loads as characterized by member displacements, is also important to consider. Unfortunately, STAAD only calculates the displacements of the joints so it will not produce results for individual serviceability checks. However, the data on the member forces can be obtained and converted into useable results.

3.1.2e Data Collection

After the geometry of the bridge was defined and the loads were applied, the software analysis could be performed. Results from the analysis were tabulated in the post-processing mode of STAAD. Results included node displacements and reactions and beam forces and stresses. Due to difficulties in interpreting the beam stress data, the forces were chosen instead and eventually converted to stresses. Next, beam forces were tabulated for reactions at the ends of members and for maximum forces within a member. The maximum forces were selected because the strength check that follows the data collection here is used to ensure that the stresses within a member do not exceed a particular limit. The maximum forces were further broken down into axial and shear forces and bending moments.

For each member, the maximum of each force was tabulated for each force and moment. Axial forces, consisting of compression and tension, were first selected. Forces were listed on one of two rows for each member, one labeled "Max +ve" for compression forces and the other "Max –ve" for tension members. The force in each member were tabulated in another excel sheet where these forces were defined as f_c and f_t; a member may only have a force in either compression or tension, not both. Axial forces would define an important characteristic of each member, whether they are in tension or compression, which could be useful in identifying errors in the software analysis.

Next, shear forces were tabulated from the analysis results. Shear forces were generated in two directions along the span of any given member, one along the local y-axis and another along the local z-axis. Additionally, the shear forces were also identified as “Max +ve” or “Max –ve” but this was for identifying the direction along the axis in which the shear was applied. If shear along a given axis had values under both characteristics, the larger of the two shear forces was selected for conservative results. Results again were tabulated in the same spreadsheet as the axial forces and were defined as $f_{v,y}$ and $f_{v,z}$. A member may have shear in both directions due to the shear axes acting independently of one another. After each force was collected, only bending moments remained.

Results for bending moments were displayed similarly to those of shear forces. Two moments were generated about a member, one about the y-axis and another about the *z*-axis, both acting independently of one another. Each of these was again classified as “Max +ve” or “Max –ve.” Next the largest values for the members were also tabulated along with axial and shear forces and were defined as $f_{b,y}$ and $f_{b,z}$. One notable difference here is the units of each force, because the forces were tabulated by pounds while the moments were tabulated by inch-pounds. However, the maximum forces and moments were all converted into stresses with consistent units.

Finally, the last remaining objective of the analysis process is to convert the forces and moments into stresses for compatibility with design strengths. Forces were converted to stresses by using the relationship between stress, force, and area.

$$\sigma = \frac{F}{A} = \frac{f_c}{bh} \text{ or } \frac{f_t}{bh} \text{ or } \frac{f_{v,y}}{bh} \text{ or } \frac{f_{v,z}}{bh}$$

Moments were converted to stresses using the flexure formula, relating bending stress to moment and section properties.

$$\sigma = \frac{Mc}{I} = \frac{f_{b,y}\bar{z}}{I_y} \text{ or } \frac{f_{b,z}\bar{y}}{I_z}$$

In the cases of shear and bending stresses, care was taken to ensure that the geometric properties used corresponded to the correct axes along the members. When each force gathered from the STAAD results was converted to stress with units of pounds per square inch (lb/in^2), the analysis results could be compared to member strength limitations specified by the NDS. If each member's stresses were within the limitations, then the members were all safely able to carry the loadings and the design of the superstructure was sufficient. However, strengths are different for each member depending on a number of factors including geometry.

3.1.3 Spot Checks to Verify Software Analysis

After the software analysis was completed spot checks were performed to check the validity of the STAAD analysis results. Because simple hand calculations are not sufficient for a complex matrix structure such as this one; spot checks were completed on all columns for compressive force and also on the decking members and flat decking support for bending force and deflection.

3.1.3a Load Combinations

In order to complete these spot checks, all loads and load combinations needed to be calculated first. There are four different load types that are pertinent for this footbridge.

- Dead load
- Live load
- Snow Load
- Wind Load

The dead load is the weight of any permanent fixtures on the bridge. The live load is the load of pedestrian traffic on the bridge. The snow and wind loads are the maximum expected snow load and wind pressure for central Massachusetts. The dead load in pounds per square foot is dependent on the member being analyzed. However, for all members the force caused by the dead load is calculated using the following equation:

$$D = \sum G * 62.4 \frac{\mathrm{lb}}{\mathrm{ft}^3} * V$$

where *G* is the specific gravity of the material and *V* is the volume of the member. The uniform live load, *L*, based on the International Building Code is 100 lb/ft^2 because it is considered a pedestrian live load (International Code Council, 2014). To get a uniform load in pounds per foot, the 100-pound load is multiplied by the tributary width of the support member. The uniform snow load, S, is also based on the Massachusetts Building Code is 55 lb/ft^2 (International Code Council, 2014). The wind load, *W*, also based on the Massachusetts building code amendment to the IBC, is 17lb/ft^2 of total wind pressure.

The LRFD Load Combinations set forth in the Massachusetts Building Code can be found in Figure 28, and were used to design the member sizes for the design. The load combination that lead to the greatest loads and thus the most conservative was the following expression:

$$1.2D + 1.6L + .5S$$

1605.2.1 Basic Load Combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

EQUATION 16-1

$$1.4\,(D+F)$$

EQUATION 16-2

$$1.2(D+F) + 1.6(L+H) + 0.5(L_r \text{ or } S \text{ or } R)$$

EQUATION 16-3

$$1.2(D+F) + 1.6(L_r \text{ or } S \text{ or } R) + (f_1L \text{ or } 0.8W) + 1.6H$$

EQUATION 16-4

$$1.2D + 1.6W + f_1L + 0.5(L_r \text{ or } S \text{ or } R) + 1.6H$$

EQUATION 16-5

$$1.2D + 1.0E + f_1L + 0.5S + 1.6H$$

EQUATION 16-6

$$0.9D + (1.0E \text{ or } 1.6W) + 1.6H$$

Figure 28: Load combination equations. Source: (International Code Council, 2014)

3.1.3b Compressive Column Loads

Based on the design, there are three support spans as shown in Figure 29, two of which are external spans and one of which is in internal span, and all will be referred to as such.

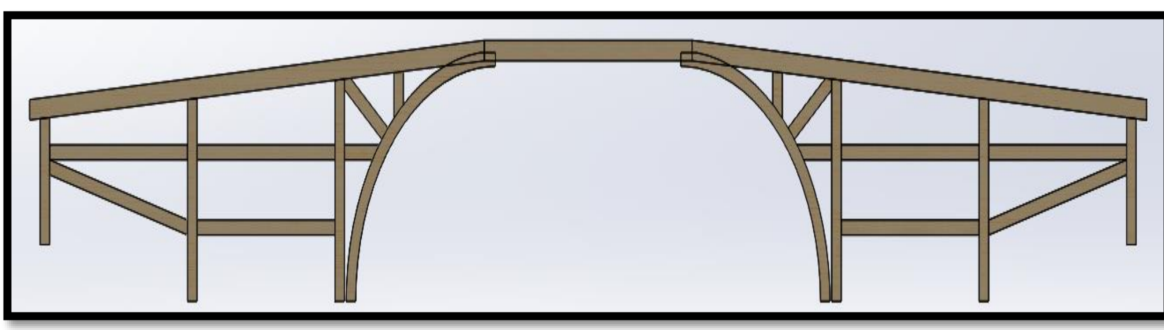


Figure 29: Single span of the final design support structure

For the compressive forces on the columns, the loads from the Massachusetts building code for snow and live load were applied to the structure. The first step was to multiply the loads by the tributary widths. The tributary width of the internal support is 66.75 inches and the tributary width of the external supports is 43.875 inches. The dead load from the railings for the external support was calculated using the equation previously mentioned, and because the railings are fairly uniform throughout, the total load can be divided by the total length of the bridge to get the load in pounds per foot. For both internal and external supports, the load from the decking was also included in the dead load, by using the same equation but dividing it by the total area of the bridge, and then multiplying the loads by the tributary widths. The loads as determined are presented in Table 7.

Table 7: Factored loads applied to the internal and external members on the top of the bridge.

load		internal	external	direction
live	w_L	552.1 lb/ft	365.6 lb/ft	-z
snow	w_S	276 lb/ft	182.8 lb/ft	-z
dead	w_D	65.5 lb/ft	58.5 lb/ft	-z
wind	w_W	0.118 lb/ft	0.118 lb/ft	y

After applying the LRFD Equation, $w_{T,-z} = 1.2w_D + 1.6w_L + 0.5w_S$, the total uniform loads became $w_{T,-z} = 1100\text{lb/ft}$ for the internal support and $w_{T,-z} = 746.6\text{lb/ft}$ for the external support.

Because the bridge is symmetric with symmetric loading, the columns on one side of the bridge can be assumed to hold the same loads at the corresponding columns on the other side of the bridge, as is shown in Figure 29. The short columns are the #1 spot check, the middle columns are the #2 spot check, and the long columns are the #3 spot check, for both the internal and external supports. The calculations and results for all columns can be found in Appendix G: Results of STAAD Analysis, and are based on the following assumptions:

- Load across the tributary area is concentrated as a point load at the midpoint of the column member
- Load is calculated at the bottom of all member sections, to take into account the self-weight of the members themselves
- Loading on the decking supports is the same as that used in the STAAD analysis
- Horizontal wind loading has minimal effect on compressive force
- Does not take into account the distribution of the flat decking member, as it assumes that will be supported by the arch member
- Does not take into account the eccentric loading causes by the sloped decking
- The complexity of the structure is not taken into account, the STAAD analysis however will include this complexity

For support #2, the tributary length was taken as it is traditionally, by dividing the total span from the column being calculated to one side in half and adding that to the total span from the column being calculated to the other side divided by two, as shown in Figure 30.

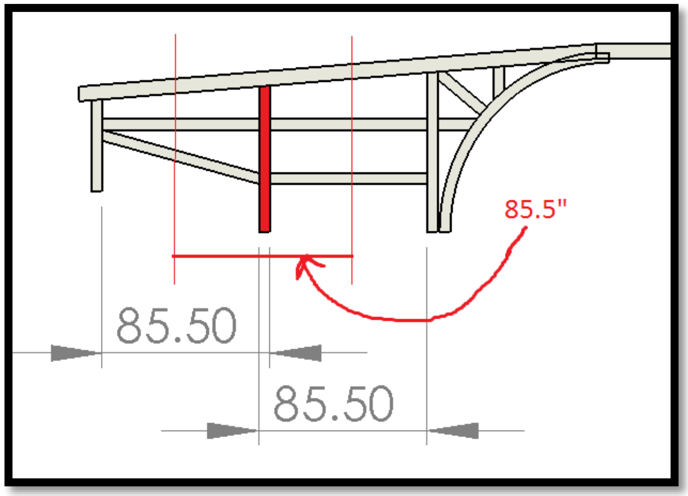


Figure 30: Example of tributary length for manual calculations.

The tributary length is then converted into feet and multiplied by the uniform load to determine a point load for the center of the column. In order to find the minimum usable cross-sectional area, the equation $\frac{Force}{Stress} = \frac{Force}{\frac{Force}{Area}} = Area$ was used. This number was then multiplied by a factor of safety of 2.0 because of that assumption that the complexity of the design was ignored for the spot checks. Then to get the compressive stress in the member the equation, $Compressive\ Stress = \frac{force}{area} = \frac{force}{b*d}$ was used where d and b are the length and width of the cross sectional area of the member. This final equation yields results in pound per square foot, which can then be compared to the output of the STAAD analysis.

3.1.3c Bending and Deflection

For the decking members, and decking support beams, deflection and bending stress were important factors that needed to be designed for in addition to column loading.

Similar to the compressive stress spot checks, the total uniform load for bending strength was calculated as:

$$w_T = 1.2w_D + 1.6w_L + 0.5w_S$$

Next, the maximum moment needed to be determined.

$$M_{max} = \frac{w_T l^2}{8}$$

Finally, using the moment and the member sizes, the bending force could be calculated using the flexure formula.

$$F_b = \frac{M_{max}c}{I} = \frac{M_{max}}{S}$$

In order for the members to be acceptable, F_b must be less than F_b'.

For deflection, the same members were analyzed using the following procedure. The maximum deflection of a simply supported beam occurs at the center of the beam. The deflection at the center of the beam is:

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

For analysis, two deflection criteria must be met:

$$\Delta_L \leq L/360$$

$$\Delta_{KD+L} \leq L/240$$

If the loaded members did not deflect more than the specified limits, then the members were acceptable.

3.1.4 NDS Design Strengths

In order to determine if the member stresses found from the STAAD analysis were acceptable they needed to be compared to the allowable design strengths based on the National Design Specifications (NDS). The design strength values can be found by taking the reference design values and multiplying them by the calculated adjustment values.

3.1.4a Reference Design Values

Once the analysis was completed the values were compared to NDS values. The NDS provides tabulated data on reference design values for each species of wood. These reference design values are unmodified strengths in bending and shear, and provide a basis for calculating the strength of a member when loaded. Adjustment factors were then applied to each design value to determine the expected maximum stress a member can undertake before failing in one of the several failure modes. Most species of wood have one design strength value per type of stress applied for each quality of wood. However, southern pine, the wood selected for the bridge, is unique in that its strengths vary depending on the cross-sectional area of the cut member. Table 8 below summarizes the design strengths for the member sizes selected for this project.

Table 8: Reference design values for southern pine wood using relevant cuts (American Forest & Paper Association, 2011).

			Southern Yellow Pine -- Design Strength Values (psi)							
grade	cut	width (in)	F_b	F_t	F_v	$F_{c\perp}$	F_c	E	E_{min}	Ref
No. 1	4x4	4	1850	1050	175	565	1850	1700000	620000	Table 4B
No. 2	4x6	4	1850	1050	175	565	1850	1700000	620000	Table 4B
No. 2	6x6, 6x8, 6x10	6	850	550	165	375	525	1200000	440000	Table 4D

3.1.4b Adjustment Factors

These design values are unfactored, meaning that they have not been adjusted at all to accommodate for changes in design capacity due to geometry or connection configurations. These adjustment factors may be greater than, equal to, or less than 1, so using unfactored strengths cannot be considered a conservative approach. The NDS proposes the use of a number of factors for each stress orientation. It is important to note before continuing that the design approach used for analyzing the bridge is LRFD due to its more modern approach to analysis and design and greater allowed stresses. The factors for sawn lumber are described below and their applicability is shown in Table 9.

Table 9: Table of applicability of adjustment factors for modifying design values for sawn lumber (American Forest & Paper Association, 2011).

		ASD only	ASD and LRFD										LRFD only		
		Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Member Factor	Column Stability Factor	Buckling Stiffness Factor	Bearing Area Factor	Format Conversion Factor	Resistance Factor	Time Effect Factor
$F_b' = F_b$	x	C_D	C_M	C_t	C_L	C_F	C_{fu}	C_i	C_r	-	-	-	K_F	ϕ_b	λ
$F_t' = F_t$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	-	-	-	K_F	ϕ_t	λ
$F_v' = F_v$	x	C_D	C_M	C_t	-	-	-	C_i	-	-	-	-	K_F	ϕ_v	λ
$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-	C_i	-	-	-	C_b	K_F	ϕ_c	λ
$F_c' = F_c$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	C_P	-	-	K_F	ϕ_c	λ
$E' = E$	x	-	C_M	C_t	-	-	-	C_i	-	-	-	-	-	-	-
$E_{min}' = E_{min}$	x	-	C_M	C_t	-	-	-	C_i	-	-	C_T	-	K_F	ϕ_s	-

Wet service factor (C_M): This factor accounts for moisture conditions in the wood. This factor decreases the strength capacities for moist wood.

<u>Temperature factor (C_t):</u> This factor accounts for exposure to atypical temperatures. This factor decreases the strength capacities for members exposed to exceptionally high temperatures.

<u>Beam stability factor (C_L):</u> This factor is designed to account for the slenderness of bending members. This factor reduces the strength in bending for members susceptible to buckling.

<u>Size factor (C_F):</u> This factor adjusts several design factors depending on the size of the member. For most species, bending, tension, and compression members are strengthened if a smaller than average size is used.

<u>Flat use factor (C_{fu}):</u> This factor also adjusts bending members depending on their geometry. Smaller member sizes are strengthened.

<u>Incising factor (C_i):</u> This factor decreases the strength of members that are incised for pressure treatment. This notably does not affect southern pine species (Breyer, 2007, p. 4.48).

<u>Repetitive member factor (C_M):</u> This factor increases the bending capacity of bending members spaced closely together. Numerous restrictions apply to this for members to be considered, including that they must be of smaller cut sizes, closely spaced, and joined by a common element (Breyer, Fridley, Cobeen, & Pollock, 2007).

<u>Column stability factor (C_P):</u> This factor reduces the compressive capacity of members due to buckling. Especially slender members, which can only withstand a smaller load before buckling.

<u>Buckling stiffness factor (C_T):</u> This factor increases the lateral torsional buckling stiffness of smaller compression members (Breyer, Fridley, Cobeen, & Pollock, 2007).

<u>Bearing area factor (C_b):</u> This factor increases strength of members to resist compression perpendicular to the grain. This applies only when another member is bearing on the member at a certain length from its end (Breyer, Fridley, Cobeen, & Pollock, 2007).

<u>Resistance and format conversion factor (ϕK_f):</u> These factors are applied together to all members when using the LRFD method. The resistance factor is used to reduce the strength of members due to unforeseen properties of the members, such as imperfections in the material. The format conversion factor is used to remove the ASD safety factors, which are deemed obsolete to the LRFD method (Breyer, Fridley, Cobeen, & Pollock, 2007).

<u>Time effect factor (λ):</u> The purpose of this factor is to account for the effect of duration that certain load combinations have. Loads that have a longer effect, such as dead loads, weaken the members over time, and need a factor to account for the loss of strength.

While sawn lumber was used for a significant majority of the members in the bridge, the arch was proposed to be fabricated out of glued laminated, or glulam, wood. Glulam members use similar but slightly different modification factors (seen in Table 10). While most factors remain the same, the newly introduced factors are described below.

Table 10: Table of applicability of adjustment factors for modifying design values for glulam (American Forest & Paper Association, 2011).

Table 5.3.1 Applicability of Adjustment Factors for Structural Glued Laminated Timber

		ASD only	ASD and LRFD								LRFD only		
		Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor [1]	Volume Factor [1]	Flat Use Factor	Curvature Factor	Column Stability Factor	Bearing Area Factor	Format Conversion Factor	Resistance Factor	Time Effect Factor
$F_b' = F_b$	x	C_D	C_M	C_t	C_L	C_V	C_{fu}	C_c	-	-	K_F	ϕ_b	λ
$F_t' = F_t$	x	C_D	C_M	C_t	-	-	-	-	-	-	K_F	ϕ_t	λ
$F_v' = F_v$	x	C_D	C_M	C_t	-	-	-	-	-	-	K_F	ϕ_v	λ
$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-	-	-	C_b	K_F	ϕ_c	λ
$F_c' = F_c$	x	C_D	C_M	C_t	-	-	-	-	C_P	-	K_F	ϕ_c	λ
$F_{rt}' = F_{rt}$	x	C_D	C_M	C_t	-	-	-	-	-	-	K_F	ϕ_v	λ
$E' = E$	x	-	C_M	C_t	-	-	-	-	-	-	-	-	-
$E_{min}' = E_{min}$	x	-	C_M	C_t	-	-	-	-	-	-	K_F	ϕ_s	-

1. The beam stability factor, C_L, shall not apply simultaneously with the volume factor, C_V, for structural glued laminated timber bending members (see 5.3.6). Therefore, the lesser of these adjustment factors shall apply.

<u>Volume factor (C_V):</u> This factor may be used to increase the bending capacity of members when loaded perpendicular to the grain.

<u>Curvature factor (C_b):</u> This factor may be used to reduce the bending capacity of curved members depending on the geometry of the member.

The adjustment factors for each member under the LRFD method were calculated according to the geometries and other properties of each member. They were then applied to each design strength as appropriate, to find the modified design strength values. Each of these modified design strength values were compared to the each of the stresses identified from the STAAD analysis. If the modified strength value was greater than the STAAD stress, the member was deemed acceptable. If the STAAD value was too high, than the member size needed to be adjusted to ensure that it would be acceptable. Alternatively, if the approach to modify the design strength were initially too conservative, adjustments would be made to see if the modified design strength could be adequately increased. Each of the members needed to be stressed below its design strength for the applicable loading conditions for the design to be acceptable.

3.1.4c Interaction Equations

After assuring that the individual stresses would be acceptable, some stresses needed to be checked in combination with others to ensure that interactions would not cause failure. This was tested through interaction equations, which check bending when combined with axial compression or tension. These interaction equations produce ratios, which must sum to less than 1. The following equation is used to check bending and tension interactions.

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \leq 1.0$$

$$\frac{f_b - f_t}{F_b^{**}} \leq 1.0$$

The value F_b* is the modified bending strength value without account for the beam stability factor. The value F_b** is the modified bending strength value without accounting for the volume factor, which is used only in glulam beams. The stresses f_t and f_b are the tension and bending stresses respectively calculated from the STAAD data, where f_b is the greater of the bending stresses f_{bz} and f_{bz}'. If each of these ratios were less than 1, the member was acceptable in combined bending and tension. Compression was also checked with bending through this interaction equation.

$$\left[\frac{f_c}{F_c'}\right]^2 + \frac{f_{bz}}{F_{bz}'[1-(f_c/F_{cEz})]} + \frac{f_{by}}{F_{by}'\left[1-\left(f_c/F_{cEy}\right)-(f_{bz}/F_{bE})^2\right]} \leq 1.0$$

Additionally, the compressive force must be less than the Euler bending forces, F_{cEz} and F_{cEy}.

$$f_c < F_{cEz} = \frac{.822E_{min,z}'}{(l_e/d)_z^2}$$

$$f_c < F_{cEy} = \frac{.822E_{min,y}'}{(l_e/d)_y^2}$$

Each of these inequalities considers the bending about each axis of the bending members, where the *z*-axis is along the width of the beam and the *y*-axis is along the depth. The STAAD compression stress f_c is introduced to this equation. The Euler stresses are dependent on the slenderness ratios (l_e/d) for each axis of the beam. When applied to the interaction equation for compression and bending, these stresses all work to ensure that the beam does not fail through buckling. If the interaction equation was satisfied, the design was finally acceptable.

3.1.4d Factor of Safety

It was also important to the integrity of the design to assure that the structure was more than capable of resisting the design loads. This was established through the use of a factor of safety, beyond that which is established by the design values. The factor of safety was identified by finding the greatest stresses in the members in relation to their design values. When the design value was divided by the stress, the factor of safety was the least ratio for any member and any stress throughout the bridge.

Design for the structure was assessed to ensure that the members were not stressed to failure. The failure was checked by finding the maximum allowable stresses through the modified NDS design strengths. The actual stresses, calculated from the STAAD member forces, were compared to theses stresses to ensure that members would not fail through individual stresses. Finally, the members with combined stresses were assessed through the interaction equations to ensure that they would not fail through combined stresses. With each check performed, the members were all adequate to a factor of safety. The design of the structural members of the superstructure was complete.

3.1.5 Deflections

One final assessment to ensure that the bridge members were adequate was for their serviceability. This was essentially a check on the deflections of the loaded members, specifically, the nine decking support members and the decking members themselves. The actual deflections were calculated for a simply supported beam of length L under a distributed load w.

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

The distributed load was determined by multiplying the design loads by the respective tributary width affecting each loaded member. Deflections had to be considered for two different loading conditions:

- Live load
- Dead load and live load

The distributed load from each of these conditions had to be factored into the deflection equation and checked with a particular criterion for serviceability. The following define the criteria for an acceptable deflection (Breyer, Fridley, Cobeen, & Pollock, 2007):

$$\Delta_L \leq L/360$$

$$\Delta_{KD+L} \leq L/240$$

It was important to note that the dead load D was multiplied by a factor K when determining the distributed load. In this case, K was 0.5 as the wood was dry rather than green. If the loaded members each passed these two deflection checks, the final design of the members was satisfactory.

3.1.6 Connections

Once the members were designed with adequate strength, the connections needed to also be designed. Connection design began with selection of connections that were applicable for the given interfaces. Then calculations were performed to ensure that the connections could handle the forces at the joints. This was completed for the bridge and railings as described in the following subsections.

3.1.6a Qualitative Selection of Connections

In addition to the design of members, the design of the superstructure needed connections to be designed. The joints of each member in the bridge and the railings needed connections, as well as the interface between the bridge and the railings and concrete of the foundation. Connections may consist of nails, lag screws, and bolts and plates. There are numerous styles of connections that may be appropriate for only some geometric configurations. Additionally there are numerous manufacturers of connections, who produce many proprietary connection products. In order to begin the process of selecting connections, professional engineer Mr. Steven Harvey was enlisted to recommend which connection types may be appropriate where.

The process of designing connections began with the consultation of Mr. Steven Harvey. Before any specific connections options were listed, Mr. Harvey recommended the use of connections manufactured by Simpson Strong-Tie Company (S. Harvey, personal communication, January 29, 2014). His recommendation came from having worked with them on previous projects, and thus he knew that they were a reputable dealer and that their connections would have the longevity necessary for this project. However, because this is a public project, the materials will all go to bid, and ultimately Simpson Strong-Tie might not be the final fabricator. To accommodate this, the team used values from the connections from Simpson as references to design for and check for. The required design strengths and connection types were then provided for Mr. Harvey and the city so that they could put the connections up for bid.

The Simpson catalogue was then reviewed to select the most applicable connections for the bridge before further consultation. In general, connections for the superstructure were selected for the following interfaces:

- Columns and arches to foundation
- Bending members to columns and arches
- Cross-supports to columns
- Columns to decking supports
- Decking to decking supports

Additionally, consideration for the railings included

- Vertical members to bridge decking support
- Members parallel to bridge deck to vertical members
  - Topmost members, above vertical members
  - Intermediate members, within vertical members
- Cross bracing to adjacent members

3.1.6b Superstructure connections

Criteria for selection of the connections were based on a few factors. Initially, the selection was based on the applicability of connections to the geometric constraints of the arrangement of the bridge's members. For example, where the columns connect to the bending members, most of the joints are perpendicular. This 90° angle allows for a very wide variety of connections to be placed. Conversely, where the short compression members connect to the arch, the options were much more limited. In this case, the angles at which the members connect are particularly specific to this project (shown in Figure 31). Additionally, the arch would require a flat inset to be cut into it to hold the flat plate of the connection. These geometric constraints played a great role in the initial selection of connections.

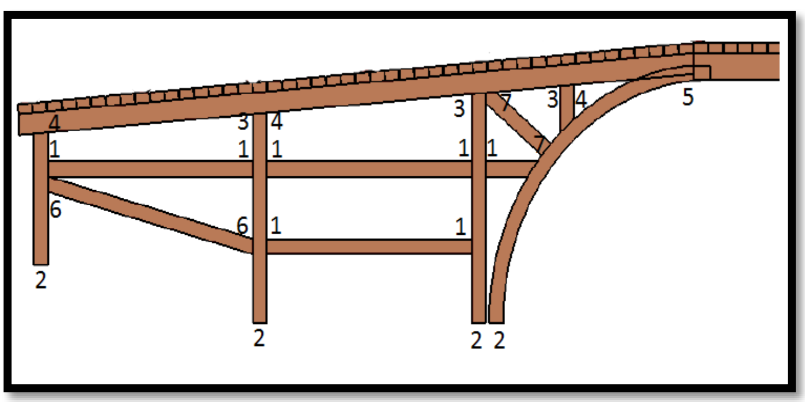


Figure 31: Image with bridge's varying connection types numbered. Each number represents a different connection based on the geometry and location of the connection.

Aside from geometry, a few other criteria needed to be assessed. Aesthetics were important to the preliminary selection to satisfy the clients' wishes for the park. It was important that the connections be discreet so as to not take away from the visual appeal of the bridge, so concealed connections were desired where possible. Some concealed connections were suspected to not be strong enough to hold the greatest of the applied loads so they were not applicable in all cases. Moisture collection was also an important issue to consider. Water from rain and melting snow pooled on flat surfaces and those concealed within the double-shear connections and led to rot at the joints, highlighting the importance of moisture control in this design of the bridge. Plates where water could be trapped were avoided when selecting connections, as well as the use of nails which withdraw much more easily in moist conditions. Finally, the depth of pressure treatment was an important consideration. According to Mr. Harvey, connections should not penetrate through the pressure treated layers of the bridge (personal communication, February 6, 2014). If the layer were penetrated, the member would be weakened, especially if moisture were to get into the non-treated layers. This may not be avoidable if bolts were used but should have applied to nails and lag screws. In conclusion, aesthetics, moisture control, and depth of pressure treatment were additional criteria to consider for selecting connections before a quantitative analysis was performed.

Requiring special consideration was the design of the baseplate connecting the superstructure to the foundation. Geometric constraints were not as rigorous as the columns at the base were square and aligned perpendicularly to the foundations; most baseplate designs would accommodate such shape and angle. Aesthetically, the concealed connection was preferred as the baseplate may be bulky and distracting to the eye. Most baseplate connections were concealed but a few others used metal plates to connect on the exterior of the column. In addition to being exposed, the metal used for these plates appeared quite thin and too weak for this application, according to Mr. Harvey (personal communication, February 6, 2014). These plates also were a concern for moisture being trapped at the joint. One significant issue with the design of the original bridge was that moisture got trapped under the wood sitting on the concrete at the interface, causing rot at such a critical connection. Therefore, it was preferable that the connection include a plastic block between the wood and concrete with drainage capabilities to prevent pooling of water. Finally, the connection could not be cast-in-place as there would be no concrete added to the foundation. Fulfilling each of these criteria was only one connection, the knife plate with pedestal.

3.1.6c Railing Connections

The design of connections for the railings did not require any additional criteria. Discrete connections were preferred but concealed connections were not feasible with the smaller size of the members. Moisture control was not a significant issue due to the lack of places for water to pool, assuming that members were aligned flush in construction. Nails or lag screws were preferable due to the ease of application in construction and the minimal visual obstruction, but they may withdraw from the wood if exposed to enough moisture. Where the railings' posts connect to the superstructure, bolts were preferred to provide a strong connection as they were in the original design of the bridge, but using additional plates here was also considered. As with all other connections, a quantitative analysis would be needed to ensure that the preferred connections were sufficient.

3.1.6d Analysis & Design of Connections

Yield limit Analysis: In order to evaluate the strength of the connections, an analysis was performed based on NDS yield limits of the connections at each joint. Each connection consisted of at least one dowel fastener, or cylindrical rod like nails, lag crews, or bolts. For each dowel connection, the value of *Z'*, the modified yield limit of the connection, was determined according to the NDS modification factors, as shown in Table 11 This represented the shear load that could be applied to a given connection before the connection would fail plastically (Breyer, Fridley, Cobeen, & Pollock, 2007). In most cases, the dowel is the part of the section that fails however this is not always the case. There are several different failure methods which dictate the yield limit of the connection.

Table 11: Applicability of adjustment factors for connections, modified for relevant connection types (American Forest & Paper Association, 2011).

Table 10.3.1 Applicability of Adjustment Factors for Connections

		ASD Only	ASD and LRFD									LRFD Only		
		Load Duration Factor [1]	Wet Service Factor [2]	Temperature Factor	Group Action Factor	Geometry Factor [3]	Penetration Depth Factor [3]	End Grain Factor [3]	Metal Side Plate Factor [3]	Diaphragm Factor [3]	Toe-Nail Factor [3]	Format Conversion Factor	Resistance Factor	Time Effect Factor
Lateral Loads														
Dowel-type Fasteners	Z' = Z x	C_D	C_M	C_t	C_g	C_Δ	-	C_{eg}	-	C_{di}	C_{tn}	K_F	ϕ_z	λ
Withdrawal Loads														
Nails, spikes, lag screws, wood screws, and drift pins	W' = W x	C_D	C_M	C_t	-	-	-	C_{eg}	-	-	C_{tn}	K_F	ϕ_z	λ

Connections may fail in a number of ways. There are six different failure methods of connections which are shown in Figure 32; all of which apply to single shear conditions but only four apply to double shear. Single shear describes the interface between two members connected by a dowel, and while usually this is used for two wooden members, in most cases of this bridge design this applied to a wooden member connected to one metal plate. Double shear may be at the interface of three wooden members, as was the case in the original bridge design, but in this design this condition applies to a wooden member between two metal plates or a metal plate inserted through a wooden member. In either configuration, the *Z* value for each loading case was determined for each failure method and the method with the lowest value was the dictating condition. The value was then modified by the factors to find the load which would cause the connection to fail at the condition. This modified load was used for assuring the strength of the connection but some connections require an additional capacity to be verified.

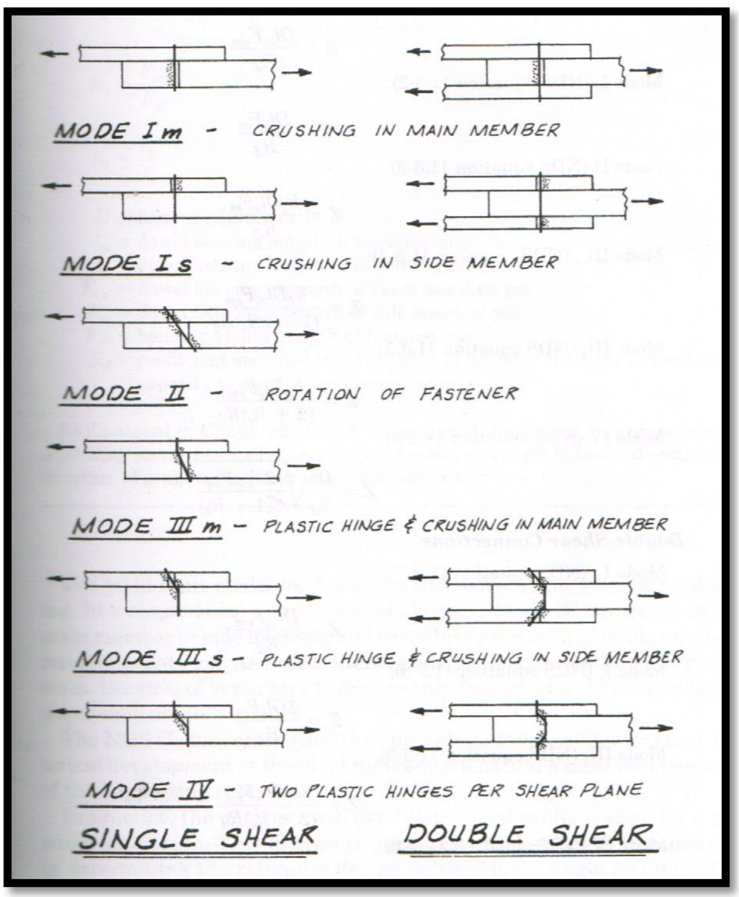


Figure 32: The six failure modes for single shear connections and four modes for double shear connections (Breyer, Fridley, Cobeen, & Pollock, 2007).

3.1.6e Withdrawal Capacity

Some connections may be forced out of their hole if not secured at each end. Connections like nails and lag screws are especially prone to this withdrawal, the outward displacement of the connection from its insertion channel. If tension or a bending moment is applied to the joint, this may force the connection out of place and weaken its strength Z by reducing the dowel bearing length. The load at the joint that causes the dowel to fail by withdrawal is W', modified similarly to Z' to account for geometry of the members and other conditions. For example, moisture within the nails' bearing significantly reduced the withdrawal strength of the member because it causes the wood to expand, forcing the nail out from the inside. Threads may provide some resistance to this withdrawal but most nails are not afforded this luxury. This strength value is not necessary to be considered for bolts because the nut on the threaded end of the dowel is secure enough to prevent withdrawal. After an understanding was gained about the potential failure methods of dowel connections, it was time to apply this to begin the process of calculating the design capacities.

3.1.6f Calculation of connection yield limits

In order to begin the calculations, a number of parameters needed to be identified. The following parameters were based on the geometry of the connection configuration. First, the capacity is proportional to the diameter of the dowel, D. Next, the capacity is also dependent on the bearing length of the dowel within the members or plates. Bolts, which go completely through the members, require only the full depth of the members, while nails and lag screws have a penetration depth that is typically not equal to the entire depth of the member. It was important to understand which shear mode was acting at the bolt so that the position of the members, either side or main, could be considered. The main member was the larger member in single shear or the middle member in double shear, while the side member was the plate or smaller member in single or the plates in double shear. (For the knife plate connection to the foundation, the plate is the main member while the sections of the column are the side members.) The dowel bearing length through one side member is l_s and through the main member is l_b. The dowel diameter and bearing lengths define the geometric properties used to calculate the load capacities.

There are several other factors for yield limits based on the strengths of the connected members. First, the dowel bearing strength of the main and side members, F_{em} and F_{es} respectively, was to be identified. The dowel bearing strength is the "load [which] represents a 5 percent diameter offset on the load-deformation curve obtained from a bolt diameter test" (American

Forest & Paper Association, 2011). For A36 steel, the dowel bearing strength is 58000psi, roughly the ultimate strength of steel (American Forest & Paper Association, 2011). The dowel bearing strength of wood varies from species to species but for southern pine, this is value is 3650psi (American Forest & Paper Association, 2011). In addition to the bearing strengths of the members, the bending yield strength of the dowel, F_{yb}, is also needed. This varies depending on the type of connection but for bolts, this is 45000psi (American Forest & Paper Association, 2011). Finally, there are a number of dimensionless factors involved in calculating the yield limits which are defined in Appendix H: Connection Calculations.

If the geometric and mechanical properties of the wood and steel at the connection are known, the yield limits can be identified. The yield limits were first calculated before adjustment factors were applied. The yield limits were calculated for each of the six failure method using the following equations for single shear.

$$\mathrm{I_m} \qquad Z = \frac{Dl_m F_{em}}{R_d}$$

$$\mathrm{I_s} \qquad Z = \frac{Dl_s F_{es}}{R_d}$$

$$\mathrm{II} \qquad Z = \frac{k_1 Dl_s F_{es}}{R_d}$$

$$\mathrm{III_m} \qquad Z = \frac{k_2 Dl_m F_{em}}{(1+2R_e)R_d}$$

$$\mathrm{III_s} \qquad Z = \frac{k_3 Dl_s F_{em}}{(1+2R_e)R_d}$$

$$\mathrm{IV} \qquad Z = \frac{D^2}{R_d}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$$

An alternative set of equations was used for the four failure methods for double shear connections.

$$\mathrm{I_m} \qquad Z = \frac{Dl_m F_{em}}{R_d}$$

$$\mathrm{I_s} \qquad Z = \frac{2Dl_s F_{es}}{R_d}$$

$$\mathrm{III_s} \qquad Z = \frac{2k_3 Dl_s F_{em}}{(2+R_e)R_d}$$

$$\mathrm{IV} \qquad Z = \frac{2D^2}{R_d}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$$

In either single or double shear, the failure method with the least yield limit was the governing condition. This least value was the multiplied by each of the adjustment factors shown in Table 11. These modification factors applied as they were for the structural elements in the bridge. Several of these factors were the same but the remainder required further explanation.

Group action factor (C_g): This factor "accounts for the non-uniform loading of fasteners" in a line (Breyer, Fridley, Cobeen, & Pollock, 2007). According to Breyer, bolts in a row do not all carry the same loading, so the connections must be designed for the bolt carrying the greatest load (Breyer, Fridley, Cobeen, & Pollock, 2007).

Geometry factor (C_Δ): This factor reduces the yield limit of connections when fastened too close to the end of a member. Connections may be torn out of place in shear when the end distance is too low.

End grain factor (C_{eg}): This factor reduces the yield limit of the connection when a dowel is driven through a member into the end grain of another member. The factor applies when the dowel is driven parallel to the grain of the side member.

Toe-nail factor (C_{tn}): This factor reduces the design strength of connections when a toe-nail is applied. These are "nails that are driven at an angle of 30 degrees to the side member" and into an adjacent main member (Breyer, 2007, p. 12.27).

The remaining factors C_m, C_t, and ϕK_f were treated the same as they were for wood member design. Finally, the factors were multiplied by the minimum Z value for each connection to find the capacity of each connection.

3.1.6g Calculation of connection withdrawal values

Withdrawal values were less involving to calculate than the yield limits of the connections. Values for each connection were tabulated based on the dowel diameter and specific gravity of the wood in Tables 11.2A and 11.2C of the NDS. These values were provided in pounds per inch so they had to be multiplied by the nail length to find the unmodified withdrawal load. This load was finally multiplied by the applicable adjustment factors for to find W'.

3.1.6h Verification of adequate capacity

Finally, the load on each connection was identified to ensure that it would not fail. The strength values were first multiplied by the number of dowels at the joint. The number of dowels on a given plate was predetermined by the catalog of connections while the number of nails was recommended by the IBC schedule (International Code Council, 2014). The modified yield stress Z' was compared to the shear stress acting on the connection, while the modified withdrawal strength *W'* was compared to the axial load on the main member of the joint. Only the largest stresses acting on each type of connection needed to be checked as the number of connections throughout the bridge was uniform for ease of construction. If the loads did not exceed the cumulative yield and withdrawal strengths, the connections were acceptable. This would complete the design of the connections for the bridge structure but the process would need to be completed again for the railings.

3.1.7 Railings

The next step was to design and assess the railing. The first step in the design process for the railings was to gather opinions about whether the railings should more closely resemble the horizontal railing structure as in the Elm Park Red Wooden Footbridge, or the X-style structure of the original 1970's design. From the earlier meetings with the City Manager's office, the Parks and Recreations department, and the Worcester Technical High School, all were in agreement that the X-style railings were more aesthetically appealing for the park. With this decision made, the design was based off of the previous bridge, and used the International Building Code (IBC) to design the members to perform above code.

As mentioned previously, the major driving factor of this bridge was that it would be accessible to all patrons, and thus meet Americans with Disabilities Act (ADA) and Architectural Access Board (AAB) standards. An additional factor in the design is that according to the IBC, the railings for the bridge are considered guards: as stated in section 1013.1, the bridge is "located more than 30 inches (762 mm) measured vertically to the floor or grade below at any point" (International Code Council, 2014). The IBC is a much more stringent code but it also deviates slightly from the ADA, so one of the challenges for this design was to make sure that both are adequately followed.

The major design considerations for the ADA were the railing height requirements. According to the Americans with Disabilities Act, Section 505, the height of the top handrail must be 34 to 38 inches tall, with a diameter of 1¼ to 2 inches. Additionally, the railing must extend 12 inches beyond the bottom of the ramp (U. S. Department of Justice, 2010). An additional accessibility standard from the Architectural Access Board, Section 24 is that there must also be an intermediate continuous railing at a height of 18 to 20 inches (Massachusetts Office of Public Safety, 2006).

From the International Building Code, the following design considerations were also met. One final height requirement from the IBC section 1013 is that the guards must have an overall eight of 42 inches above the walking surface (International Code Council, 2014). This code lightly differs from the ADA requirement of a maximum handrail height of 38 inches, but both can still be implemented. To do this the full height of the railing were designed to the IBC height of 42 inches with aluminum rails attached to the inside of the whole X-style railing structure at heights of 18 inches and 36 inches.

Due to the large openings in the X-style railing design, this alone would not meet the International Building Codes requirement that the openings between horizontal or vertical supports must be small enough such that a 4-inch diameter sphere cannot pass through (International Code Council, 2014). The X-style railings alone would allow a diameter of up to 15 inches to pass through, and thus additional members were added to rectify this lack of code compliance. To do this, two approaches were followed. The first approach assumed that all intermediate members would be made of the same wooden material as the structural X's and would be attached intermittently throughout the X-style structure Figure 33. The second approach assumed that the X-style railing would remain a simple X with a single horizontal bracing, and an additional internal aluminum railing would be attached to the external wooden X-style railing Figure 34. For both approaches, the IBC stipulates an allowable load.

Figure 33: Railing design option 1: Wooden X-style structure with intermittent wooden members.

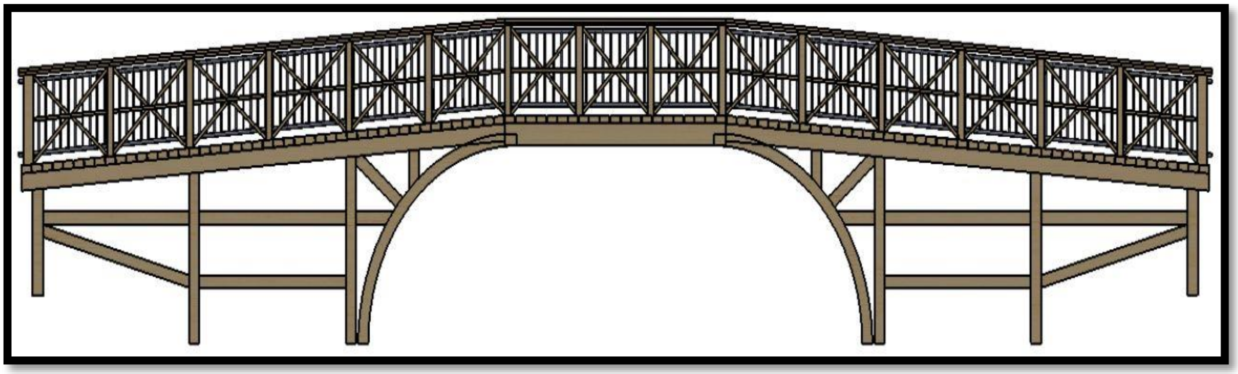


Figure 34: Railing design option 2: Wooden X-style structure with aluminum internal railing.

For both approaches, the support structure is assumed to be the backbone structure of the simple X's, and the top handrail and the mid horizontal support. From the IBC, the maximum loading that this structure must be able to withstand is any and all of the following. As stated in section 16.07.7.1 the guards must be able to hold a uniform live load of 50 pounds per linear foot in the horizontal and vertical directions along the top of the railing; and it must also be able to withstand a concentrated load of 200 pounds at any point along the top of the railing in either the horizontal or vertical direction (International Code Council, 2014). These loads however are not expected to be acting concurrently. Thus, the railings were designed once for each of the four loading scenarios. All loading scenarios were calculated by hand and by using the STAAD software.

3.1.7a Railing Calculations

The procedure for calculating stresses using STAAD for the railing design is similar to the procedure for obtaining stresses for the bridge structure design. The first step was to model the structure in the finite element analysis software, STAAD. There are two ways to model in STAAD, through manual placement of nodes, or adding them to the input command file. For the railing design, because the structure is relatively simple and follows a pattern. Each individual node location was added to the STAAD input command file and then members were created by manually connecting nodes to each other. The nodes connecting the railing to the bridge were assigned fixed connections, as this most closely resembles the actual connections to the bridge. With the members created the member sizes were added based on the preliminary hand calculation information.

Unlike the structural analysis modeling, all loads were given. From the loads given, six different load scenarios were included. One load was the 50 pound per linear foot horizontal load, another 50 pound per linear foot vertical load, and four separate point loads. Two horizontal point loads of 200 pounds at the mid-span of the hand rail and at the column, and two vertical loads of 200 pounds at the mid-span and the column. After the loads were applied and the members were assigned the analysis was run and each load scenario was evaluated to find the loading that would cause the greatest stresses to the bridge members.

3.1.7b Spot Checks

For all scenarios the bending stress, deflection, and shear stress on the top railing were calculated. Additionally the compressive stress, or bending, shear and deflection depending on scenario, was calculated for the columns and diagonals. The following tables show the equations for all four scenarios for the handrail calculations. For all instances involving P or w_{max} $P \text{ or } w_{max}$ the values for these are 200lb and 50lb/ft respectively.

Table 12: Bending equations for railing spot checks.

Bending Stress on Handrail	Uniform Horizontal Load	Uniform Vertical Load	Point Horizontal Load at Center of span	Point Vertical Load at Center of span
$\boldsymbol{F_b}$	M_{max}/S	M_{max}/S	M_{max}/S	M_{max}/S
$\boldsymbol{S}$	$\frac{db^2}{6}$	$\frac{bd^2}{6}$	$\frac{db^2}{6}$	$\frac{bd^2}{6}$
$\boldsymbol{M_{max}}$	$\frac{w_{max} l^2}{8}$	$\frac{w_{max} l^2}{8}$	$\frac{Pl}{4}$	$\frac{Pl}{4}$

Table 13: Shear equations for railing spot check.

Shear Stress on Handrail	Uniform Horizontal Load	Uniform Vertical Load	Point Horizontal Load at Center of span	Point Vertical Load at Center of span
$\boldsymbol{F_v}$	$\frac{3V_{max}}{2A}$	$\frac{3V_{max}}{2A}$	$\frac{3V_{max}}{2A}$	$\frac{3V_{max}}{2A}$
$\boldsymbol{V_{max}}$	$\frac{1}{2} w_{max} l$	$\frac{1}{2} w_{max} l$	$\frac{P}{2}$	$\frac{P}{2}$

Table 14: Deflection equations for railing spot check.

Deflection on Handrail	Uniform Horizontal Load	Uniform Vertical Load	Point Horizontal Load at Center of span	Point Vertical Load at Center of span
Δ	$\frac{5w_{max}l^4}{384EI}$	$\frac{5w_{max}l^4}{384EI}$	$\frac{Pl^3}{48EI}$	$\frac{Pl^3}{48EI}$
I	$\frac{1}{12}db^3$	$\frac{1}{12}bd^3$	$\frac{1}{12}db^3$	$\frac{1}{12}bd^3$

Depending on the scenario, the columns and diagonal X braces were either being subjected to a vertical compressive force, or a horizontal force. This horizontal force at the top of the column most closely resembles a cantilever scenario for the column, and would thus cause bending stresses and shear stresses as opposed to the compressive stresses from the vertical loading. The equations for shear, bending, and deflection follow the same equations as above with the exceptions of the maximum shear force and maximum moment as shown in the table below. For example with the handrail the maximum loading would occur if the point load was concentrated at the middle of the span; but for the columns, the maximum loads are obtained if these point loads are concentrated at the columns. Also shown are the equations that were used for compressive stresses.

Table 15: Column force equations for railing spot checks.

Forces on columns and diagonals	Uniform Horizontal Load	Uniform Vertical Load	Point Horizontal Load at Post	Point Vertical Load at Post
M_{max}	Vl	–	Pl	–
V_{max}	wl	wl	P	P
F_c	–	$\frac{V_{max}}{A}$	–	$\frac{V_{max}}{A}$
F_b	$\frac{M}{S}$	–	$\frac{M}{S}$	–
F_v	$\frac{3V}{2A}$	–	$\frac{3V}{2A}$	–

The second part of the railing design was to take the X-style railings and modify them so that they would meet the IBC code for intermediate openings. The two options for this were to add wooden posts throughout the X's or to attach an inner aluminum railing. To add the additional wooden members, the procedure was quite simple. Since it was assumed that the additional wood added minimal strength value to the X they were considered an architectural addition. To add these, pieces measuring four inch in lengths were added. To account for the possible strength loss from the additional nails in the wood, the NDS allowable strength design factors were modified. This modification decreased the design strength slightly.

The other option for filling the openings, the aluminum internal railing, can be manufactured two ways. After meeting with the Worcester Technical High School, it was concluded that the WTHS would be able to fabricate the aluminum railings in their shop. However, as they are not being created in ideal situations, the strength design might not be met. Additionally, this would be the school's first project of this type, and could add additional time to the project. The second procedure for obtaining this railing was by going through a reputable supplier. The benefit of this is that the supplier would guarantee and provide a copy of the strength properties of option the railing. Though getting it from a supplier would cause an additional cost, it would also have a greater longevity guarantee.

3.2 Foundation Bearing Capacity Analysis

There were many external variables that had to be considered regarding the existing footings in Elm Park before any bearing capacity analysis could be completed. It is important to note that these footings are relatively massive for the structure that they support. Furthermore, the existing concrete footings were poured in 1972 and no major degradation, cracking or settlement is currently visible. Therefore, it is reasonable to assume, that the foundation system currently in place will sustain the loads associated with the new bridge and that the bearing capacity will be adequate.

The exact site conditions and plans for when the foundations were poured are unavailable. As a result, it is uncertain how deep these footings are poured below the surface, or if the existing soil was excavated in 1972 and replaced with a better quality soil, or even if there are underlying support piles that sustains the footings given the extreme variability of the soil profiles adjacent to the footings.

In addition to the external variables pertinent to the bearing capacity evaluation, there were also multiple ways in which these footings could be studied, some more conservative than others. One way to consider would be evaluating the entire footing as one solid piece, with no voids as there are now. This would simplify the required calculations in that the footings would be considered a concrete mat, however they would not be conservative, as the total load imparted by the massive mat would be inevitably higher due to the increased weight of the concrete material. Another option would be to assume the existence of piles underneath the footings, and proceed with design calculations in a similar manner. However, this option seems very sophisticated in terms of practicality and constructability for such a small-scale project, especially in 1972, and therefore it is reasonable to assume that piles were not used as part of the footing structures.

A third option would be to consider the foundation as three individual strips with the intermediate perpendicular strips not contributing to the structures allowable bearing capacity. Since each strip supports similar column loads subjected by the bridge at exactly the same points along the span, a resultant pressure distribution can be calculated and thus compared to the bearing capacity of the underlying soil. In this manner, a more accurate estimation of the pressure exerted on the soil by each strip can be achieved, while still maintaining conservative values with adequate factors of safety given the existing site and footing conditions. It is this method that was utilized to evaluate the bearing capacity of the existing footings in Elm Park.

3.2.1 Subsurface Investigation

A good understanding of the soil conditions around the foundation is necessary when performing analysis and design investigation for bearing capacity and settlement. Boring logs drilled in proximity to the bridge provided the necessary subsurface information about the soil strata, the type of soil and ground water levels necessary for footing assessment and analysis.

In order to obtain the subsurface information that was needed from the site, a site map with proposed boring locations was provided to The City of Worcester Department of Parks and Recreation. This diagram listed ten boring locations with five on each side of the bridge in locations that would be critical to the foundation, abutment, and the pond edge wall. Figure 35 below shows the map with the boring site locations that was provided to the City. The City then hired the firm Soil Exploration Corporation of Leominster, MA, to complete the subsurface investigation.

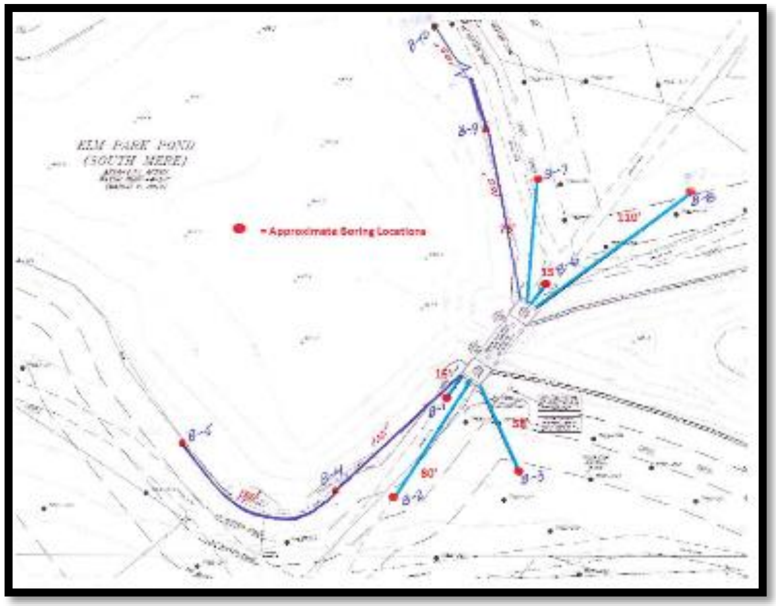


Figure 35: Site map with bore-hole locations in Elm Park.

On January 9th, 2014 the soil exploration took place at Elm Park. The boring logs generated by Soil Exploration Corporation were then provided in order to extract the information that was needed for future calculations. It was found that the maximum boring depth reported was at 27 ft. below the surface. The complete boring logs can be seen in Appendix B: Soil Boring Logs.

When looking specifically at what soils would be acting on and around the footings, Boring #1 on the Russell Street side of the bridge and Boring #6 on the Park Avenue sides were considered. To better understand these two borings the first step was to create a soil profile so different layers of soil could be highlighted. Using Microsoft Excel, blocks were made with certain heights to represent the depth of the soil strata from the log. From this we were able to see the profile in detail. These tables can be seen in the results section.

Next, the unit weight of the material in each stratum needed to be determined. This was needed because a complete analysis of the soil was not done when the borings were taken. To do this, the column on the field boring log labeled "*Blow/6 in.*" was assessed. This column provided the number of blows it took to drive down 6 in. in a 2-ft section. Each set had four numbers; the first number and the last number are disregarded leaving only the middle two. When these numbers are added together, an 'N-value' is generated. This value is important because there is a direct correlation between the N-value of a particular soil and its corresponding properties. The logs provided a key at the bottom of each page that helped with this process by providing apparent density for cohesionless or course materials like sand, and consistency of cohesive or fine materials like clay, based on the N-value. These N-Values were then added to the table generated, as seen in the results section. After confirming the results of the log, the Unified Soil Classification Chart was consulted (ASTM, 2014). This chart helped generate a more specific group name for the different soil strata from the logs, which were needed to identify the unit weight. To classify each stratum, it was important to first look at the layer and determine if it was a course, fine, or organic soil. Depending on the outcome, it would correspond to another classification, as shown in Table 16 below further defining the materials' group symbol and name. Furthermore, the physical characteristics of the soil strata listed in the boring logs helped to derive specific engineering properties. These processes can be seen in chapter 4 of *Geotechnical Engineering Principles and Practices* (Coduto, Yeung, & Kitch, Geotechnical Engineering Principles and Practices, 2011).

Major Divisions				Typical Names
Course-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of course fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

Table 16: Unified Soil Classification (USC) System (from ASTM D 2487)

Once this was completed, Table 4.1 from *Geotechnical Engineering Principles and Practices* was utilized, which gave an estimate for the unit weight of the material based on the classification. (Coduto, Yeung, & Kitch, Geotechnical Engineering Principles and Practices, 2011). These values were added to the soil profile that was generated for future uses.

Next, the effective cohesion factor and angle of internal friction for each soil stratum needed to be determined, because that too was unknown due to an incomplete analysis of the borings. These values were derived from tables provided in *Foundation Design: Principles and Practices* chapter 3 (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001). The tables were organized by group symbols listed by the USC. The table for internal friction angles listed a minimum and maximum angle; the maximum value was selected in order to be more conservative. From this data a soil profile was created and utilized for conducting the foundation bearing capacity analysis.

3.2.2 Initial Assumptions

There were some initial assumptions that needed to be made before continuing with the bearing capacity calculations for the existing footings. These are all reasonable assumptions given past and current best practice construction methods and existing site conditions:

- Assume three separate combined, rectangular, strip footings connected with perpendicular interconnection pieces that do not contribute to structural support or bearing capacity values.
- Assume a total depth of six feet from the 492' elevation mark.
- Assume that peat and clay material below footings was excavated in 1972 and replaced with suitable bearing soil, as these soil types are not suitable for sustaining structural loads.
- Assume three feet of footing above pond floor.
- Use column loads to calculate resultant force on strip derived from STAAD analysis design values for bridge

In addition to these assumptions, some other parameters for both the underlying soil conditions and the footing strips needed to be established before any calculations could be completed. The underlying soil was assumed to be a medium-dense granular fill.

Table 17 summarizes these assumed parameters, again based on best practice values and known conditions:

Table 17: Foundation Assumed Parameters

B (Width of Footing Strip)	2ft
L (Length of Footing Strip)	17.625ft
γ_w(Water Unit Weight)	62.4 lb/ft^3
Ground Water Table Depth	0 Feet (Pond)
c' (Effective Cohesion of Soil)	0
ϕ'(Angle of Internal Friction of Soil)	35°
γ (Unit Weight of Soil)	120 lb/ft^3
Vesic's Method	*Consider only shape, depth, bearing capacity factors

Having established all relevant initial assumptions and parameters, it was then necessary to evaluate the pressure exerted by the three individual strips that compose the footings in Elm Park.

3.2.3 *Determine the Pressure Imposed by Footing Strips*

In order to properly evaluate the pressure distribution at the bottom of the footing strips, it was first necessary to determine the loads subjected by the structural columns of the bridge at the different points along the spans of the strips. These column loads contribute to the total bearing pressure subjected by the footing strips on the soil, and thus are an extremely important aspect of the bearing capacity analysis.

A model of the new bridge to be constructed in Elm Park was previously created in a structural analysis program called STAAD. From this program, resultant loads at the bottom of the structural support columns and arches could be acquired and utilized as a basis for determining the overall resultant load on each strip. For purposes of simplicity; the load imposed by the arch on each strip and the column directly adjacent to it were considered as one combined load. Ultimately, three separate loads were determined at the exact same locations on each of the three strips. Knowing the magnitude and location of these loads, a resultant force at some distance along the span of the strips could be calculated by summing the total moment around the front edge of the strips, as shown in Appendix I: Bearing Capacity Hand Calculations. It was important to confirm that this resultant load acted in the middle third of the strip, as this would ensure that compression forces acted continuously throughout the span of the footing strips. Furthermore, it is the safest and most ideal situation for strip foundations (Nilson, Darwin, & Dolan, 2010).

After the resultant loads were determined for each separate strip, the weight of the footing strips themselves had to also be considered regarding the total pressure exerted by the footings on the underlying soil. It was assumed that the concrete was of normal weight, approximately 150lb/ft^3. Having already measured the exact dimensions of the strips of the footing in the field, it was then simple to determine the total weight of each strip.

Once the resultant loads and loads from the weight of the footing strips were established, the total pressure distribution at the bottom of each footing could be calculated. This was accomplished by adding the resultant forces and weight of the strips together, and dividing by the square areas of the footings themselves, as seen in Appendix I: Bearing Capacity Hand Calculations. Since strips 1 and 3 mirrored the column loads they supported, their corresponding pressure distributions were mirrored as well. These final pressure distributions, which were mirrored across the pond for the opposite footing, could then be compared to the bearing capacity of the soil to ensure that they were less than the allowable bearing capacity for the assumed soil type.

3.2.4 Determine the Bearing Capacity of the Underlying Soil

As established previously, it was assumed for the purpose of the bearing capacity calculations that the unsuitable material found adjacent to the footing sites was excavated beneath the footings and replaced with a more suitable soil material capable of withstanding the loads subjected by the footing and the bridge itself. With this in mind, the bearing capacity calculations were completed in accordance with the methods outlined in *Foundation Design: Principles and Practices, Second Edition* by Daniel Coduto. Vesic's method was employed due to its ability to provide more accurate bearing capacity values and apply to a wider range of geometric and loading conditions. Per Vesic's method and given the assumed soil conditions mentioned previously, the ultimate bearing capacity could be calculated using the equation:

$$q_{ult} = \sigma'_{zd} N_q s_q d_q + 0.5 B \gamma' N_\gamma s_\gamma d_\gamma$$

where σ'_{zd} is the vertical effective stress at the assumed depth, B is the width of the footing, γ' is the effective unit weight dependent on the depth of the ground water table, and the *N*, *s*, and *d* factors are the corresponding bearing capacity, shape, and depth values, respectively. Approximate values for these parameters were derived from the equations outlined on pages 184-186 in *Foundation Design: Principles and Practices*.

Once an ultimate bearing capacity was calculated, a factor of safety was then incorporated to determine an allowable bearing capacity. The pressure distribution from the loads of the bridge and the weight of the footing should not exceed this allowable bearing capacity value. Many factors should be considered when determining an adequate factor of safety, including soil type, soil variability, and importance of the structure and resulting consequences from a failure. Using engineer's discretion, a factor of safety of 2 was determined for the bearing capacity calculations.

The allowable bearing capacity was compared to the resultant distribution force for all three strips to ensure that it exceeded this resultant force enough to be considered adequate. If this was the case, then the soil was considered able to withstand the structural loads imparted by the footings and the new bridge and the bearing capacity check would be satisfied.

3.3 Abutment Design

In order to design the concrete abutments that were to be placed on the foundations in Elm Park for the new bridge project, multiple phases of stability checks needed to be utilized. As mentioned previously, when designing any type of concrete earth retaining system, both the external stability and the internal stability of the system need to be designed and assessed for an adequate factor of safety. Typically, an engineer would address the external stability of a system first, in the form of overturning, sliding, and bearing capacity of the new wall. After these factors of safety are confirmed, the engineer would then check the internal stability to ensure that required reinforcement ratios are met within the concrete wall so that enough resistance is provided for the pre-determined factored design loads. The new concrete abutments would then act as “cantilever” retaining walls in the context of the Myra Hiatt Kraft Footbridge Project, and so the methodology to design these abutments incorporated the same process as one would use for the design of a cantilever retaining wall. First, the external stability was checked for factors of safety against overturning, sliding, and bearing capacity, and then internal stability was checked against the required design loads. The following sections outline the process that was used for the abutment design.

3.3.1 Preliminary Design Considerations

There were multiple assumptions and considerations that needed to be made before the external stability check could take place. First and foremost, the methods being used to assess the stability of the cantilever retaining wall followed the guidelines set forth by the IBC. These guidelines can be found in IBC 1610.1 and IBC 1807.2.

Table 18 summarizes the important aspects of the code that needed to be taken into consideration:

Table 18: IBC Guidelines for Cantilever Retaining Wall Designs

Necessary Checks	Overturning, Sliding, Foundation Bearing Pressure
Design Pressure	Active Pressure: 30 psf/foot of depth
Factor of Safety for Design Checks	1.5
Backfill Soil Quality	Well-graded, Clean gravel, gravel-sand mix
Snow Load Assumption	Massachusetts: 55 psf
Live Load Assumption	Public Spaces serving People: 100 psf

In addition to the standards from the IBC that needed to be met, the Massachusetts Department of Transportation Highway Division also provides Construction Standard Details that the City of Worcester Parks Department frequently references in their projects. In this construction standard, assumptions are provided for cantilever retaining wall design regarding the backfill soil properties as well as the surcharge pressure, in this case pressure on the landing above the wall in Elm Park, as well as necessary reinforcement ratios for the wall itself. Although this standard is typically applied for retaining walls of much larger magnitude and scale, the same values can also be applied for the purposes of assessing the external stability of the cantilever retaining wall in Elm Park, as the situations are similar in nature. Table 19 summarizes these values found in the MassDOT Construction Standard Detail Drawing 305.3.0: Cantilever Retaining Walls, Dense Foundation Soils, Level Backfill, and Surcharge Pressure:

Table 19: Mass DOT Construction Standard Detail Design Assumptions

Backfill Soil Type	Gravel Borrow for backfilling structures and pipes
Angle of Internal Friction (ϕ)	37°
Effective Unit Weight of Soil (γ)	120 pcf
Factored Bearing Capacity	9400 psf
Surcharge Pressure	240 psf
Friction Factor	.57
Concrete Strength (f'_c)	4000 psi
Steel Strength (F_y)	60,000 psi
Concrete Type	Normal Weight, 150 pcf

After the initial design criteria were summarized, it was also important that the dimensions of the wall itself were established, including the height, length and thickness of the wall. The wall length was easiest to assume. The current widths of the foundations in Elm Park are 13.0625-ft. The wall was to be placed directly on top of this foundation, and therefore would need to be at least the same length as the foundation itself. The height of the wall was calculated based on the height of the Myra Hiatt Kraft Footbridge design above the foundation. Using the Autodesk design programs, a model was created of the new bridge on the foundation. This model, which can be seen in Figure 36 below, also incorporates a foundation design that does not include the existing bridge abutment. As mentioned in earlier chapters, the Elm Park Red Wooden Footbridge had its own additional abutment added onto the foundation when it was constructed. It was assumed for the purposes of this design that the existing abutment would be removed from the foundation and a level surface as indicated in the image would be left in its absence.

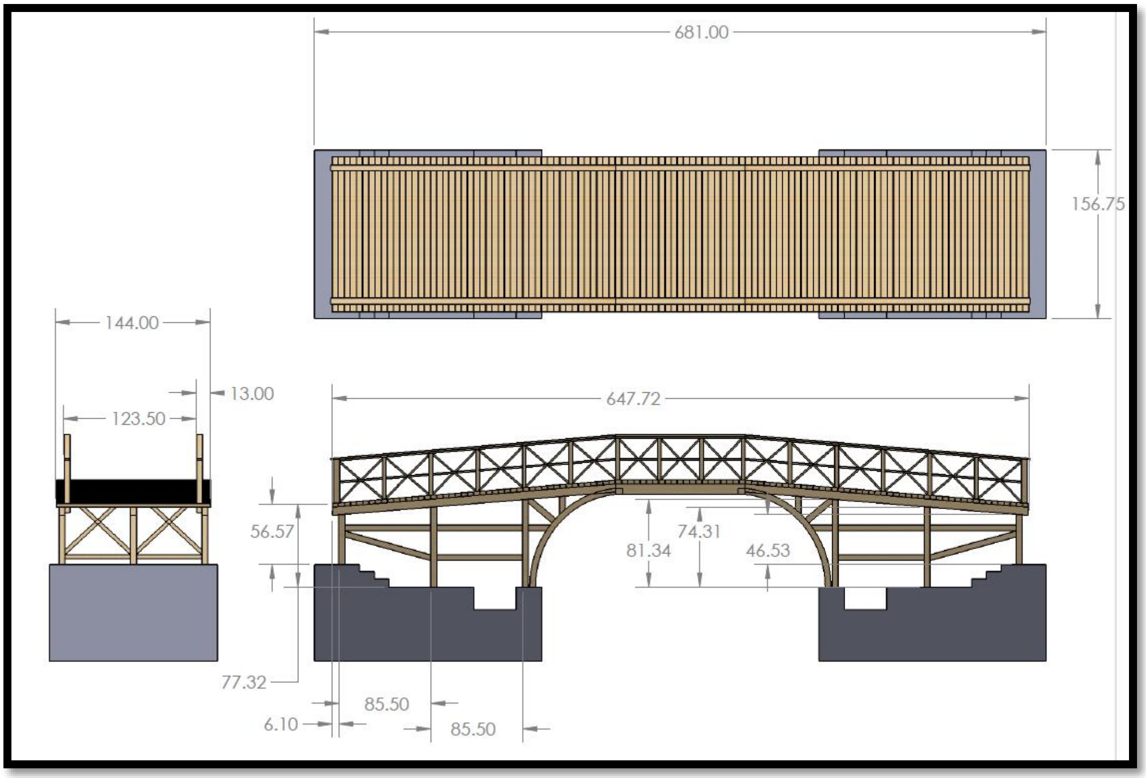


Figure 36: CAD design of the Myra Hiatt Kraft Footbridge

As can be seen in Figure 36, the height from the top of the foundation where the wall will be placed to the top of the bridge decking is about 56.57 inches or 4.71 feet. The minimum thickness of the wall was determined based on best practices for concrete design. The wall itself was considered a one-way slab for the purposes of design. Additionally, the wall was much shorter than typical retaining walls, so deflection of the wall was not as much of an issue as it would be in larger scale projects. However, in order to avoid deflection calculations for the wall, a minimum thickness of *L*/10 for cantilever slabs is required per table 6.1 of *Design of Concrete Structures* which was adopted from ACI Code 9.5.2, in which *L* refers to the height of the wall. As a result, a minimum thickness of wall was assumed to be 6 inches. Table 20 summarizes the dimensions required for the cantilever retaining wall:

Table 20: Cantilever Retaining Wall Dimensions

Height	56.57 in.
Length	156.75 in.
Thickness	6 in.

3.3.2 External Stability Check

In addition to the preliminary design considerations, external stability checks needed to be completed. These checks included the overturning stresses, the bearing pressure, and the sliding forces. The abutment design needed to be designed such that all design loads were within the allowable design strengths with an acceptable factor of safety.

3.3.2a Overturning Design Considerations

The first step in determining whether or not there was an adequate factor of safety to resist overturning pressure by the wall was to calculate the location and magnitude of the resultant force from the lateral pressure of the backfill exerted on the wall. This resultant force was a combination of both the lateral force of the backfill soil as well as the surcharge pressure on the landing above the wall. Once an adequate surcharge pressure was determined above the soil, it was then converted to an equivalent height of the soil below it, and ultimately added to the force distribution on the wall from the backfill.

As mandated by the IBC, the calculations considered active earth pressure from the backfill soil. Therefore, the coefficient for active pressure, K_{ah}, was calculated using the formula:

$$K_{ah} = \frac{1 - \sin\phi}{1 + \sin\phi}$$

Next, the total surcharge pressure was calculated. This was done by summing three different loads, the dead load from the slab that would serve as the required landing at the bottom of the bridge wing, the live load on the landing considering the total area filled to maximum capacity, and the snow load in Worcester, as given by the IBC. Summing these loads produced a total load of 225 psf, which is relatively close to the previously mentioned assumed surcharge pressure given by the MassDOT Construction Standard of 240 psf. In order to be conservative, a surcharge pressure of 240 psf was assumed. To convert this surcharge pressure to an equivalent height of soil, the following formula was used:

$$h' = \frac{s}{w}$$

where *s* equals the maximum surcharge pressure and *w* equals the unit weight of the soil.

Next, the location of the resultant force from the bottom of the wall and the magnitude of the resultant force were calculated using the following formulas.

$$y = \frac{h^2 + 3hh'}{3(h + 2h')}$$

$$P = \frac{1}{2} K_{ah} wh(h + 2h')$$

Once both of the above values were calculated, an ultimate overturning moment caused by the wall itself could be determined as $P * y$.

In order to identify if the resulting overturning moment was detrimental to the foundation stability, the weight of the foundation and its total restoring moment were calculated by dividing the foundation into separate components and calculating those individual weights and restoring moments about the front toe of the foundation. To do this, a depth of 7ft from the 492ft elevation mark for the foundation was assumed. To be conservative it was also assumed that the voids in the foundation filled with soil had the same unit weight as the concrete itself.

After a total restoring moment was determined, the location of the resultant from the front edge of the foundation was determined using the formula:

$$a = \frac{M_r - M_o}{Total\ Weight}$$

where M_r and M_o are the restoring and overturning moments respectively. The location of the resultant needed to be calculated to establish if it was located within the middle third of the foundation length. If this were the case, then compression would act throughout the length of a foundation section, which was ideal for overturning and bearing pressure considerations. Furthermore, a factor of safety against overturning was calculated.

3.3.2b Bearing Pressure Consideration

Upon calculating the location of the resultant restoring force on the foundation, a maximum bearing pressure felt under the footing could be calculated by adapting the equations in Figure 17.5a of *Design of Concrete Structures*. This maximum pressure should not have exceeded the permissible or allowable bearing pressure of the soil, or settlement and differential settlement can occur, which could be a detriment to the bridge itself.

As mentioned previously, the resultant was calculated to be located within the middle third of the foundation. This implied that compression forces would act throughout the foundation along the bottom of the foundation. It is standard practice to have the resultant located within the middle third of the foundation, as it not only reduces the magnitude of the bearing pressure but also prevents too large of a non-uniform pressure. Since this was the case for the Elm Park foundations, the minimum and maximum bearing pressures were calculated using the following equations:

$$q_1 = (4L - 6a)\left(\frac{R_v}{L^2}\right)$$

$$q_2 = (6a - 2L)\left(\frac{R_v}{L^2}\right)$$

where L is the length of foundation, a is the location of the resultant, and R_v is total weight of the foundation section. From these equations, a maximum bearing pressure was obtained and compared to the allowable bearing pressure by determining a factor of safety against this hazard.

3.3.2c Sliding Consideration

Sliding or bodily displacement was another concern of the design. It is known in foundation design that a wall, such as a cantilever wall, which rests on a base slab or foundation, may be bodily displaced by the earth thrust *P* that acts on the vertical wall plane and in turn slides along a perpendicular horizontal plane. This sliding is resisted by the friction between the soil and the footing along the same plane. In order to prevent sliding from occurring, the forces that resist sliding must exceed those that produce sliding by an adequate factor of safety.

The horizontal force contributing to sliding was calculated previously in the overturning design check by determining the magnitude *P* of the lateral soil forces acting on the back of the wall. This was checked against the corresponding frictional force, which is a result of the total weight of the foundation multiplied by an appropriate friction factor,

$$F_f = f * R_v$$

where *f* is assumed in this case to be 0.57 based on the soil and foundation conditions. A factor of safety between the two forces was calculated to ensure external integrity.

3.3.3 Internal Stability Check

After ensuring that all external stability checks were satisfied, the second step in the abutment design process was to evaluate the internal stability of the new retaining wall to ensure that the specified internal reinforcement could resist the proper moment and shear forces acting on the wall. The following section outlines the process that was adapted for the Elm Park retaining wall internal design.

In order to begin internal reinforcement calculations, some initial parameters first had to be established on which to base the design calculations. Some of these included assuming strength of steel and concrete, as well as dimensions of the wall. Table 21 below summarizes the initial parameters needed to complete the calculations:

Table 21: Initial parameters for abutment design calculations

Parameter	Value
f'_c (strength of concrete)	4000 psi
f_y (strength of steel)	60,000 psi
Height of wall	56.57in
Thickness of wall	6in
Unit Length of Wall	12in
ϕ_T (strength reduction factor, tension)	0.90

3.3.3a Moment and Shear Acting on the Wall

The second step for the internal design portion of the wall was to check the factored load combinations for both the shear and moment forces acting on the wall, to ensure they yielded an adequate factor of safety. The structural design of a retaining wall should be consistent with methods used for all types of members, and therefore should be based on factored loads in recognition of the possibility of an increase above service loading (Coduto, Foundation Design: Principles and Practices 2nd Edition, 2001).

Following ACI Code, lateral earth pressures were multiplied by a load factor of 1.6. In general, the reactive pressure of the soil under the structure at the factored load stage is equal to 1.6 times the soil pressure found for service load conditions in the external stability analysis (Nilson, Darwin, & Dolan, 2010). For this reason and to be conservative, dead, live, and soil loads were multiplied by the highest factor of 1.6 in order to determine the resulting moment and shear values.

The maximum moment acting on the wall was determined in a similar manner to the maximum soil pressure calculations. However, a maximum moment was instead determined at the very bottom of the wall. The shear analysis was completed in the same way, calculating the maximum factored shear force at a distance d above the base, and then checking it against the allowable shear value, which was determined using the equation:

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}bd$$

where $\phi = .75$ for shear. After confirming both values, reinforcement calculations could begin.

3.3.3b Required Vertical and Horizontal Reinforcement

After the moment and shear values acting on the wall were determined, ACI code was adapted to compute the minimum and maximum steel reinforcement values for the given wall.

First, an effective depth of the wall was calculated using the formula d=h-2.5 in. This is a reasonable assumption considering the diameter of the bars that would be used and that a mandatory 1.5 inch cover is needed everywhere in the wall because it is exposed to ground, according to ACI Code 7.7. After establishing an effective depth for a unit length of the wall, the required reinforcement for the wall was obtained using the formula:

$$\phi M_n = \phi\rho f_y b d^2 \left(1 - .59\frac{\rho f_y}{f'_c}\right)$$

where ϕM_n was the previously calculated moment acting on the wall. The value obtained for ρ was then compared to the minimum and maximum reinforcement ratios as stipulated by ACI Code per table A.4: *Design of Concrete Structures*:

$$\rho_{min} = 3\frac{\sqrt{f'_c}}{f_y}$$

$$p_{max}(\varepsilon_T = .005) = .0181$$

Once a proper reinforcement ratio value was established, the required vertical reinforcement area of steel for a 12 inch unit length of the wall was calculated using the equation provided by ACI Code:

$$A_s = \rho b d$$

Upon determining an A_s value, adequate reinforcement could be stipulated for the wall and placed accordingly. It was also important to consider that in reinforcement design for the given situation, reinforcement spacing could not exceed 3*h* or 18 inches.

In accordance with ACI code 14.1.2, cantilever retaining walls should be designed following the flexural design provisions specified, with minimum horizontal reinforcement to account for temperature and shrinkage provided in reference to ACI Code 14.3.3. This section stipulates a minimum reinforcement ratio of .0020 for bars not larger than No. 5. As before, a minimum area of steel for a unit length of the wall was required and adequate reinforcing bars were chosen and placed accordingly.

3.3.3c Development Length for Dowels and Hooked Bars

Considering that the designed retaining wall would be attached in the field to the existing foundation, it was necessary to also consider reinforcement that would be embedded in both the new wall and the old foundation. Specifically, a development length for the vertical dowels that would be used to attach the two concrete structures needed to be determined for design purposes.

Development length is the length of embedment necessary to develop the full tensile strength of a reinforcing bar within concrete, controlled by either pullout or splitting. In the case of the Elm Park retaining wall, the development length is the length necessary to embed the vertical dowels that will serve as the connection pieces to existing foundation within the new wall. There are many influences and factors that can affect development length in a given scenario. ACI code provides a simplified equation for determining the development length of vertical bars of size no larger than No. 6, with clear spacing between bars at least $2d_b$, and with clear cover at least d_b, as shown in Equation 5.5 of *Design of Concrete Structures 14th Edition*:

$$l_d = \left(\frac{f_y \psi_t \psi_e}{25\lambda\sqrt{f'_c}}\right) d_b$$

Here, ψ_t, ψ_e are reinforcement location and epoxy coating factors respectively, and the diameter of the bar is equal to the required diameter of the previously specified vertical reinforcement. The values used for these factors were determined based off the information provided in *Design of Concrete Structures 14th Edition*, and can be seen in Appendix D: Reinforced Concrete Design Tables.

In addition to the development length required for the vertical reinforcement, a development length for hooked bars also needed to be determined in accordance with ACI Code 12.5. One of the abutment structure design options would consist of additional wings that require attachment to the retaining wall being designed. This attachment would come in the form of horizontal reinforcement in the wings and it would need to be hooked at the intersection with the perpendicular retaining wall in order to fully develop the required tension forces. The required development length for these hooked bars is given by equation 5.6 in *Design of Concrete Structures, 14th Edition*:

$$l_d = \left(\frac{f_y \psi_e}{\lambda\sqrt{f'_c}}\right) d_b \geq 12 d_b$$

where the diameter of the bar is equal to the diameter of the bar specified for the required horizontal reinforcement for the retaining wall structure.

3.3.4 Attachment to Existing Foundation

The new abutment piece will connect to the existing foundation. This means that existing concrete must connect to new concrete which is less than ideal conditions. In order to ensure a proper connection, deformed reinforcing bars (rebar) will need to extend from the foundation up into the abutment. To do this holes will need to be pre-drilled into the foundation, and a bar along with an adhesive must be injected into the hole to insure it stays in place.

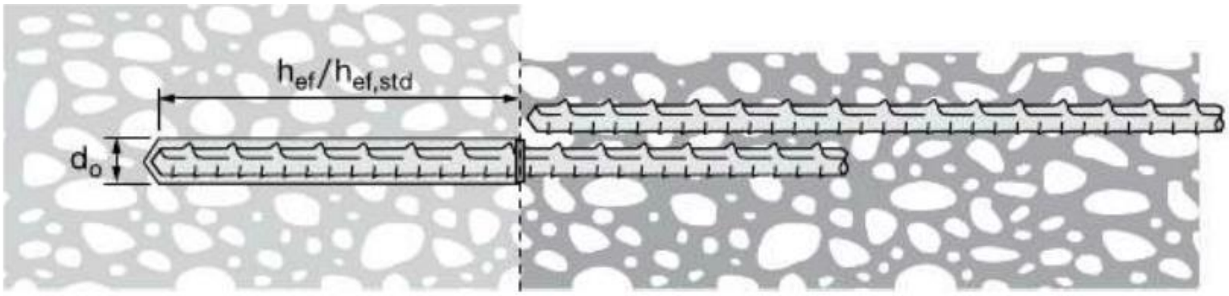


Figure 37: Rebar installed with HIT-HY 200 adhesive (Hilti, n.d., p. 71)

After basic research and suggestions by professional engineers a company that dealt with the products necessary for our anchoring system was discovered. This company, HILTI, has a very user friendly website that used to find the proper adhesive needed for our design. The first step in selecting the proper adhesive was to consult an adhesive summary table that HILTI provided, the table is shown in Appendix J: Hilti Adhesive Chart.

Using the table adhesives were eliminated that were unsuitable for the conditions of the design. All of the adhesives met the size, head type of rebar, and corrosion resistance. When looking at the base material of concrete, and features, the field of possible choices was narrowed down to five possible candidates. The final category used to choose the adhesive type was the hole cleaning option. This was an unknown for our purposes because we did not know how the hole was to be cleaned so we chose the adhesives that fit all three subcategories. This left HIT HY 200-A, and R hybrid adhesives remaining for selection.

Once the adhesive was selected the Hilti Product Technical Guide Vol. 2 Anchor Fastening product was used (Hilti, n.d., p. 71). This guide helped with the technical details of the rebar that would connect the abutment to the foundation. When the abutment was designed the vertical reinforcement was determined to be a No. 5 bar. Knowing this, the tabulated embedment depth for a No. 5 bar was given as a minimum of 3-1/8 inches and a maximum of 12-1/2 inches. Table 22 below shows the values discussed.

Table 22: Rebar installed with HIT-HY 200 adhesive, source: (Hilti, n.d., p. 71)

Setting information		Symbol	Units	Rebar size							
				3	4	5	6	7	8	9	10
Nominal bit diameter		d_o	in.	1/2	5/8	3/4	7/8	1	1-1/8	1-3/8	1-1/2
Standard effective embedment		$h_{ef,std}$	in. (mm)	3-3/8 (86)	4-1/2 (114)	5-5/8 (143)	6-3/4 (171)	7-7/8 (200)	9 (229)	10-1/8 (257)	11-1/4 (286)
Effective embedment	minimum	$h_{ef,min}$	in. (mm)	2-3/8 (60)	2-3/4 (70)	3-1/8 (79)	3-1/2 (89)	3-1/2 (89)	4 (102)	4-1/2 (114)	5 (127)
	maximum	$h_{ef,max}$	in. (mm)	7-1/2 (191)	10 (254)	12-1/2 (318)	15 (381)	17-1/2 (445)	20 (508)	22-1/2 (572)	25 (635)
Minimum concrete member thickness		h_{min}	in.	h_{ef} + 1-1/4		$h_{ef} + 2d_o$					
			(mm)	h_{ef} + 30							

The Hilti website was also helpful in determining the amount and cost of adhesive needed for this project. The site has an anchor volume calculator, which allowed for input of the parameters of a design and then shows how many cartridges of adhesive are needed. The most conservative numbers were used so an embedment depth of 12-1/2 inches, and 20 anchors were used. This accounts for the maximum effective depth of the rebar, and also a 1/3 surplus of adhesive.

3.3.5 Design Options

Considering the information above as well as the realistic implications of the project, two design options were created that were to be evaluated by the relevant stakeholders including the WPI MQP Group, the city representatives for the project, and the Worcester Parks and Recreation Department. The goal was to create an abutment design option that not only satisfied the requirements for the new difference in height between the elevation at the bottom of the new bridge landings and top of the existing foundation, but also was an economical and practical design option that could be constructed easily in the field given the existing conditions. Two design options were created and are outlined below.

3.3.5a Option 1-Solid Block

In terms of construction costs, concrete and steel reinforcement are cheaper materials to work with and thus easier to buy in bulk than other materials such as metals or those that require additional craftsmanship (Nilson, Darwin, & Dolan, 2010). Furthermore, the size of the abutment is small in magnitude, only needing to cover less than a five foot height difference between the new bridge and the existing foundation. Therefore, it is not unreasonable to design an abutment block with a square area equal to that of the required landing at the bottom of the bridge wing and a required depth of about seven feet, which would protect against any frost heaving action by the soil on the foundation.

The abutment design should contain minimum vertical and temperature and shrinkage reinforcement as calculated above in the areas exposed directly to the loads and weather and is to be attached to the existing foundation by means of vertical dowels at the front of foundation, as shown in Figure 38.

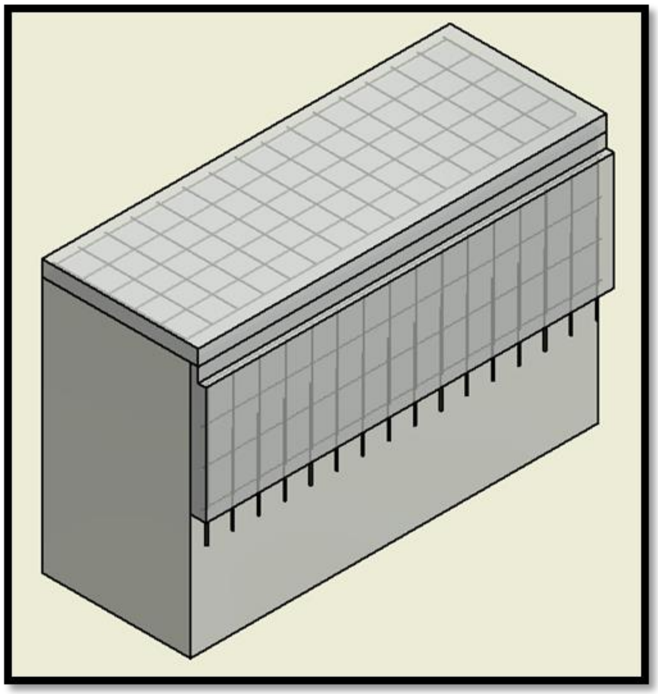


Figure 38: Abutment design option 1 – solid block.

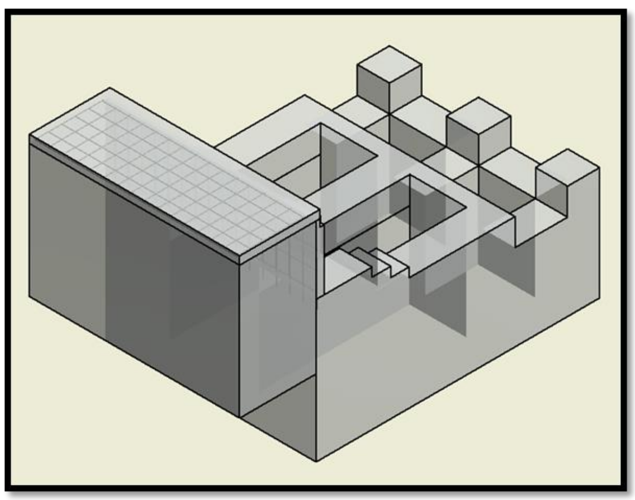


Figure 39: Abutment design option 1 – solid block, shown on foundation.

This option is heavily overdesigned and very bulky, however most of it will be hidden beneath the backfill soil and again, in the context of the entire Myra Hiatt Kraft Wooden Footbridge Project, concrete and steel are very cheap materials and will not play a major factor in determining the final price of the project. In terms of constructability, this design is ideal and practical to install in the field. However it will cost more due to the amount of concrete utilized in the design.

3.3.5b Option 2-Hollow Block with Wings

A second design option was also created with the purposes of achieving a more economical design and minimizing the amount of materials brought into the site. This option utilizes the six inch thick wall designed in the sections above, and applies the design to two additional wings added on either side of the retaining wall itself and a landing placed on top of the abutment, as seen in Figure 40.

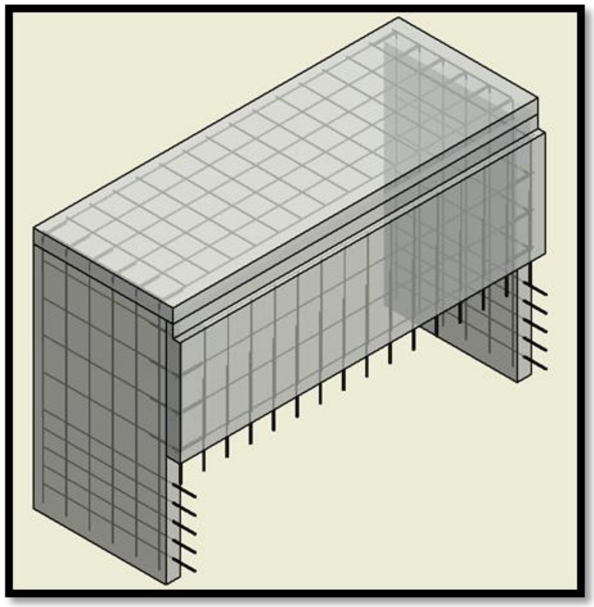


Figure 40: Abutment design option 2 – hollow block with wings.

In this way, the wings and landing will contain the same reinforcement as what was required for the initial retaining wall. This is a conservative approach as the soil forces acting on the wings are not equal in magnitude to those acting on the retaining wall itself, and it would also allow for a more easily constructible design. It is best practice to minimize the amount of differing sizes of reinforcing bars as it can cause confusion in replicating the plans in the field. Additionally, the wings should serve as mechanisms to allow for grading from the new pond walls up to the required elevation of the walkways approaching the bridge.

As can be seen from the design, instead of a massive block approach as used in design option 1, there is a void between the wall and the landing, which will be filled with backfill soil. This amount of soil would be equivalent to less than 20 cubic yards of fill, and would cost less than filling it with concrete and steel reinforcement instead. In addition, the abutment would be attached in the same manner as the option 1 to the existing foundation, through means of vertical and horizontal reinforcement, as seen in Figure 41.

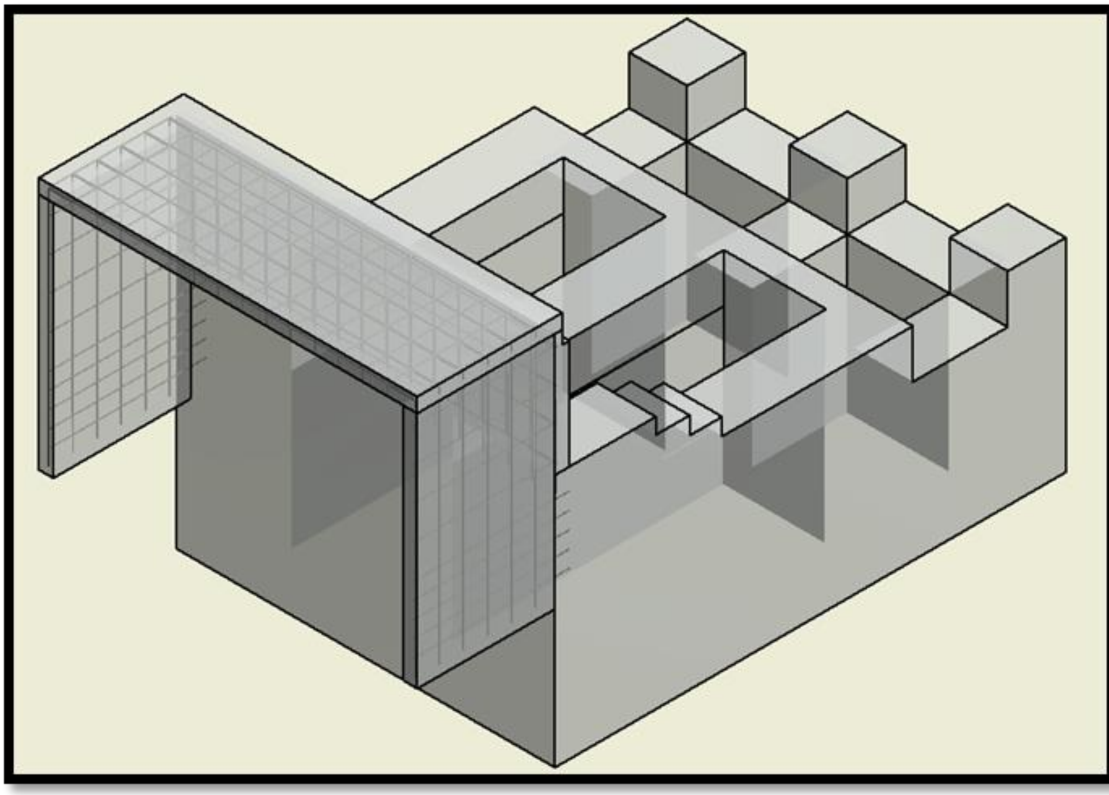


Figure 41: Abutment design option 2 – hollow block with wings, shown on foundation.

The two designs shown above were presented to the stakeholders involved in the landscape portion of the project and a final design was chosen with regards to constructability, feasibility, and cost.

3.4 Site Design

This section details the phases that were involved in the development of landscape design. It reviews the initial steps that were taken to map the existing conditions of the park, the intermediate steps that involved many different meetings and presentations to relevant stakeholders regarding site design alternatives, and the final steps of generating engineering plans for the site work as well as a cost analysis for the materials needed to complete the work.

3.4.1 Initial Observations

To begin the site design process, the landscape that surrounds the Elm Park Red Wooden Footbridge on the east and west sides of the park was evaluated. These areas have been graded and maintained over the past 125 years to adhere to the original bridge design, whose landings at the bottom of each bridge wing are at an elevation of about 493 feet, according to the City of Worcester

Sewer Datum. In order to maintain the required clearance under the bridge while still being accessible, the new elevation at the landing of each bridge wing necessitates a new elevation of about 498 feet. This inevitably creates a challenge, especially considering that the highest existing elevation within a 50 foot radius of each side of the bridge is only about 493 feet, which is roughly a five foot difference from what is needed for the new bridge design.

In order to make up the difference between the existing and new elevation and maintain both an ADA accessible bridge design and walkway design leading up to the bridge, an extensive amount of site grading and site design was required. There were many variables that had to be considered before site design could take place. The first of these was integrating the new landscape design with the existing topography. Also of consideration was maintaining ADA-compliant walkways while eliminating any drops that may have necessitated the addition of railings. Third, the location of trees, lampposts, and benches in the vicinity of the bridge also needed to be considered. Finally, the cost associated with new material and labor needed to be minimized.

In order to integrate the old site landscape and new site design, any unnatural discontinuities with the elevation had to be eliminated. This meant that the grading of slopes needed to mesh smoothly into the existing landscape. This also came into play in considering ADA pathway requirements, which mandate a 5% grade or shallower to remain within standards. Altering the elevation of certain areas and walkways in the park would have an effect on other park objects such as lights, trees, and benches that were present prior to the construction. These objects needed to be relocated or removed depending on certain circumstances. Changes to the site included the introduction of new materials such as stones, concrete, and soil for new pond walls, abutments, and additional fill. Therefore, the cost of these new materials had to be considered, so that a cost effective design solution could be implemented in the park landscape contiguous to the new bridge.

The site work required for this project also necessitated efficient coordination with the Worcester Parks and Recreation Department, as they are the lead project managers for all the phases of construction that has taken place during the Elm Park renovation. Meetings were scheduled with the City of Worcester Parks and Recreation team in order to determine the vital areas of landscape design surrounding the bridge, the limitations inherent with the new landscape design, and most importantly what they were looking for as the client. From those meetings, a proper methodology and course of action considering the landscape design process was developed, which was broken down into the following phases:

1. *Map the existing conditions of the landscape.*
2. *Create site design alternatives.*
3. *Present alternatives to the City of Worcester Parks and Recreation Department.*
4. *Final Design Option*

3.4.2 Mapping of the Existing Landscape Conditions

In order to determine the extent of landscape design and site grading that was needed for this project, it was imperative that a solid understanding of the existing conditions in the park was developed. This included a sound knowledge of the existing topography, locations of trees, lampposts, benches, underground piping, underground wiring, and relative boundaries between the mere's and the land adjacent to the bridge.

This first required that a detailed site survey be completed. A survey is a way to map out the park and its existing topography using data points gathered through the global positioning system (GPS) instruments so that it can be translated into a table of points. These points were then imported into a computer aided drafting (CAD) program such as Civil 3D or Revit Structure for analysis and modification. Before construction in the park began, the Worcester Parks and Recreation Department contracted Beal's & Thomas Inc., a local surveying/engineering company, to complete a detailed site survey of the park and create site plans for the Elm Park renovation project from that survey data. This was completed in February of 2012, using a Zeiss Elta Total Station instrument, with relative elevations acquired from the City of Worcester Sewer Datum.

The survey points were loaded into a CAD program where they created a surface. The generated surface was a raw form of a topographical map, without marked contours, elevations, or park objects. Then the surface was modified and specific layers are added to include all contour lines and additional elements in the park. The final product, seen in Figure 42, included a survey that extended from Park Avenue on the West side of the park to Russell Street on the East side, and from Highland Street at the North end of the park to Elm Street at the South end. All elevations were marked on contour lines, as well as locations of all trees, bushes, piping, wiring, lighting, and path boundaries identified.

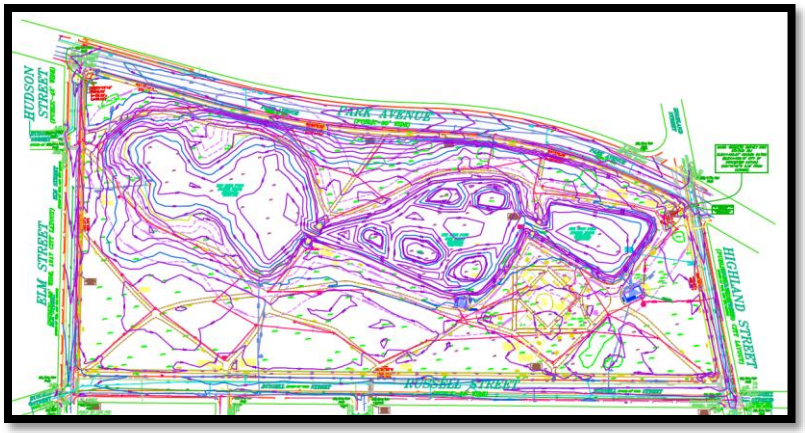


Figure 42: 2012 Elm Park site plan

The Worcester Parks and Recreation Department was able to send this file along with other site documents showing where work had already been conducted since the creation of the CAD file. Changes that had been already made within the park that were in the area of the bridge were the creation of the pond edge walls on the North Mere and also renovation and grading of existing paths.

The CAD file provided by the city was opened in Civil 3D. It was found that file was very detailed to the point that many of the components were not needed. The main interest was in the existing walking paths and changes to them, elevations and contours, and general locations of trees and lights within the area surrounding the Myra Hiatt Kraft Footbridge construction site. With the areas of interest narrowed down, the software was used to hide the elements that were not necessary and generate the view seen in Figure 43. This file was then saved and opened in another CAD software, Revit Architecture, that would be used to model the new site design in a three dimensional format.

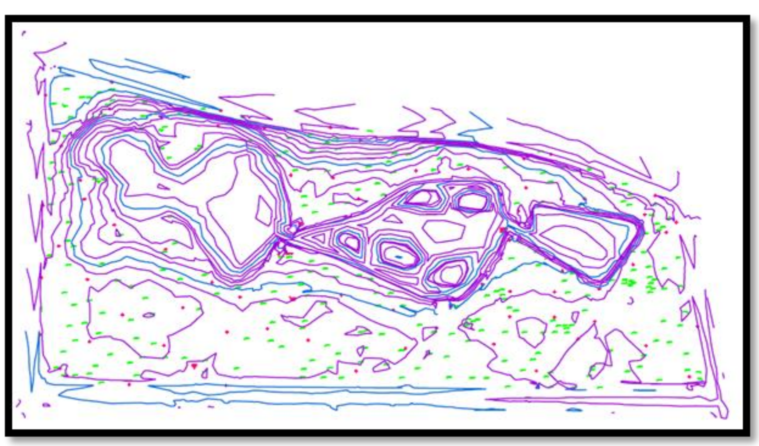


Figure 43: Elm Park site plan with contours only.

Since the first file displayed the entire park, it was a necessary first step to isolate the region of interest surrounding the bridge where additional site work would need to be conducted. The area of interest was enclosed using a box and individual elevation points were placed on the existing contour lines to generate a new surface in the program. With this surface, the software was able to differentiate the land, water, and walking paths seen in Figure 44.

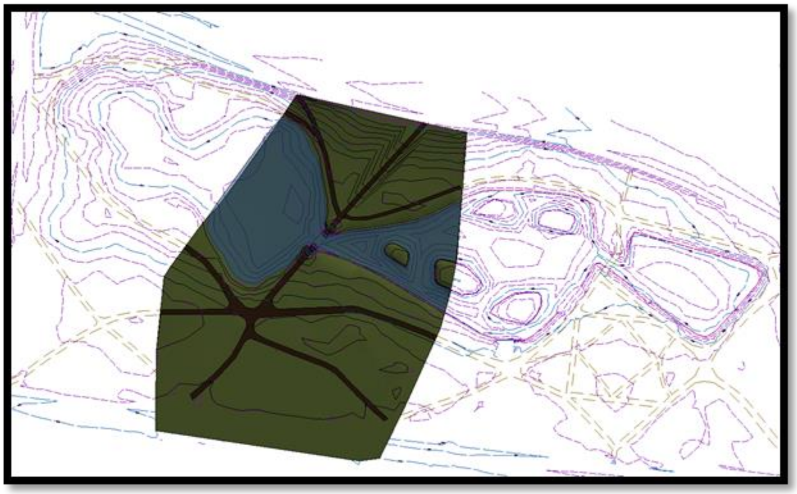


Figure 44: 3D Conceptual model of existing site conditions.

This format, now three-dimensional, allowed viewers to observe the changes in elevation more easily and define where the water would rise to at its highest point on the bank. The North Mere also needed to include the pond walls that had already been constructed to be completely accurate. Using CAD software, a model of the wall at the exact height and size based on the plans provided by the Worcester Parks and Recreation Department was created. The final elements of the existing landscape that were added to the drawing were the original foundation in their exact locations. With the all of the different site elements incorporated in the existing landscape model, it could then be used as basis to start building new site design alternatives.

3.4.3 Site Design Alternatives

Once an existing model of the site was created, new site design alternatives were considered and designed. In order to better follow the process that was taken to work with the site, it was divided into six regions, as seen in Figure 45. Each region represents a different area of work and will be referred to accordingly in the following sections.

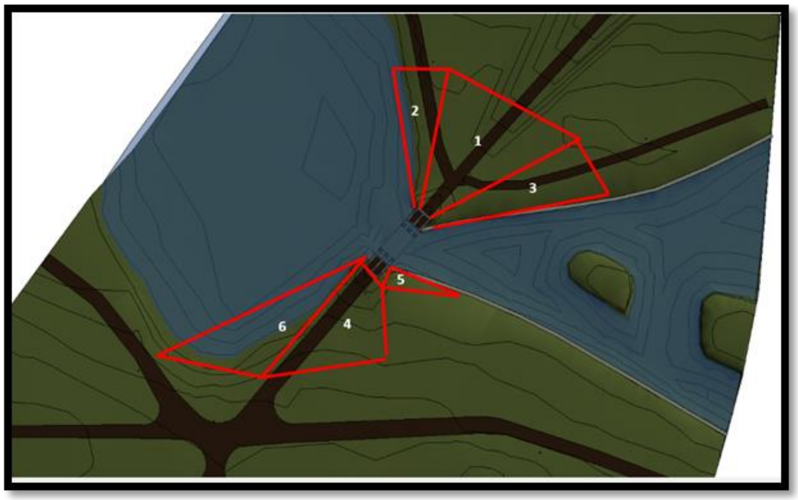


Figure 45: CAD Modeling of regions

To begin, the design worked off the knowledge that the bridge wing would terminate at an elevation of 498 feet and the top of the existing pond wall discontinued at 492 feet. These elevations allowed for the establishment of a starting and ending point that could in turn be graded from. Prior to any major changes being made, a pond edge wall was placed in the South Mere where it would potentially serve as a definite boundary between the existing landscape and the pond. An example CAD drawing of the new bridge was also placed in its exact location in the park for site grading purposes.

One of the main concerns regarding the new site design was impeding the soil erosion near the foundation as it had occurred in the past. To combat this, retaining walls were implemented in the park landscape adjacent to the bridge as a first alternative. Through discussions with stakeholders it was decided that retaining walls might present the best option in terms of making up the required height difference dictated by the new bridge while mitigating existing erosion issues.

The first step in altering the landscape design was to grade the landscape on the Park Avenue side in zone one from the side of the bridge landing down to the existing elevation. This area of the park had recently gone through extensive renovations to make the walkway descending to the bridge ADA compliant, so it was important to ensure that this work was considered and incorporated in the new design. The slope of the walkway needed to adhere to 1:20 slope requirements, so to be conservative; a 1:25 slope was graded, with four 25-foot concentric increments measured directly out from the edge of the bridge landing toward Park Ave. It was at this point 100 feet from the bridge and at an elevation of 494 feet that existing grade was met.

Regarding the two opposite slopes coming from the bridge and from the walkway descending from Park Ave, the point that was common on both walks would be at an elevation of 494 feet and a distance of 87 feet from the bridge. Knowing this location, the slope of the new walkway approaching the bridge was designed so that it met with the existing ADA walkway and was still compliant. It was also noted that the two other paths that approach the bridge on west side of the park needed to be graded in a similar manner. As a result, the approach of radial grading was adopted for the entire area on the west side of the park rather than just the paths themselves. This would allow for smoother transitions between pathways and a more natural looking landscape.

Zone three was examined next. This side provided a challenge in that the grading was constrained by the existing pond edge wall and the extreme elevation change. The proposed design included a retaining wall that would account for the maximum 30-foot vertical drop, and would taper in height and lateral distance until it merged with the existing pond wall. This wall would extend 150 feet from the bridge until it terminated. Grass between the two walls could then be at a 1:3 grade, which was a predetermined limit to allow for mowing and routine maintenance. Figure 46 demonstrates how this wall may look if one were standing on the Park Ave side looking at the bridge and the South Mere. In zone two, a similar wall was proposed to that in zone three, however the retaining wall and pond walls did not converge as on the North side. Instead, the wall was to terminate into the soil, extending 75 feet from the bridge. Mowing the grass in this area would also be possible at the grade of 1:3. Figure 47 shows a view of this retaining wall arrangement.

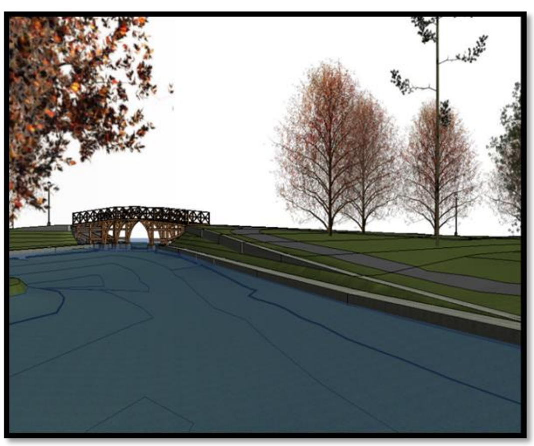


Figure 46: View of site from Park Ave. side of Elm Mere facing South Mere

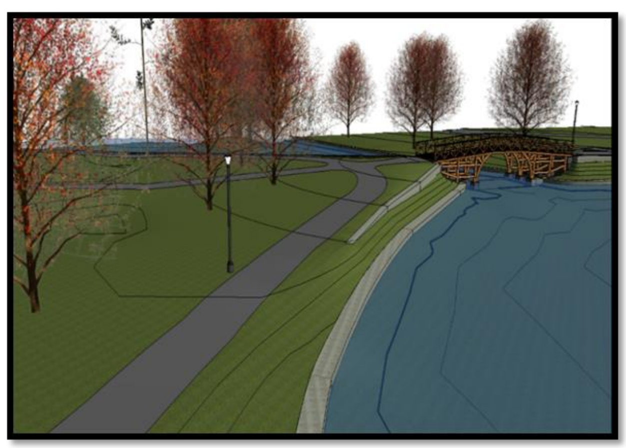


Figure 47: View of site from Park Ave. side of South Mere facing Elm Mere

Once the Park Ave side was completed, focus turned to the Russell St. side of the park. As on the Park Ave side, the path leading up to the bridge was constructed first in zone four. The walkway intersection leading up to the bridge was at an elevation of 495 feet. Therefore, the grading followed straight along the path from a 498 feet elevation at the edge of the bridge wing, out 75 feet to an existing elevation of 495 feet. This slope complied with the 1:20 standard required for accessible walkways.

In zone six there was very little room to drop the elevation down from the path to the pond wall. Therefore, it was determined that the best solution was to add a 30 foot long retaining wall on both sides of the path. Then starting at 24 inches below the top of the wall, the land was graded down to the proposed pond wall. This resulted in a slope of about 1:1.5.

The final site area considered was zone five. This area was the most simple to grade, especially being able to start from the retaining wall in place from the previous steps. It was found that the point that was 24 inches below the top of the retaining wall on the north side of Russell St. allowed for a starting point that enabled a mesh of the new slope with the existing slope at a 1:3.5 grade. The area of most concern in this zone was between where the retaining wall ended and the pond wall started. This area had about a 1:1.75 slope. Figure 48 and Figure 49 are conceptual representations of what the retaining walls would look like from both the north and south directions.

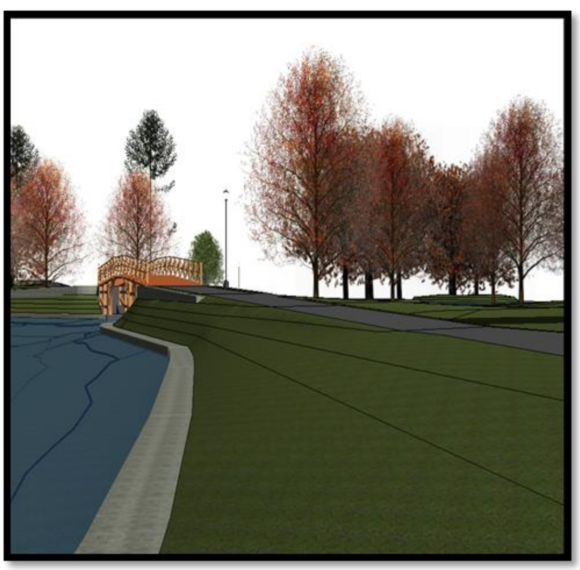


Figure 48: View of retaining wall looking north

Figure 49: View of retaining wall looking south

With the grading now complete, the focus moved to the park in general. The site design required the alteration of the location of a few of the existing walking paths so that they curved with the mere more fluidly. This design also included an additional short cut in zone five to better control the flow of foot traffic. Figure 50 shows the park after all changes were made from a plan view facing in the direction of Park Ave. The locations of the existing trees and additional lampposts should also be noted.

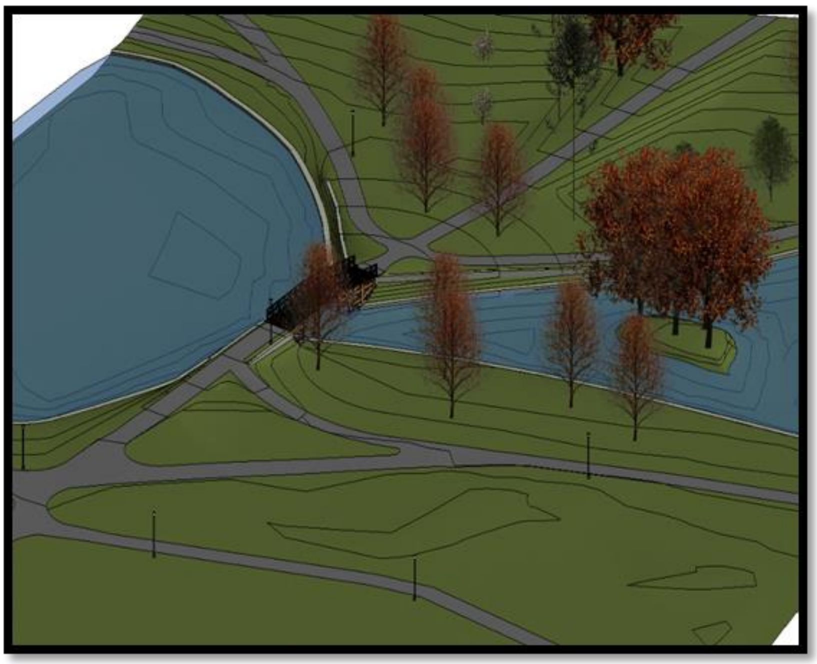


Figure 50: Bird's eye view of initial site design alternative

3.4.4 Present Alterations

In December, the proposed changes to the site design were presented to the relevant stakeholders involved in the site design process. After the presentation, the City of Worcester Parks and Recreation Department examined the proposed changes to discuss their own thoughts internally. In mid-January, a meeting with the City of Worcester Parks and Recreation Department was held again and they provided feedback about the initial design. They expressed concerns about the retaining walls causing too much of a visual impact in the park. The side they were most concerned with was the south mere view of the bridge. They recommended that this view have no visible retaining walls within it. After further discussion, it was decided that the other side of the bridge, Elm Mere, should not have retaining walls either if possible. The Parks and Recreation team wanted to see more use of placed stone as it would provide a more natural, rustic look, coinciding with the rest of Elm Park. Additionally, the Parks and Recreation team recommended that the boundaries of the proposed South Mere pond wall be adjusted to allow for more gradual slopes to be established between the new walkways and the South Mere. It is important to note that the amount of fill added to accommodate the new locations of the walls must then be subtracted somewhere else along the South mere edge to offset the change in area, per requirements from the Massachusetts Conservation Commission and the Worcester Parks and Recreation Department.

Some other things that the Parks and Recreation team wanted to be considered in the next design proposal included which type of post was going to be placed in the paths adjacent to the bridge to prevent vehicle traffic, and finally an idea for a dedication plaque for the bridge.

3.4.5 Creation of Final Site Design Alternative

The feedback from the Parks department was very helpful and manageable. The ideas they gave were taken into consideration and as a first step, all of the retaining walls were removed from the site. This created a discontinuity in the surface but it was the best place to begin work. A modified version of the bridge with a footprint of the final abutment and landing design was then placed in the landscape setting. The foundation setup would serve as a starting point to begin grading from. The team then examined the boundaries of the proposed south mere pond wall addition, and explored what changes could be made to the wall to allow for more gradual, maintainable slopes per the requests of the Parks and Recreation Department. Once a final location of the south mere pond was established that would keep the net area of the pond relatively the same, alternative grading was added to adhere to both the new abutment location and south mere pond wall locations.

The group also considered ways to create less drastic slopes directly adjacent to the bridge on both the east and west sides of the park. It was decided that placed granite stone may serve as a feasible option. Placed stone would not only reinforce the steeper slopes near the foundation, but also create a more natural appearance as seen throughout the rest of the park, especially compared to the previously proposed retaining walls.

Railings or curbs on the landing were also considered due to the drop-off between the elevation at the top of landings and the adjacent placed stone grading next to the landings. However, it was a priority for the project group to implement a drop-off between the two elevations that was below the 30 in. limit as stipulated by the ICC.

The project group also looked at ways to implement both a dedication plaque and a bollard (vehicle traffic impediment) in the vicinity of the bridge. Different options were explored that were both visually appealing and appropriate in terms of the entire bridge structure and surrounding site.

A final site design proposal was again created in Revit Structure and presented to the Worcester Parks and Recreation Department and Beal's and Thomas Engineering firm in mid-February. Conceptual models were created for the final design as well as engineering plans, which can be seen in the results section below.

3.5 Cost Analysis

After all the designs were completed, a cost analysis was conducted. As this project is sponsored by the city of Worcester, it was necessary to compile a rough cost estimate for them so that they may allocate their budget accordingly for the Elm Park Project. This cost analysis included the materials and equipment needed for constructing the bridge, the new abutment and site design, and a prediction for future maintenance costs.

The cost estimate for bridge materials and connections came from two separate suppliers. These suppliers were chosen based on previous relations with the Worcester Technical High School and Steve Harvey. Ultimately, the materials will go to bid, so though these may not be the same as will be used, they provide a basis for establishing a preliminary understanding of the magnitude and cost of work to be completed.

The other cost estimates that were compiled, such as for the abutment and site design, were derived from values seen in R.S. Means Cost Data books. The values provided in these books are meant to only serve as an estimate for construction quotes and may not exactly reflect current prices from today's manufacturers. Some mark-ups were assumed as part of the cost estimation process in order to be conservative. Detailed cost estimations can be seen in the results section.

4.0 Results

The results of this project include the final bridge design and the final site design. Included in the final bridge design are the architectural design and member sizes as well as the connections. The site design included the foundation analysis, the abutment design and the final site design. The final cost analysis was also conducted to provide an estimated cost of this project for the city.

4.1 Bridge Design

The final architectural design agreed on by the Worcester City Manager's Office, the Worcester Technical High School, the Parks Department, and the Professional Engineer was the one that all felt would optimize the design architecturally, historically and economically. This design can be seen in Figure 51. The chosen design featured adequate structural members, complied with all accessibility regulations, and maintained the historical integrity of the superstructure and the railing design.

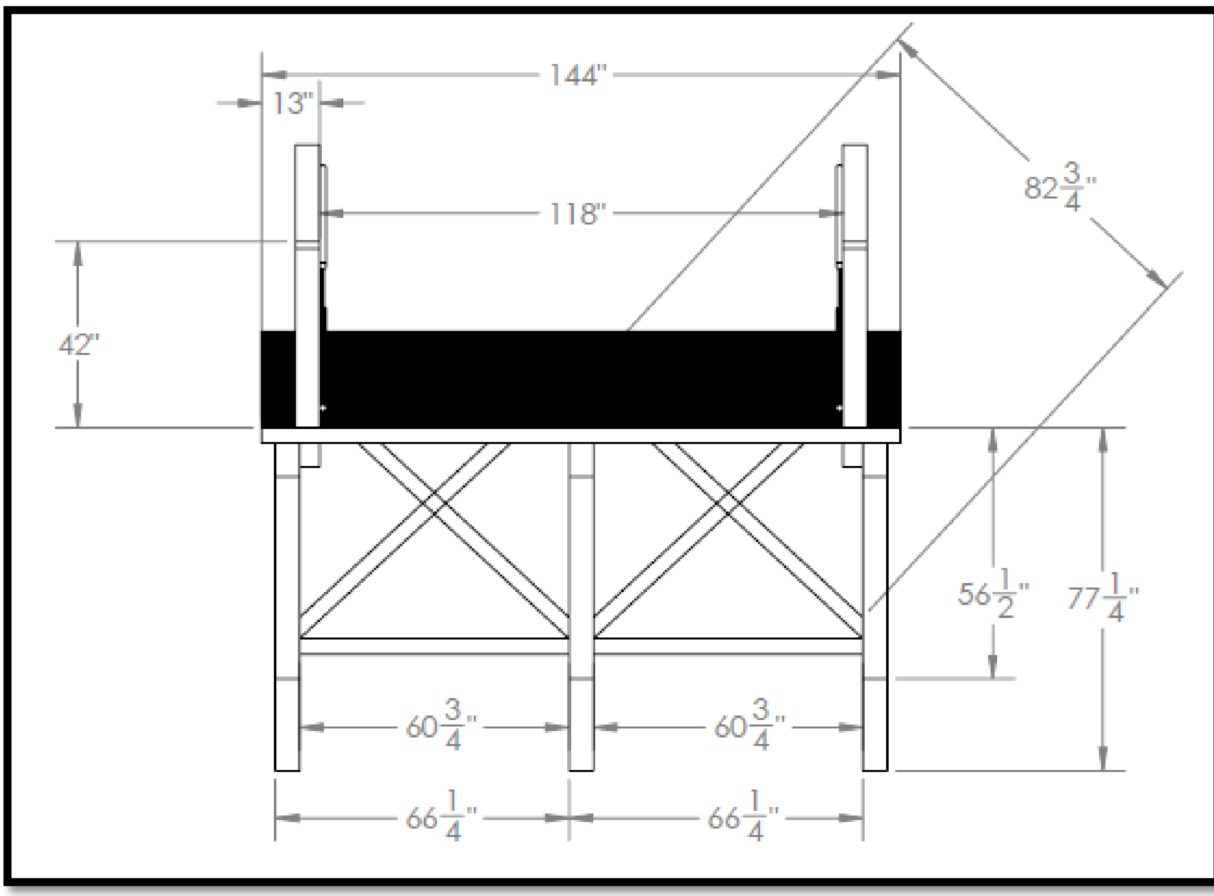


Figure 51: Final architectural dimensions - front view.

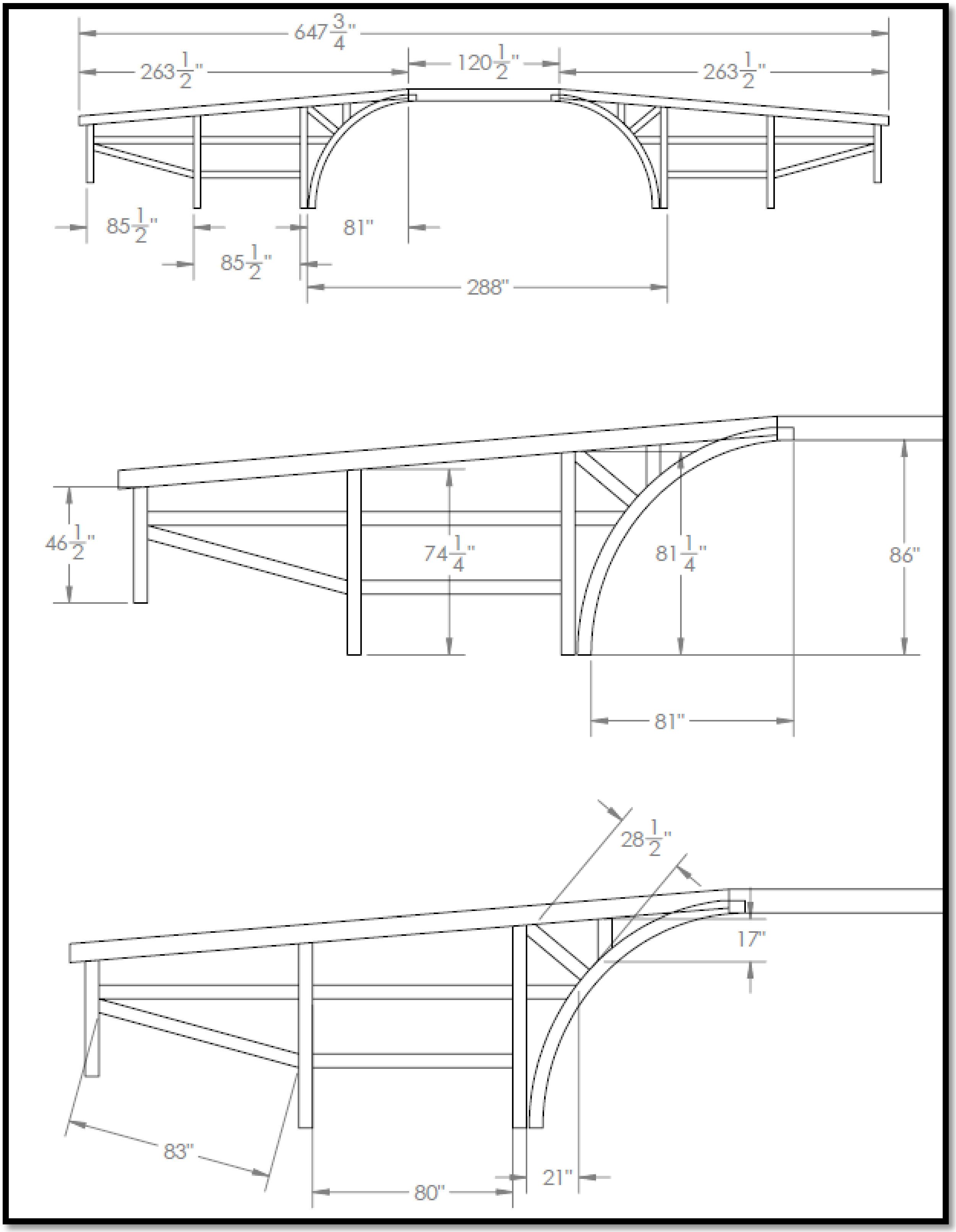


Figure 52: Final architectural dimensions – side view.

Three major changes occurred to the support structure of this bridge. The first major change was designing an ADA compliant slope. The maximum ADA compliant slope is 1:12 or 4.75°, so a slope of 4.7°was used to be slightly conservative. The second major change was reducing the number of columns from four to three to reduce costs of material and connections. The final major change was changing columns from two adjacent 4x6 columns with double shear connections to a single 6x6 column utilizing concealed connections. The CAD images of the bridge can be seen in Figure 53 through Figure 57.

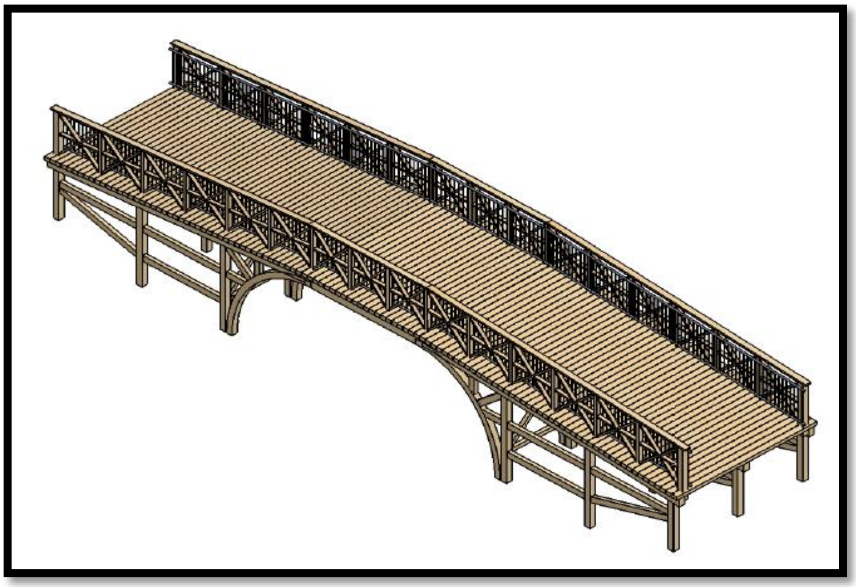


Figure 53: Isometric view of the bridge from above.

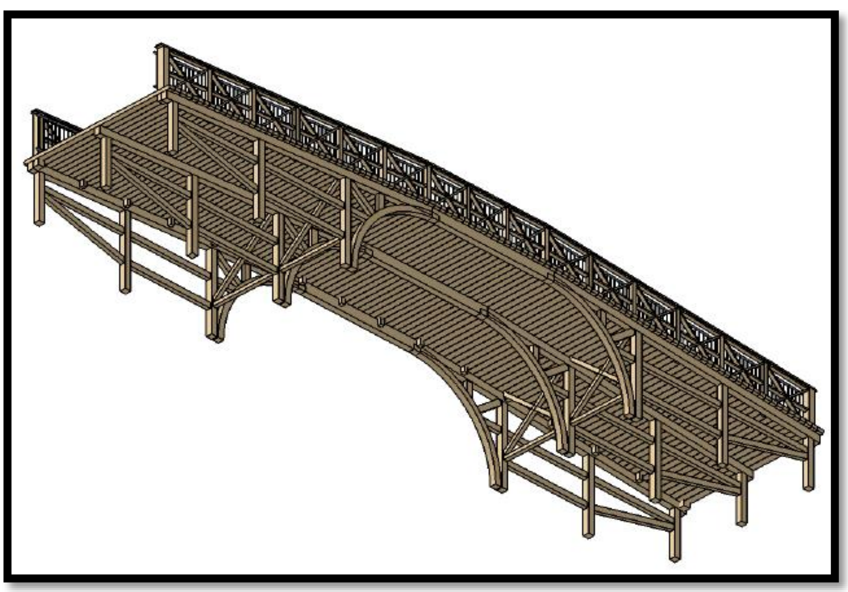


Figure 54: Isometric view of the bridge from below.

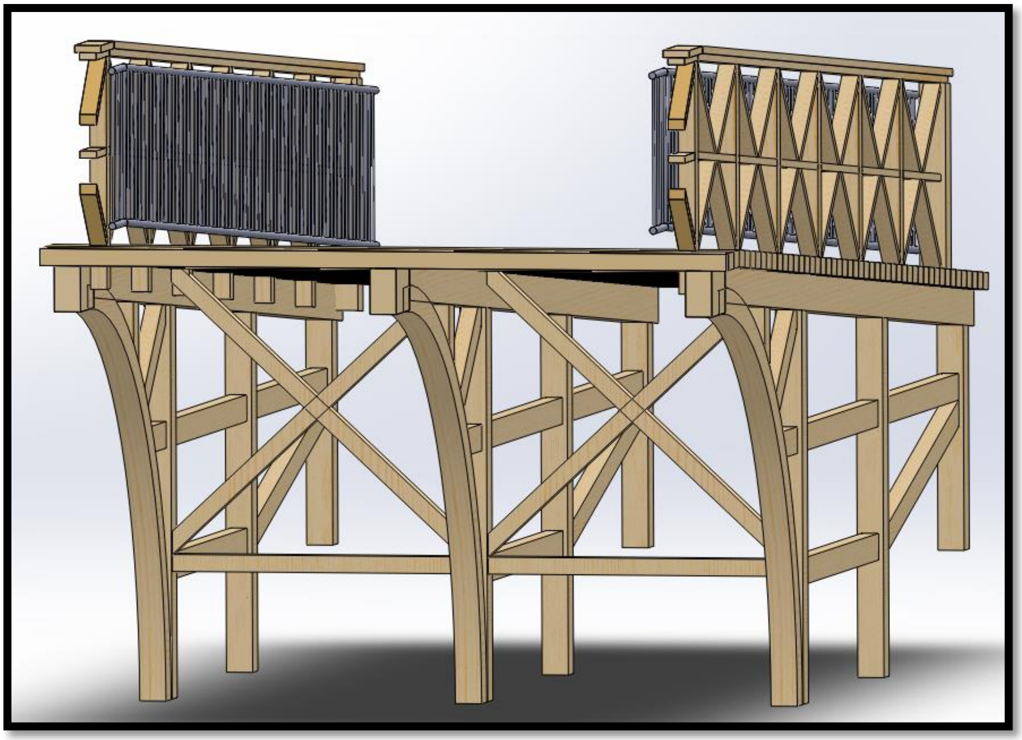


Figure 55: Middle section view of the bridge.

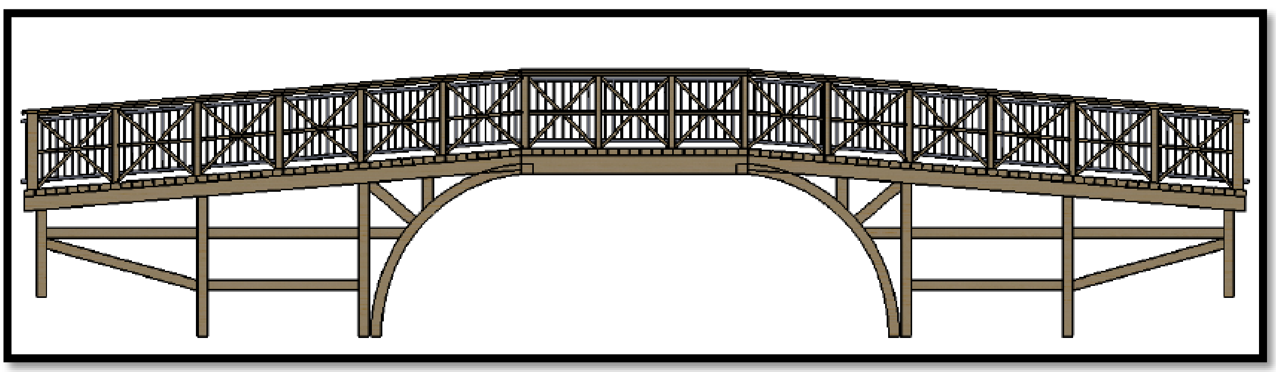


Figure 56: Side view of the bridge.

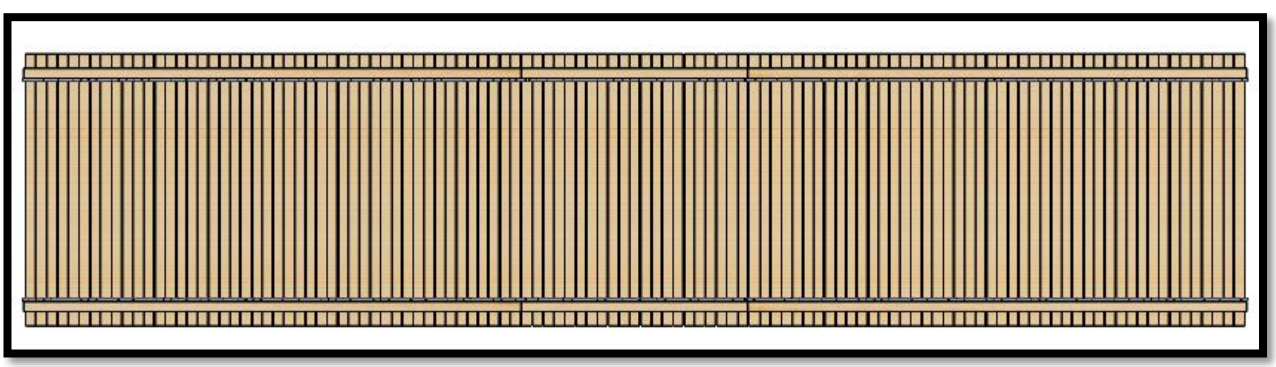


Figure 57: Top view of the bridge.

With the final design, the spans, nodal locations, and loads to design for the final bridge were presented to the City of Worcester. To go along with the architectural design of the bridge, the following bill of materials in Table 23 describes each member and its dimensions based on the final engineered design. Also in the table is the total number of those members that are present in the whole structure – 'Member Qty' – and the member size and quantity to be ordered from the supplier – 'Size Using' and 'Qty Required'.

Table 23: Bill of materials for timber members of final design.

Item No.	Member size	Description	Member Qty	Size Using	Qty. Req.
1	6x6x82	Column 1 - External	6	6x6x16	3
2	6x6x75	Column 2 - Middle	6	6x6x16	3
4	6x8x265	Diagonal Decking Support	6	6x8x12	12
5	5x5½	Arch Member (Glulam—ID 48, curved)	12	Custom	12
6	4x6x80	Horizontal Beams	18	4x6x8	18
7	4x6x84	Diagonal Beams	6	4x6x8	6
8	4x6x20	Arch Connection piece Horizontal	6	4x6x12	1
9	4x6x30	Arch Connection piece Diagonal	6	4x6x12	2
10	4x6x17	Arch Connection Piece Vertical	6	4x6x12	1
11	6x10x120	Flat Decking Support	3	6x10x12	3
12	4x4x48	Railing Post for Wings	20	4x4x12	6
13	4x4x48	Railing Post for Flat Section	8	4x4x12	3
14	6x6x48	End Railing Posts on Wings	4	4x4x16	1
15	4x6x144	Decking Members	113	4x6x12	113
16	4x4x61	Horizontal Cross Brace Beam	4	4x4x12	2
17	4x4x86	Diagonal Cross Brace Members	8	4x4x8	8
18	2x4x26	X-brace piece for bridge wing	48	2x4x12	12
19	2x4x26	X-brace piece for bridge wing	48	2x4x12	12
20	2x4x40	Railing Middle Beam for bridge wing	24	2x4x12	8
21	2x4x25	X-brace piece for flat section	24	2x4x12	6
22	2x4x35.5	Railing Middle Beam for flat section	6	2x4x16	1
23	2x4x260	Bottom handrail for Bridge wing	4	2x4x12	8
24	2x4x120.5	Bottom handrail for flat section	2	2x4x12	2
25	2x6x120.5	Top handrail for flat section	2	2x6x10	2
26	2x6x266	Top handrail for bridge wing	4	2x6x12	8

The members are labeled on the support in Figure 58. Both external supports and the internal support have the same sized members and the same orientation of the members; all members are also symmetric on either side of the bridge. Cross sectional members are labeled in Figure 59.

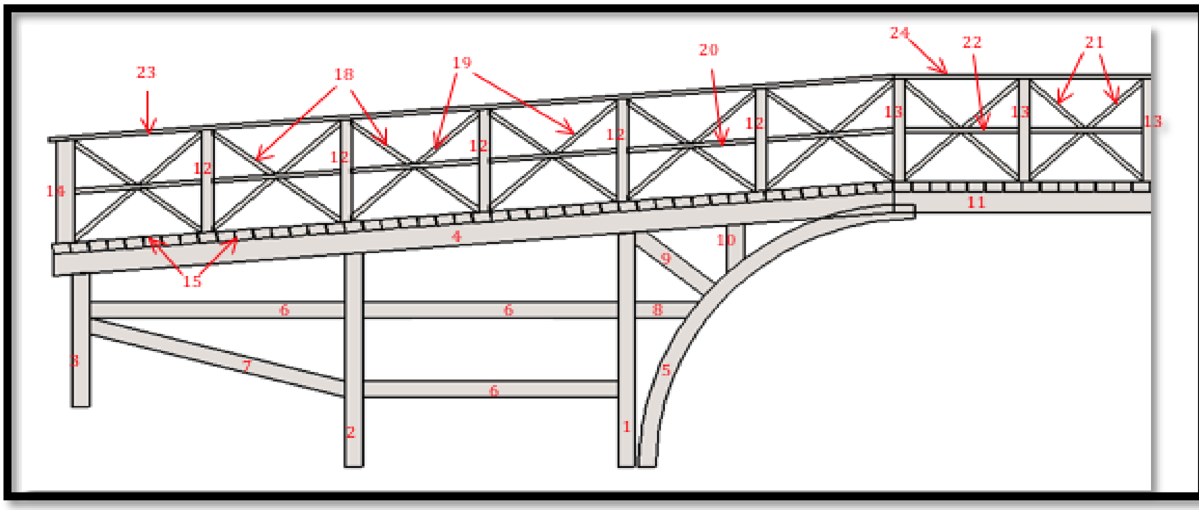


Figure 58: Figure Corresponding with Table 20, member size and placement of support.

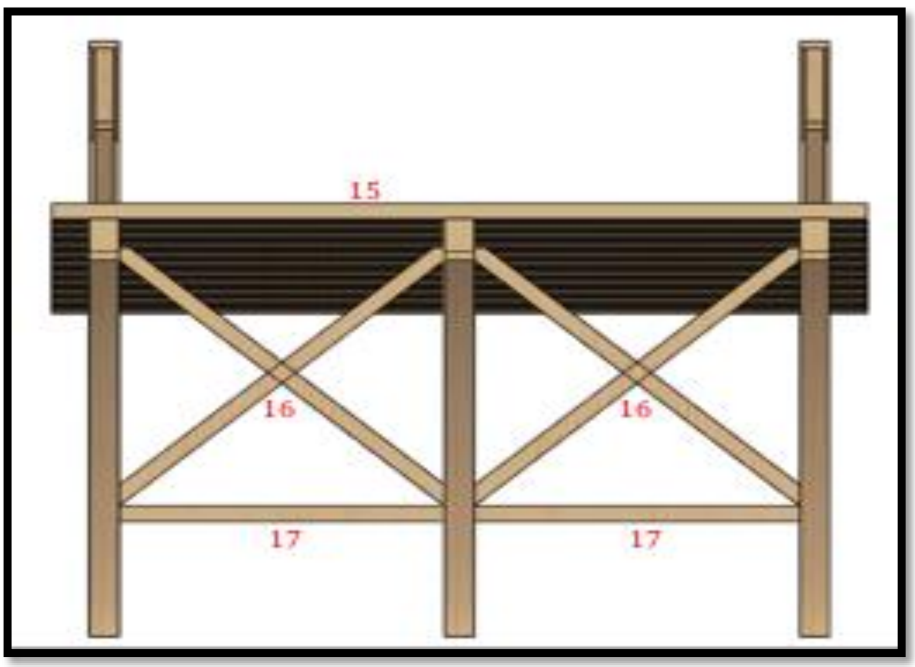


Figure 59: Figure corresponding with Table 20, member size and placement of cross bracing.

4.1.1 STAAD Results

The STAAD analysis yielded the following results. Each maximum value represents the maximum value for that member in either the internal span, or one of the external spans. For example, the external column (column 1 and member number 3 in figure 71) had a compressive stress of 37psi on one external span, 42psi on the internal span, and 37psi for the other external span, and thus a maximum compressive stress of 42psi. The following tables show these maximum values for all beams, columns, decking supports, and the arch.

Table 24: STAAD results compared to allowable for columns.

	C1 (member 3)		C2 (member 2)		C3 (member 1)		A1 (member 5)	
	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated
$F'_{b,z}$ =	1468 psi	309 psi	1468 psi	269 psi	1468 psi	728 psi	1468 psi	758 psi
$F'_{b,y}$ =	1469 psi	0 psi	1469 psi	12 psi	1469 psi	0 psi	1469 psi	0 psi
F'_t =	0 psi	0 psi	950 psi	30 psi	0 psi	0 psi	0 psi	0 psi
$f_{v,y}$ =	285 psi	12 psi	285 psi	8 psi	285 psi	41 psi	285 psi	134 psi
$f_{v,z}$ =	285 psi	0 psi	285 psi	0 psi	285 psi	10 psi	285 psi	0 psi
F'_c =	893 psi	326 psi	886 psi	41 psi	890 psi	307 psi	893 psi	627 psi

Table 25: STAAD results compared to allowable for beams.

	B1 (member 6)		B2 (member 6)		B3 (member 8)		B4 (member 7)	
	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated
$F'_{b,z}$ =	4086 psi	401 psi	4086 psi	401 psi	4143 psi	11 psi	3322 psi	401 psi
$F'_{b,y}$ =	4156 psi	0 psi	4156 psi	0 psi	4156 psi	0 psi	3370 psi	0 psi
F'_t =	2359 psi	0 psi	2359 psi	0 psi	2359 psi	12 psi	1853 psi	7 psi
$f_{v,y}$ =	302 psi	27 psi	302 psi	27 psi	302 psi	1 psi	302 psi	27 psi
$f_{v,z}$ =	302 psi	0 psi	302 psi	0 psi	302 psi	0 psi	302 psi	14 psi
F'_c =	945 psi	105 psi	945 psi	130 psi	3383 psi	0 psi	880 psi	4 psi

Table 26: STAAD results compared to allowable for beams and decking members.

	B5 (member 6)		B6 (member 9)		B7 (member 10)		Decking Members	
	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated
$F'_{b,z}$ =	4082 psi	1 psi	4132 psi	2 psi	4143 psi	0 psi	4027 psi	1840 psi
$F'_{b,y}$ =	4156 psi	0 psi	4156 psi	0 psi	4156 psi	0 psi	3538 psi	8 psi
F'_t =	2359 psi	0 psi	0 psi	0 psi	0 psi	0 psi	0 psi	0 psi
$f_{v,y}$ =	302 psi	0 psi	302 psi	0 psi	302 psi	0 psi	302 psi	38 psi
$f_{v,z}$ =	302 psi	0 psi	302 psi	0 psi	302 psi	0 psi	302 psi	0 psi
F'_c =	873 psi	151 psi	2989 psi	408 psi	3381 psi	260 psi	1377 psi	5 psi

Table 27: STAAD results compared to allowable for decking supports and diagonal cross supports.

	D1 (member 16)		D2 (member 17)		S1 (member 4)		S2 (member 11)	
	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated	NDS Allowable	Maximum Calculated
$F'_{b,z}$ =	4116 psi	1268 psi	4114 psi	4027 psi	1463 psi	948 psi	1457 psi	985 psi
$F'_{b,y}$ =	4156 psi	105 psi	4156 psi	3538 psi	1469 psi	40 psi	1469 psi	40 psi
F'_{t} =	2359 psi	0 psi	2359 psi	0 psi	950 psi	0 psi	0 psi	0 psi
$f_{v,y}$ =	302 psi	35 psi	302 psi	302 psi	285 psi	121 psi	285 psi	102 psi
$f_{v,z}$ =	302 psi	2 psi	302 psi	302 psi	285 psi	5 psi	285 psi	4 psi
F'_{c} =	410 psi	14 psi	375 psi	1377 psi	712 psi	99 psi	631 psi	218 psi

Because of minor model discrepancies, the factor of safety of the entire bridge was calculated. The members with the smallest factor of safety are the decking support members with a factor of safety of 1.5. A factor of safety of 1.5 for our design is conservative and acceptable.

4.1.2 Railings

The final design for the railings can be seen in the images below. As they show the X-style railing with the internal aluminum railings which was considered to be the best option.

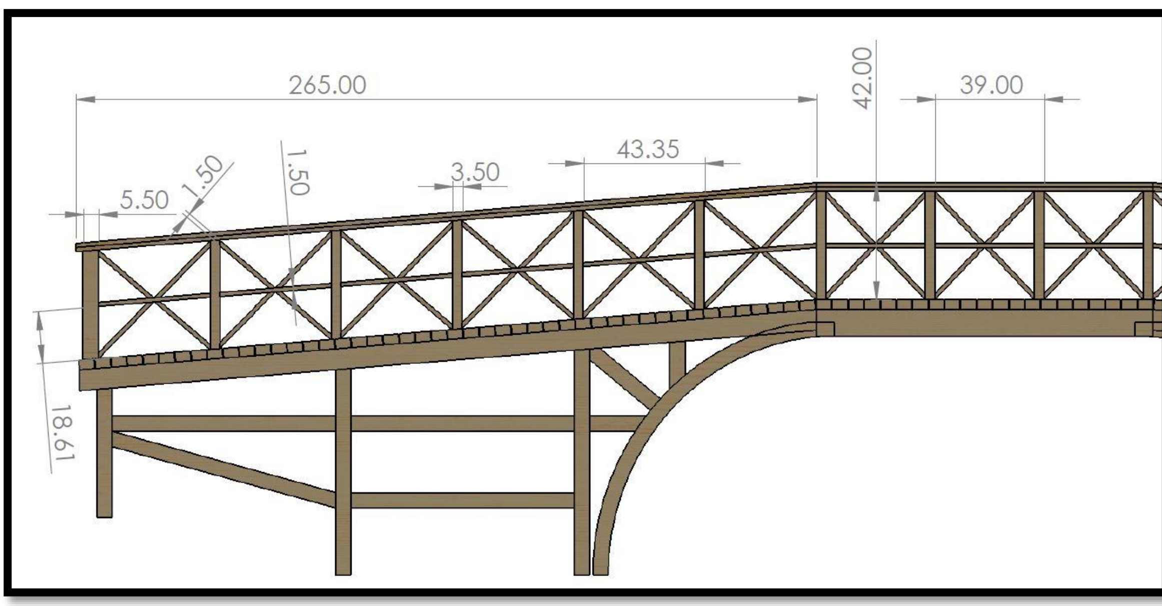


Figure 60: Wooden railing design with dimensions.

From the previous handrail design, preliminary sizes were chosen based on the hand calculations as well as the previous sizes. The initial size used for the handhold was a 2x6 member based on the IBC handhold size stipulation, but with a factor of safety, this member failed in bending. Thus, a second handrail of size 2x4 was added. This additional member, below the 2x6 handhold member, adds the necessary strength and code compliance. Similarly, the minimum size required for the railing columns is 4x4, and the minimum required size for the diagonal members is 2x4. The hand calculation results for each of the loading conditions are shown in the tables below, with the maximum stresses shaded.

Table 28: Railing design manually-calculated results.

Handrail Member					
Fv	9.03	11.11	9.03	11.11	psi
Fb	234.72	577.78	74.56	183.53	psi
Δ	0.0275	0.0542	0.0027	0.0052	in

Column Member			
Fv	19.90	24.49	psi
Fb	909.621	1119.5	psi
Fc	6.633	8.163	psi

Diagonal Member			
Fv	46.43	57.14	psi
Fb	1500.80	1847.1	psi
Fc	21.887	26.937	psi

From this the members were then analyzed in STAAD to get the full member stresses. And from this analysis, the following results were gathered from each analysis for each loading scenario. These results compared to the NDS design values are shown below.

Table 29: Railing design output results from STAAD.

LRFD Allowable Values vs. STAAD outputs									
	End Post		Angle Post		X members		Hand hold		
	Alllow	STAAD	Allow	STAAD	Allow	STAAD	Allow	STAAD	
$F'_{b,z}$	1467	*58.4*	4146	*987.1*	3866	*2312.9*	3278	*87.2*	psi
$F'_{b,y}$	1467	*351.5*	4146	*70.3*	3489	*364.5*	2794	*276.0*	psi
F'_{t}	950	0.0	1996	0.0	1814	0.0	3024	0.0	psi
$F'_{v,y}$	285	0.5	302	15.0	302	19.6	302	8.8	psi
$F'_{v,z}$	285	1.0	302	2.7	302	19.0	302	0.0	psi
$F'_{c\perp}$	703	0	1059	0	1059	0	1059	0	psi
F'_{c}	883	3.3	2998	371.2	1650	5.7	775	8.8	psi

As all of these results show, the hand calculations and the STAAD analysis yielded similar results. In STAAD the complexity of the structure was taken into account, and shows that the original dimensions are still safe with a factor of safety and thus acceptable.

4.1.3 Connections

For each joint in the bridge, connections were assigned to safely transfer loads between the members. The connections were optimized to be as small as possible and use the least amount of dowels while still being able to carry the loads without fracturing or rupturing the wood. The tables below show the number of each connection to be used throughout the bridge structure as well as the recommended dimension for each fastener.

Table 30: Total quantity of each type of connection in the bridge.

	Product	n_{plate}	n_{dowel}	n_{bolt}	n_{pin}	n_{screw}	n_{nail}
Concealed Post Tie	CPT66Z	24	120	48	72		
Concealed Joist Tie	CJT3	72	648		216	432	
Heavy Angle	HL53	54	312	312			
Custom plate		12	48	48			
Nails	8d		750				750
TOTAL		162	**1878**	408	288	432	750

Table 31: Dimensions for fasteners used at each connection locations.

	Fnd'n to Col.	Col. to Beam ⊥	Col. To Beam θ	Beam to Col.	Col. To Deck Sup	Deck Sup to Col.	Arch to Beam 11	Arch to Beam 20	Arch to Beam 21	Arch to H Deck Sup	X-Sups	H-Sups	Decking	
	CPT66Z	CJT3	CJT3	CJT3	HL53	HL53	CJT3	CJT3	CJT3	custom	nails	nails	nails	
d_{fast}	3/4	1/2	1/2	1/4	1/2	3/4	1/2	1/2	1/2	3/4	0.131	0.131	0.131	in
L_{fast}	5 1/2	3 1/2	3 1/2	3	5 7/8	9 2/3	3 1/2	3 1/2	3 1/2	6	3	3	5	in

The yield and withdrawal strengths for each connection can be found in Appendix H: Connection Calculations.

Connections were also identified for the members of the railings. In all cases within the railings, the members were connected by nails. However, when connecting the posts to the bridge structure, bolts were selected. The following table shows the connections to be used throughout the railings.

Table 32: Total quantity of each type of connection in the railings.

Design Values		Support to End Post	Support to Inner Post	Post to X	Mid Hold to X	Post to Upper Hold	Post to Mid Hold	
TOTAL	n	8	56	240	240	64	120	fasteners
		64	bolts	664	nails			

4.1.4 Deflection Check

The deflections on several loaded members were checked manually to ensure that they would not deflect too much. The nine members were loaded and the calculations were performed for three different load cases. The results of the deflection checks are located in Appendix K:

Bridge Member Calculations. All loaded members passed the deflection checks and therefore showed that the final bridge design was acceptable under loading.

4.2 Foundation Analysis

The following section reviews the findings for the existing foundations inspection. The findings address the results of the subsurface investigation, initial visual inspection as well as the results for the structural analysis of the foundations.

4.2.1 Subsurface Investigation

Figure 61, was generated using the information obtained as a result of the soil exploration. The left hand side is a representation of Boring #1 while the right side shows Boring #6. To the left of each representation are numbers indicating the depth of the soil strata. Each of the individual colored sections shows the varying soil strata throughout each boring. The sections were labeled at first with the soils visual identification. The blue line shown for each boring identifies where water was discovered after drilling was complete.

Throughout each of the test locations it was found that the soil beneath the surface varied extensively. Near the surface the soil tended to be more coarse sand and gravel. In all boring logs, peat organics were observed at varying depths. Below that, in general, were moist clay layers. The final layer tended to be a dense material consisting of predominantly sand.

Within each soil strata an N-Value, Gamma, Phi, and c-value are shown. These values were derived as a result of the initial logs provided to the group. The N-Value that was calculated for each of the soil layers was used to calculate the rest of the relevant values needed for future calculations. Once the N-value for each layer was known, the other values were derived from tables in two different online publications. The N-value was also useful in the classification of each layer because there is a direct correlation between N-value and soil type. The soil types are given within each soil layer as well. Figure 61 below was used to calculate the bearing capacity of the existing foundations as well as for calculations involving the abutment design for the new bridge.

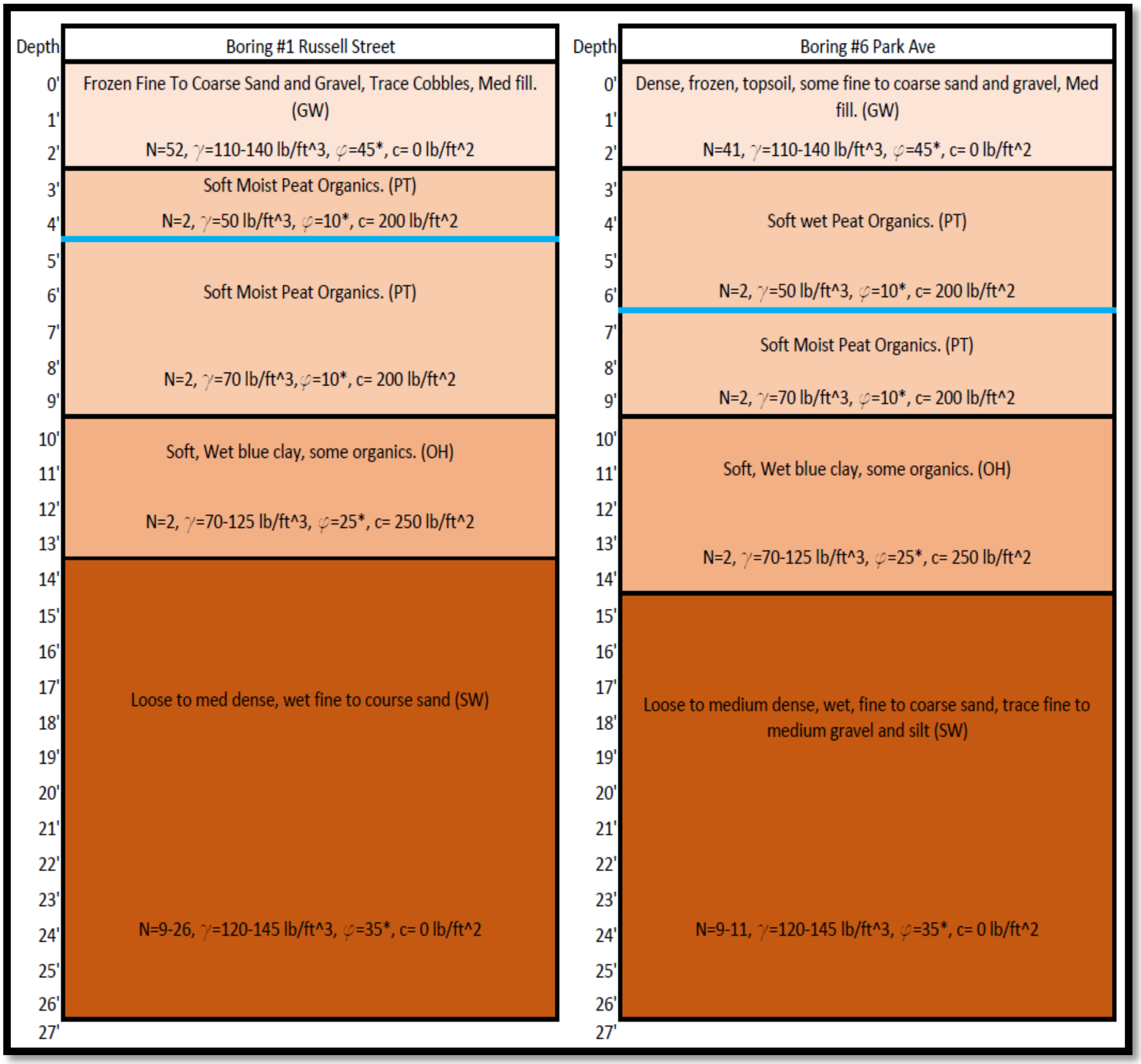


Figure 61: Soil boring results for Russell Street and Park Ave.

4.2.2 Visual Inspection

Upon conducting the initial foundation inspection, there were a few cracks that were apparent on the structures, as can be seen in Figure 62. However, there were no major deformities or any cracks that could be viewed as hazardous to the foundations structural integrity. Additionally, the erosion that had occurred on the foundations below the high water mark of the pond was noted. This erosion can be attributed to the constant exposure of the foundation to the pond water, and is feasibly that it would occur after 40 years of service. However the erosion, shown in Figure 62, can be considered negligible regarding the strength and capability of the foundation.

Figure 62: Foundation erosion.

During the assessment the anchor bolts and bearing locations on each of the foundations were examined. These are the areas where the wooden support posts of the bridge meet the concrete foundation. From visual inspection, it appeared these anchor bolts were in good condition and could more than likely be utilized for the purposes of anchoring the new bridge. However, the pedestals filled with mortar that surrounds these bolts were extremely damaged and heavily eroded, as seen in Figure 63. It is recommended that these original mortar pedestals be removed from the foundation and replaced with a new bearing surface that will receive the wooden support posts.

Figure 63: Pedestal erosion

Lastly, mold was observed on different areas throughout both foundations, especially, on the horizontal portions of the concrete. It is recommended that the mold and other superficial features of both foundations be addressed and cleaned properly before completion of the bridge structure (See Figure 64 and Figure 65).

Figure 64: Image of current bridge footings - close up view of a single pier.

Figure 65: Image of current bridge footings - wide view of multiple piers.

After the visual inspection of the foundations was complete, the measurements of the foundations themselves, including their spans, distance between each other, and relative heights in reference to the water line were collected. The foundations were hand drawn into engineering notebooks on site and the measurements were taken with a 300 foot measuring tape in English units. All the relevant vertical and horizontal distances were measured accordingly and accurate to a tenth of an inch. A three dimensional drawing of the foundation was created from the measurements that were collected and is shown in Figure 66.

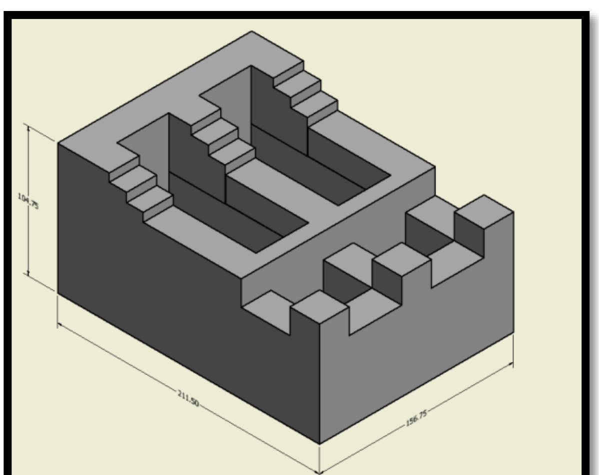


Figure 66: Isometric view of one half of the foundation.

In addition to the school's assessment of the foundations, Steve Harvey, a Professional Engineer at Harvey & Tracey Associates, a local Worcester engineering firm, conducted a private assessment of both foundations. Mr. Harvey's findings essentially validate the project teams' and speak to the irregularities that were noted in the initial inspection. A copy of his letter of approval to the Worcester Parks and Recreation Department for the foundations can be seen in Appendix L: Foundation Assessment Letter of Approval

4.2.3 Bearing Capacity

The pressure distributions for the three individual strips considered are summarized in Table 33 below. These pressure distributions are the result of the summation of both the weight of the individual foundation strips and the assumed loads caused by the new bridge structure. It is also important to consider that these pressure distributions are mirrored across the channel on the east and west foundations. Therefore it was only necessary to calculate the pressure distributions for one foundation and its corresponding three strips:

Table 33: Pressure distributions for strip foundations.

Strip 1 (Closest to South Mere)	1.32 ksf
Strip 2 (Middle Strip)	1.45 ksf
Strip 3 (Closest to North Mere)	1.32 ksf

Using the previously stipulated equation for bearing capacity as outlined by Vesic, an ultimate and allowable bearing capacity for the assumed underling soil was calculated and compared against the pressure distributions listed above for each individual strip. Table 34 summarizes these values:

Table 34: Bearing capacity values for strip foundation.

Ultimate Bearing Capacity	10,401.41 lb/ft^2 = 10.4 ksf
Factor of Safety (Assumed)	2
Allowable Bearing Capacity	5,200.71 lb/ft^2 = 5.2 ksf
Factor of Safety for Strips 1 &3	5.2/1.32 = 3.94 ∴ Adequate
Factor of Safety for Strip 2	5.2/1.45 = 3.59 ∴ Adequate

The supporting manual calculations for these values can be seen Appendix I: Bearing Capacity Hand Calculations. Furthermore, *Foundation Design: Principles and Practices* by Daniel Coduto, provides an Excel worksheet which expedites the calculations for the values above, and also provides a means to confirm the values that were found by hand. The values obtained from this spreadsheet were exactly the same as those found by hand, confirming the accuracy of the results. An example of this spreadsheet can be seen in Appendix C: Bearing Capacity Excel Spreadsheet.

4.3 Abutment Design

The subsections below outline the results of the final abutment design that was recommended to the city for implementation in the park. The final design reflects the standards and methods for design of reinforced concrete structures per ACI Code.

4.3.1 Structure External Stability

Table 35 below summarizes the results from the abutment external design process. The results confirm the structures adequate factor of safety for all three design checks as well as its external stability against excessive loading conditions. These results can also be seen in Appendix M: Abutment Design Hand Calculations.

Table 35: Abutment Design Calculations

Overturning Check	
Overturning Moment	901.459 ft-lbs
Restoring Moment	184,572 ft-lbs
Factor of Safety Against	204.7
Adequacy	✓
Bearing Check	
Max Soil Pressure	1362.5 psf
Allowable Bearing Pressure	5200 psf
Factor of Safety	3.8
Adequacy	✓
Sliding Check	
Sliding Force	901.45 lb
Resisting Force	10,971.36 lb
Factor of Safety	12.1
Adequacy	✓

4.3.2 Structure Internal Stability

The internal design follows the standards and methods outlined by ACI Code for design of internal reinforcement for concrete structures. The design was completed for a 12-inch unit length of the wall. Table 36 below summarizes the results of the internal design process. The results indicate adequate factors of safety against the moment and shear forces subjected on the wall by the soil. The reinforcement was chosen based on the calculations for minimum reinforcement for the design as well as proper engineering practice for placement of internal reinforcement. The reinforcement requirements for connecting the new abutment to the existing foundation were derived directly from the standards outlined by the Hilti Manufacturer. The results can also be seen in Appendix M: Abutment Design Hand Calculations.

Table 36: Abutment Design calculations

Moment and Shear Check	
Max Moment	1443 ft-lbs
Max Shear	763.392 lb
Allowable Shear	3984 lb
Shear Factor of Safety	5.22 ∴ Do Not Need to Design for Shear
Constraints	
Effective Depth	3.5 in.
Cover	1.5 in. Everywhere
Reinforcement Design	
Vertical Reinforcement	#5's @ 10 in. O.C.
Temperature/Shrinkage Reinforcement	#4's @ 14 in. O.C.
Development Length for Dowels	24 in.
Development Length for Hooked Bars	10 in.
Existing Foundation Connection	
Embedment Length for Dowels	12.5 in.
Adhesive	HIT HY 200-R

4.3.3 Final Design

Figure 67 and Figure 68 below are elevation sections of the proposed cantilever abutment design detail. The sections are not drawn to scale but reflect the internal reinforcement that was designed for per ACI code. The reinforcement pattern is consistent throughout all walls of the abutment structure. Figure 69 shows the final 3D detail of the abutment design.

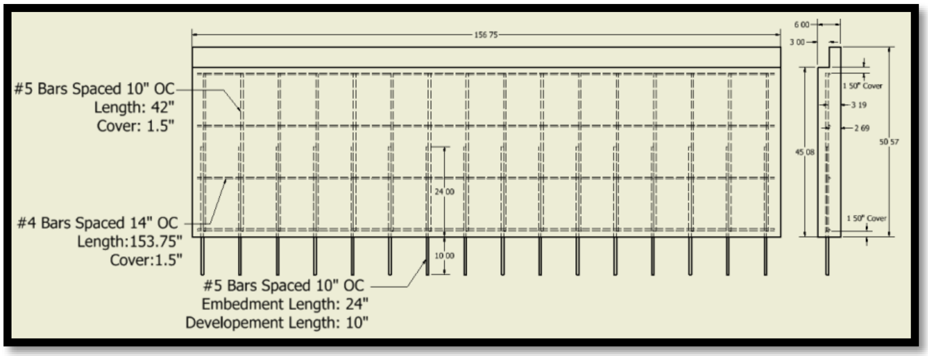


Figure 67: Abutment retaining wall detail

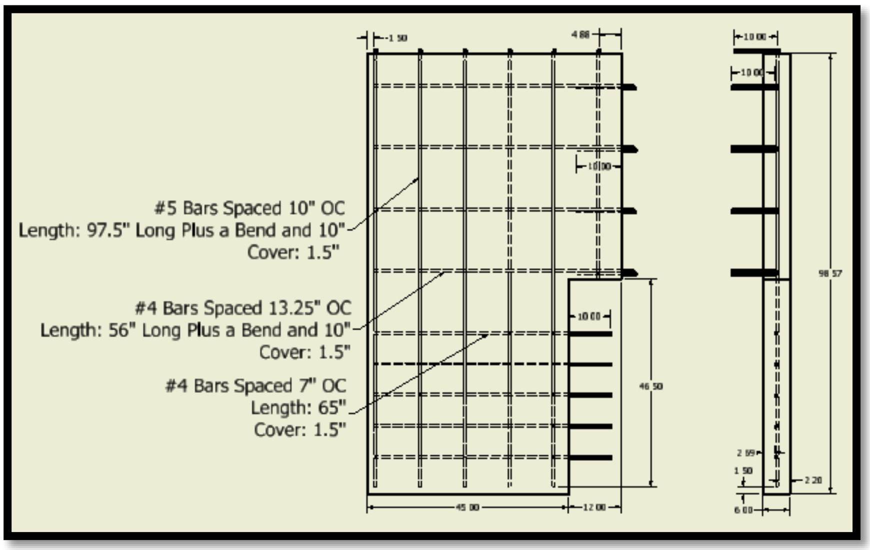


Figure 68: Abutment side wall detail

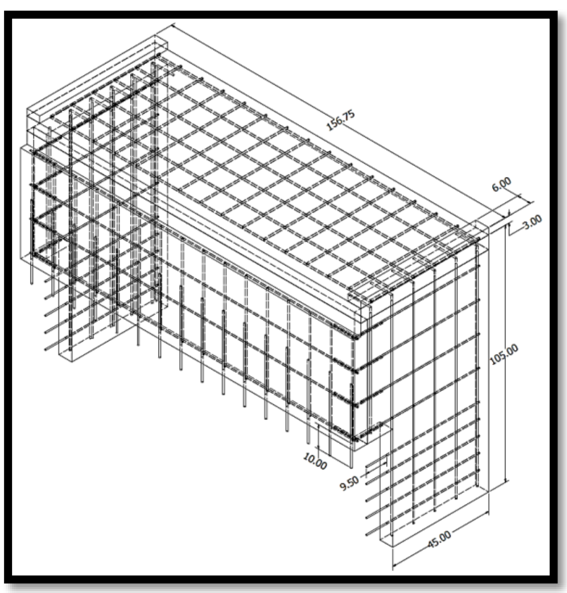


Figure 69: 3D Abutment detail

Figure 70 below is a realistic three dimensional model of what the abutment design will look like attached to the existing foundations in Elm Park. The structure will utilize steel dowels and construction grade adhesive that will fasten it to the existing concrete structures. The existing foundation will need to be treated and have holes drilled before attaching the new abutment.

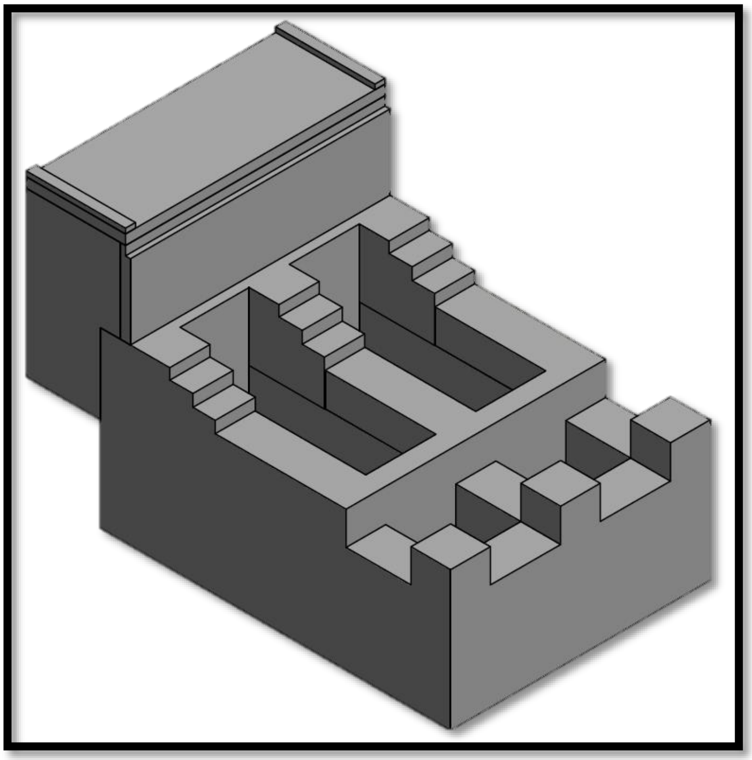


Figure 70: 3D Model of abutment design on existing foundation

4.4 Site Design

The site design was a major aspect of this project because it serves as the catalyst that will bring pedestrians to the bridge. Since the bridge was altered to meet ADA compliance, the height at the end of the bridge wings was elevated. Had the site not been altered around the bridge then there would be as significant drop off from the end of the bridge to the existing pathway. After having created multiple design iterations, a final design was chosen by the Worcester Parks Department. This design was then transposed into engineering plans that will be used in the field to implement the needed changes to the site. A basic cost analysis for the site was also completed.

4.4.1 Conceptual Design

From feedback provided by the Worcester Parks Department, the final conceptual design of the site was created. With a representative bridge, abutment, and landing put in place terminating at an elevation of 498 feet, the design of the surrounding paths and slopes were created. From previous design solutions, the paths leading up to the bridge landings were graded at a 1:25 (1 foot of rise for 25 feet of horizontal distance) which conforms to all regulations concerning grading of paths or walkways. On the either side of the bridge, the average elevation that needed to be met was 494 feet. On the Russell Street side of the park, the grading went along the path about 75 feet until it meshed with the existing site. The Park Avenue side however required the grading of three paths leading up to the bridge. Therefore a radial style of grading was used where concentric semi-circles of grading extended out about 100 feet from the bridge until it too meshed with the existing park elevations. On the sides of the landing and abutment adjacent to the bridge there was limited room to grade due to existing site conditions, which caused a significant drop off in height. Therefore, special attention was paid to these areas.

There were two slope requirements that existed for these areas that would not serve as pathways; the first was that placed stone would need to be utilized at slopes of 1:2. Maintainable slopes could also not exceed 1:3. In order to achieve even a, 1:3 slope on the South Mere side, the pond wall needed to be adjusted several feet further into the pond. On either side of the bridge the slopes facing the south mere required a slope of 1:2 extending from the pond edge wall up to the landing. This degree of slope would continue along the wall until transitioning to a 1:3 slope or less. This transition between placed stone and grass can be seen in Figure 71 below.

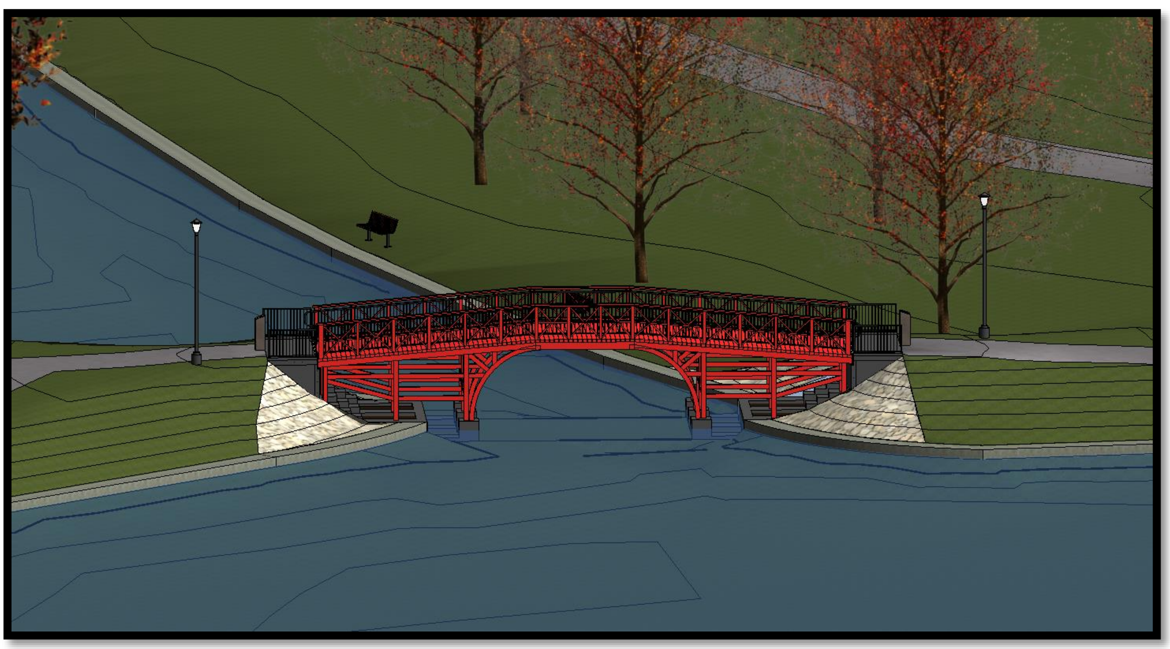


Figure 71: View from South Mere

The same approach for grading the earth around the bridge was taken on the both the Elm Mere and South Mere sides. The only difference being on the Elm Mere side there was extra space, therefore the grass slopes were able to be tapered down to a 1:4 slope, which is favorable for purposes of maintenance. A view of the slopes on the Elm Mere side can be seen below in Figure 72

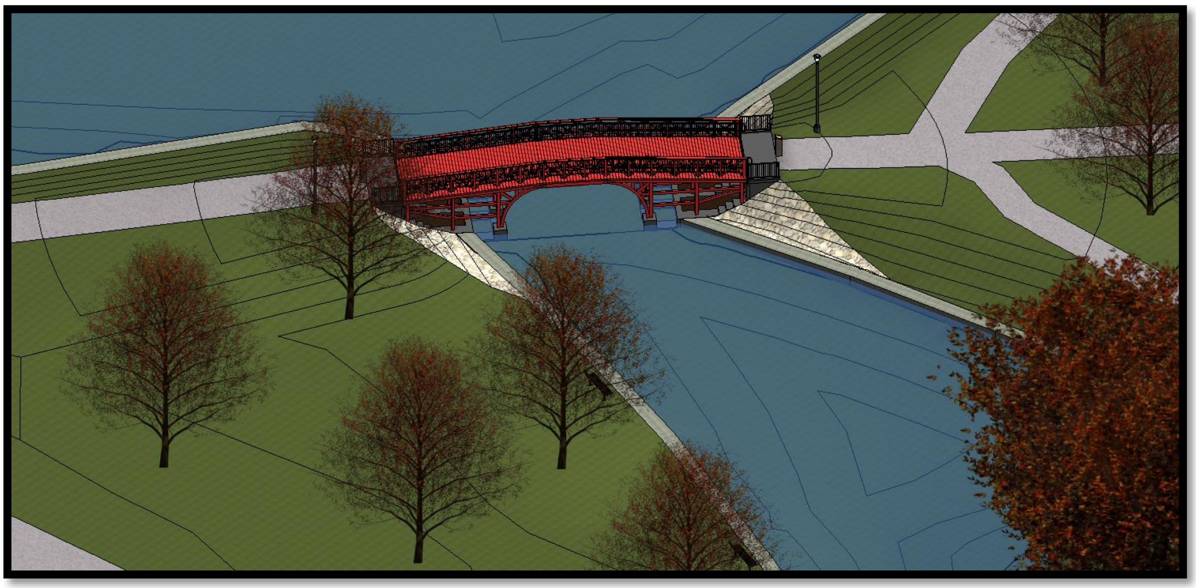


Figure 72: View from Elm Mere

With all of the necessary sites grading issues addressed, the next step was to consider some of the additional concerns that the Parks and Recreation department had expressed. The site required an impediment, known as a bollard, to stop vehicle traffic attempting to traverse the bridge and also a decision for a dedication plaque. These two concepts merged into one with a granite post centered on either end of the bridge at the edge of the landings, there by serving serving a dual purpose. As the post would not be removable, it could first serve as a bollard for impeding vehicle from crossing the bridge to either side. Second, aesthetically the granite post could serve as a dedication plaque to the late Myra Hiatt Kraft. A conceptual representation of the placement of the post can be seen in Figure 73 below. Additionally a picturesque image of the approach to the bridge from Russell Street can be seen in Figure 74.

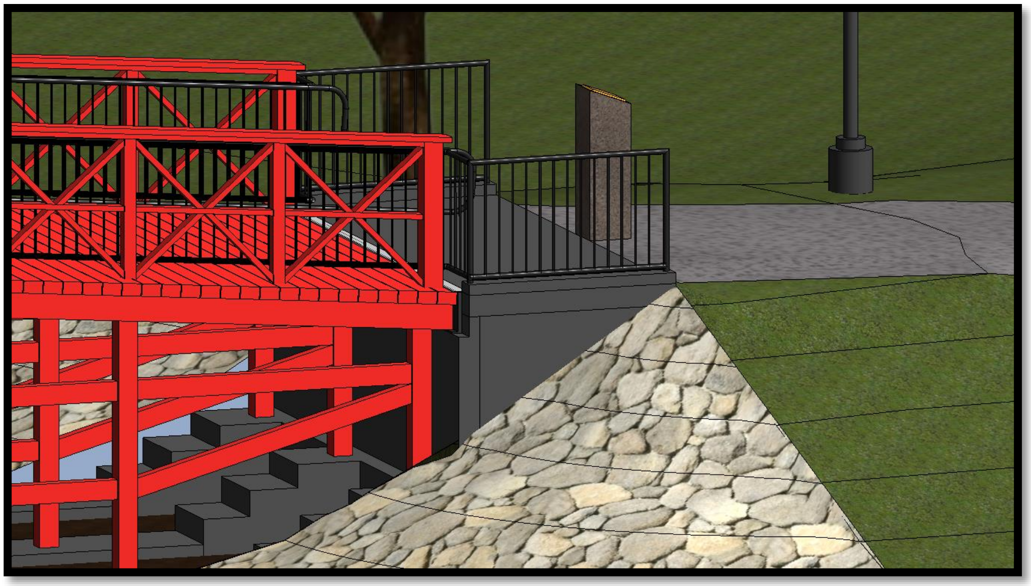


Figure 73: View of Transition from Bridge to Landing to sidewalk. With Granite Marker

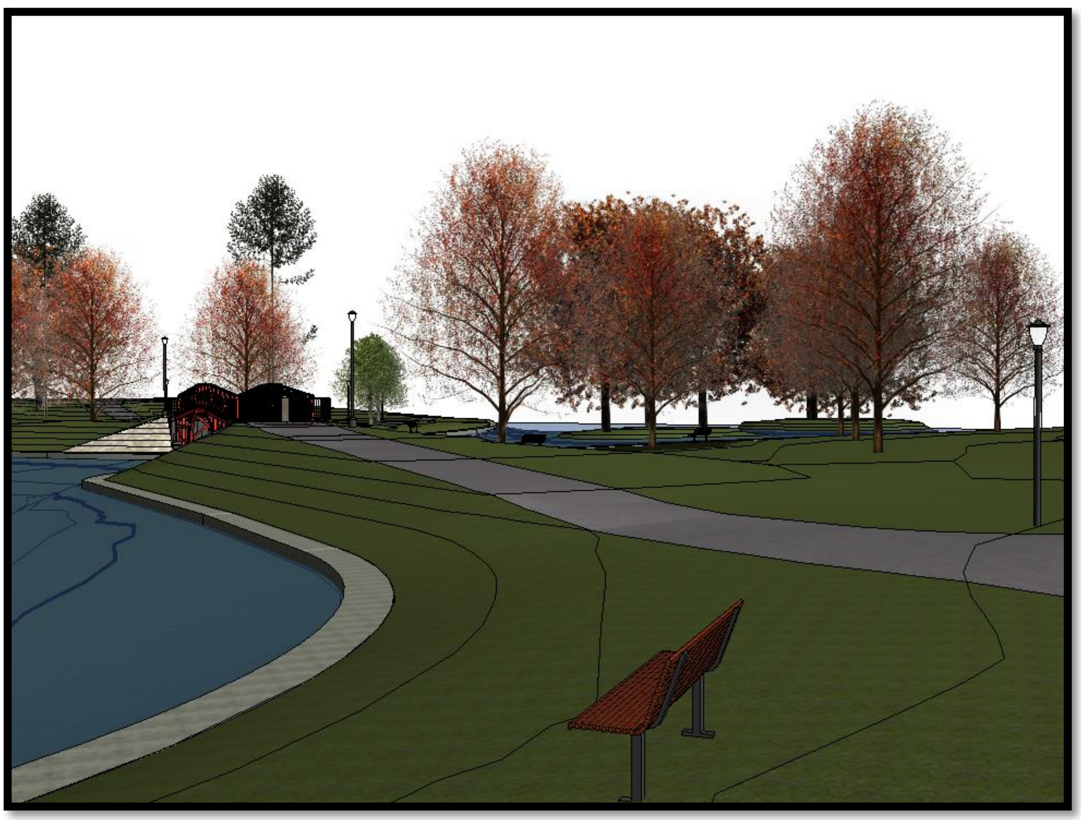


Figure 74: Approach to bridge from Russell Street

Another vital aspect of the site design process was the addition of pond walls around the South Mere pond edge. At this point, walls had only been installed in the North Mere and Elm Mere, with no plans for any pond walls to be constructed around the South Mere pond edge. This caused for very ambiguous pond boundary lines and a dilapidated appeal to the park. During background research it was found that pond walls did exist at one point around the South Mere, setting the precedence to incorporate them again in the park atmosphere. This would not only solve the issue of ambiguous pond boundaries, but also provide a point from which site grading could terminate. The walls help to more clearly define the different aspects of the park and will be of the same design that is seen in the walls that have already been constructed in the North Mere and Elm Mere. A detail drawing of these walls can be seen in Appendix N: Pond Wall Detail Drawing. Figure 75 below is a final rendering of the site design grading and additional pond walls.

Figure 75: Final rendering of site design

4.4.2 Engineering Plans

The conceptual design results were presented to the Worcester Parks Department and David LaPointe from Beal's & Thomas Inc. Once all parties had agreed that the design was acceptable, it was sent to Regan Harold at Beal's & Thomas Inc. Mr. Harold was able to take the proposed design and convert it to the engineering plans which can be found in the Appendix O: Elm Park Final Site Plans Due to the nature of the design incorporating the reduction of an area of the pond, it was presented to the Conservation Commission in March 2014. The Commission voted in favor of the design with very few questions.

4.5 Cost Analysis

The following section outlines the total costs the Elm Park Red Wooden Bridge project. The costs associated with the bridge structure, new abutment design, and site design are listed as well as total cost for the entire structure.

4.5.1 Bridge Structure

The price per linear board foot of each species depends on the supplier, availability, and dimensional cut. In fact, the market prices for lumber in stock may change on a day to day basis, according to Wiersma (D. Wiersma, personal communication, December 17, 2013). The cost amount is only valid for ten days but is challenged by competitor prices. Table 37 demonstrates the total cost estimate of the lumber material that was used for the new Myra Hiatt Kraft Footbridge. The ultimate cost amounted to about $5,334, plus the addition assumed ~$500 of the arch members, for a total of about $5,850.

Table 37: Cost estimate of lumber elements

Member Size	Quantity	Cost Per Member	Total Cost
6x6x16	6	$ 42.49	$ 255
6x6x12	12	$ 36.54	$ 438
4x6x12	117	$ 23.79	$ 2,783
4x6x8	24	$ 15.89	$ 381
4x4x16	1	$ 28.82	$ 29
4x4x12	11	$ 18.44	$ 203
4x4x8	8	$ 10.62	$ 85
2x4x16	1	$ 10.19	$ 10
2x4x12	48	$ 7.05	$ 338
2x6x12	10	$ 9.34	$ 93
2x6x10	2	$ 8.01	$ 16
6x10x12	3	$ 53.54	$ 161
6x8x12	12	$ 45.04	$ 540
		total sum=	$ 5,334

The other materials for this bridge are the connections. Based on the costs given by the Simpson Strong Tie suppliers, the following cost analysis could be created (Simpson Strong Tie, 2014).

Table 38: Connections cost analysis table

Connections	Product No.	Qty.	Price per	Total	Note
Column to Foundation	CPT66Z	24	$ 41.00	$ 984	
Beam to Column	CJT3	72	$ 33.10	$2,383	
Column to Decking	HL53	54	$ 27.31	$1,475	
Decking Supports to Arch	Custom Plate + Bolts	6	$ 75.00	$ 450	Shot drawings submitted to dealer and then to Simpson
Railing Column to Decking Support	Custom Plate + Bolts	64	$ 30.00	$1,920	
Decking Members	Nails - 8d, length=5in	678	$ 0.10	$ 68	For decking to support
Railing Members	Nails - 8d, length=3in	664	$ 0.10	$ 66	Railing members to each other
	Total			$7,346	
Inflation for shipping and handling				**$8,815**	

In addition to the direct materials cost for the bridge design are the costs of equipment and tools for the Worcester Technical High School. Joseph Lonergan, Carpentry Department Head of the Worcester Technical High School provided the following list of equipment and costs.

Table 39: Cost analysis of tools and equipment

Tool list for bridge

List was priced from Grainger catalog

3—1/2" drive electric DeWalt impact wrench. 4 JB72 $346.50

2—1/2'drive Hex impact Socket set 4PRH2 $ 189.70

2—10 ¼" Milwaukee circular saw 4NYEZ $ 436.00

4—3/4" Milwaukee D hand drill 6Z330 $1461.00

1—Milwaukee corded hammer drill 3TB72 $ 188.50

1—12" DeWalt compound miter saw 10D912 $ 783.50

2—Pole auger bits 13/16" 18" in length 6DLZ4 $65.90

4—Makita bit set 14F187 $ 140.0

TOTAL COST $ 3611.10

EQUIPMENT	
Equipment priced at	
Nationwide ladder	
23 Colton Street Worcester Ma 01601	
4 – Section of 5’ x 5’ Scaffold frames	$ 588.00
4- Guardrail sections for scaffold frames	$ 780.00
8- Planks to fit scaffold frames	$ 1750.00
16- Leveling jacks for scaffold frames	$ 432.00
3- 14” wide 16’ long alumni pick	$ 951.00
2- 16’ foot alumni extension ladders	$ 400.00
1-Set of out riggers for planks	$ 118.00
Total cost	$ 5019.00

The total cost for the bridge materials and the equipment for constructing the bridge adds to a total of about $23,300. But it must be noted that final prices will be determined by the bidding process.

4.5.2 Abutment Design

A cost analysis for the proposed abutment structures was completed using data from R.S. Means. Normal weight concrete and corresponding rebar sizes were used to complete the analysis. The costs of the HILTI adhesive for the connection to the existing foundation were obtained directly from the HILTI manufacturing website and a customer sales representative. These costs all reflect current market value for each material. Total cost estimation for the two abutments was estimated at $5695.85. Table 40 below summarizes the cost analysis that was completed for the proposed abutment. In addition, all detailed cost estimations can be viewed in Appendix P: Site and Abutment Cost Analysis.

Table 40: Abutment cost summary

Concrete	773.86 sqft. (6”-thick)	$2.03/sqft. (6”-thick)	$1570.94
No. 4 Rebar	522.32 L.F.	$0.71/L.F.	$370.84
No. 5 Rebar	1493.64 L.F.	$1.09/L.F.	$1628.07
Hilti Adhesive	2 Packs (50 tubes)	$1063.00/pack	$2126.00
		Total For Two Abutments	$5695.85

4.5.3 Site Design

Once site plans were created, with the help of Beal's and Thomas, approximate cut and fill requirements for the new grading surrounding the bridge were generated. There were two different sets of data to consider due to the fact that there was going to be cut and fill to generate the slopes leading up to the bridge and additional cut and fill requirements associated with the proposed pond wall edge in the south mere. The data that was provided for analysis also included fill that was already introduced to the park in another phase of construction. Therefore, the total cut and fills requirements and associated costs are over-estimates. Once a total net fill was calculated, costs for common fill on the R.S. Means website were researched. Only the material cost was considered in the analysis, exclusive of the extra costs associated with transportation or labor. Once an appropriate cost per cubic yard was estimated, a total cost for site grading could be determined.

The total amount of fill required for achieving the necessary slopes was calculated to be roughly 1,553 C.Y. (seen in Appendix Q: Site Design Cut and Fill Data). This may seem like a sizable amount of fill however it is important to keep in mind the idea that a 5-foot height difference between the Elm Park Red Wooden Footbridge and new Myra Hiatt Kraft Footbridge needed to be addressed. Furthermore, the number mentioned above is a very conservative over estimate of the actual amount of fill that will likely be required. Considering the cost for the actual soil, R.S. Means places a value of common fill soil at $28.00 per C.Y. Therefore the total calculated cost for the soil will be in the vicinity of $43,503.30. (Note that this is an over estimate because some fill has already been introduced to the site in other phases of the parks renovation). A detailed cost estimation for the site design can be seen in Appendix P: Site and Abutment Cost Analysis.

4.5.4 Material Costs

From all of these cost analyses, the total cost of the project was determined as shown below.

Table 41: Final cost analysis of project materials

Final cost of bridge materials and equipment	$22,500
Final cost of abutment design	$5695.85
Final cost of site work	$43,500
Total Cost	**$71,695.85**

5.0 Discussion

As mentioned in the methodology, many steps were conducted to obtain final results. Each piece of the project – bridge design, foundation analysis, abutment design, and site design – all had many intermediate results that lead to the final designs. The discussion that follows includes all the steps that were taken to obtain the final results for the project.

5.1 Bridge Design

One of the biggest considerations for this bridge was that it be accessible to all park patrons. The previous bridge had a slope that was difficult to traverse for a non-disabled individual and impossible for patrons in wheel chairs, with walkers, or who did not otherwise have the physical ability to climb the large slope. As such, the final bridge featured an ADA-compliant slope that would allow all patrons who wish to cross the bride the ability to do so. The maximum allowed ADA slope is 1:12 or approximately 4.75°. To be slightly more conservative and to allow for small errors in construction, the slope has been designed to be 4.7°. In order to reduce the slope, compensations had to be made elsewhere in the dimensions, resulting in increasing the height at the end of the bridge wings. While the previous height at the ends of the bridge was only about only 47 inches above the top of the foundation, the new design was 77 inches taller. This increase in 30 inches of height made the entire site renovation necessary to ensure that all park visitors can reach the bridge.

Another significant change from the previous bridge was the decrease the number of columns in a span from 4-double columns to 3-single columns per half span. This decreased the total number of columns from 24 to 18, which in turn removed all connections to this column, thus greatly decreasing the number of connections. Decreasing both number of members and number of connections minimized the amount of labor required to build the bridge as well as minimizing the cost of materials. The choice to go from double columns to single columns was for the longevity of the structure. From the conditions assessment, it was found that many of the failures occurred at the beam to column connections because they allowed water to pool, which lead to degradation of both the members and the connections. By removing these connections, the amount of rot at the connections was expected to be reduced and increased the lifespan of the bridge.

After creating multiple architectural designs, all parties involved were able to agree on one design that would lead to the most optimized version of the structure. The optimization would decrease excess labor, materials, and costs. In order to get this final engineered design the group had to verify the member strengths, the connection strengths, and the allowable deflections for all members. In order to find the specific member properties to compare to the member loads and deflections, the first step was choosing a material.

5.1.1 *Material and Coloring*

Timber is especially appropriate as a building material for a lightweight footbridge, despite its limits of span and carrying capacity, as compared to concrete or steel. Wood has strength qualities for resisting compression, tension, and bending. Utilizing structures such as arches allows for larger spans to be met. One challenge faced with timber was protecting it against rot and insect attack. Fortunately, possibilities are available for anti-rot treatment as well as choices of structure, shaping, and size.

Different lumber species were considered as a building material for the Myra Hiatt Kraft Footbridge. The *National Design Specification (NDS) for Wood Construction with Commentary* and the online Wood Database were the sources from which the wood specimen information was studied. Given that Douglas Fir-Larch was used for the Elm Park Red Wooden Footbridge, this wood species was used as a point of reference in researching alternative species. The NDS Manual was first used as a guide to learn about the physical properties of each species, as well as their availabilities for different dimensions and sizes. Then, the online Wood Database was used for additional information on the specific characteristics of each wood type, such as rot resistance, workability, and pricing. A table was prepared listing the lumber species with the most desirable chemical qualities and highest strength values. The species with the least preferable characteristics in terms of vulnerability to insect attack or low strength values were eliminated from consideration. Table 42 displays the list of lumber species that were considered as well as their advantages and disadvantages.

Table 42: Material Properties Table for Select Structural Grade Lumber

Lumber species	Modulus of Rupture (lbf/in2)	Elastic Modulus (psi)	Shrinkage: Radial (%)	Rot Resistance	Workability
Douglas-Fir	12,500	1,765,000	4.5	- Moderately durable in regard to decay - Susceptible to insect attack	- Typically machines well - Holds stains, glues, and finishes well
Red Pine	11,000	1,630,000	3.8	- Moderately durable to non-durable regarding decay resistance - Readily treated with preservatives	- Easy to work with both hand and machine tools - Holds glues and finishes well
White Oak	14,830	1,762,000	5.6	- Very durable	- Easy with hand and machine - Mediocre dimensional stability - Reacts with iron - Steam-bends well - Stains, glues, and finishes well
Pressure-treated Southern Yellow Pine	12,297	2,234,652	4.8	-Very durable especially against weather when it is pressure treated	- Good qualities for machining - Lots of resin which can clog abrasives used in sanding, -well suited for nailing, with very little splitting - Easily given a finish that can make it highly durable to minimize wear of the pine

After studying the physical properties of each timber species and consulting with wood experts, it was decided that southern yellow pine would be the recommended material for the bridge. Pressure-treated southern yellow pine was assumed to be provided by Koopman Lumber, Co. Inc., who supplied grades for the different member sizes. This treatment option is important for construction because it possesses properties that keep timber from rotting easily. Southern yellow pine is generally an easy material to work with using both hand and machine tools.

However, the arches of the new bridge design would pose a challenge. It was therefore suggested by the lumberyard head, Dave Wiersma, to cut arches out of a 2ft by 12ft sheet of timber and compile multiple layers together.

As mentioned, Koopman Lumber Co. Inc. was consulted to obtain member costs and grades. Though the procurement will go to bid and could end up being won by another company, the sizes and grades that their lumberyard had in stock were used, and thus will be the sizes and grades sent to bid. For this, the members sizes 2x4, 2x6, and 4x4 are all to be grade No. 1, and the member sizes 4x6, 6x6, 6x8, and 6x10 will be grade No. 2. This decrease in grade with increase in member size is common for many suppliers, and thus was followed for the design of the bridge structure.

Color plays an important role in the overall aesthetic effect of the footbridge. Long ramps with low grading usually look hefty and unappealing. In order to mitigate this effect, the footbridge should be painted in a color that can blend in with its surroundings. The rest of the structure can also be painted the same color in order to carry out the entire aesthetic scheme. The Elm Park Red Wooden Footbridge was also known as the Red Wooden Bridge because of its red color, which carried important historical aesthetic implications. However, other colors may be explored in order to highlight any important features that express the character of the new bridge design.

In order to maintain the historic elegance of the bridge, red was recommended as the remaining color for the new bridge. Paint alone would not be sufficient to prevent rot resistance, therefore the bridge members would also need to be stained. The Paint Department at the Worcester Technical High School would work with Worcester Parks and Recreation in order to find a suitable shade of red for the bridge as well as the different types of paint that would provide a sleek finish and some water resistance. Oil-based paint would be the best option and the bridge would be painted on site after it is completely built (Lonergan, 2014). The number of coats would be dependent on how the bridge looks after each coat. The expectation is that the paint would last for about two years and need minimal upkeep.

5.1.2 Strength of each member

In order to design the bridge, the allowable strengths were calculated using NDS design strengths for each member of the structure and the modification factors. The design strengths, F'_n, were calculated as the reference design values multiplied by a number of modification factors. The reference design values, F_n, were the strengths of southern pine under each loading condition, varying between axial, shear, or bending stresses, for example. The modification factors, C_n, were calculated for each member to reduce or increase the reference design values. These design values and factors are located in Appendix K: Bridge Member Calculations.

As the design strengths were based on geometry as opposed to loadings, corresponding members on each span of the superstructure used the same strengths. The design strengths for each member of one span of the bridge are also located in Appendix K: Bridge Member Calculations. With these design strengths a preliminary design was created, and then compared to the stresses obtained with the STAAD analysis.

5.1.3 Hand Calculated Design

Based on the design strengths hand calculations were used to choose the initial member sizes. The resulting member sizes from this were:

- 4x6 nominal beams
- 6x6 nominal columns
- 6x8 nominal diagonal decking support members
- 6x10 nominal flat decking support members
- 4x4 nominal cross bracing members

From the bending and deflection spot checks the decking size was allowed to remain the same as the previous 4x6 members. Although 2x6 members could hold the bending stresses, they would fail in deflection under maximum loading conditions. A full analysis can be seen in Appendix G: Results of STAAD Analysis. Based on the initial spot check analysis, the column sizes could be as small as 4x4 or 4x6. However, because this check was very simple, it was determined that the greater member size would be a better option because the member would undergo more than just compression forces. Another reason for choosing the larger sized members was the fact that the bridge was to be in a heavily used public park. The public's perception of safety as well as the abuse the bridge will take from graffiti or vandalism led to an over-engineered design of these members. Additionally, the best connection from the column to the foundation required a 6x6 or larger member, and the cost difference between the 6x6 and the 4x6 is not enough to justify the decrease in longevity of the bridge if another connection that was more susceptible to rot were chosen.

The beam sizes were also chosen by the connection sizes. In order to use the embedded connection, the minimum height of the beam was 6 inches nominally. This member was not engineered specifically but through the analysis of the whole bridge it was verified by STAAD results. The STAAD analysis results showed that the 4x6 member was able to withstand the loads it would be exposed to with the maximum loading. The final bridge substructure member size to determine was the cross bracing. Again, because it was an indeterminate structure, these members were unable to be analyzed using hand calculations, so the first design decision was to replicate sizes seen on the original bridge and run it through the finite element analysis software. After using the finite element analysis, it was determined that these members were well within the acceptable range. All hand calculated designs can be seen in Appendix R: Hand Calculation Results.

5.1.4 STAAD Strength Analysis

The hand calculations were calculated based on the assumption that select structural Southern Pine would be used. After speaking with the supplier it was determined that this grade lumber would greatly increase the timeline for the project, as well as the cost. With the new information from the supplier, a STAAD model was developed for the structure with the new lumber grades. Models and strength analyses of the bridge were performed several times until the model most closely represented the bridge structure and the member stresses it would feel. In each instance, the members of the bridge were analyzed using STAAD structural analysis software. During the original analysis some members were experiencing unexpected stresses, such as columns being put into tension, or incredibly high lateral bending moments, so the process of analysis was reconsidered. The issue with this was that the model did not contain the decking members, simply the dead weight of these members as loads. The final model contains these members, however the software shows them as being connected at the midpoint of the decking supports, not resting on top as is how the bridge will be built. This minor discrepancy a factor of safety check was completed to ensure there was an acceptable factor of safety. Final stresses from the STAAD analysis can be found in Appendix G: Results of STAAD Analysis.

5.1.5 Combined Stress Checks

In addition to comparing individual stresses to their respective limiting values, some stresses were assessed when combined with others. Specifically, tension and compression each needed to be checked with bending forces. These calculations were performed by using multiple interaction equations, four for tension and bending and one for compression and bending. While tension only requires two interaction equations to be checked, due to the three-dimensional loadings on this structure, two checks for bending in each equation were necessary about the *y*- and *z*-axes. When each equation was performed, the resulting ratios could not exceed 1.0.

The interaction equations yielded positive results. Due to all tensile stresses being well below their limiting stresses, all members loaded in tension and bending yielded acceptable ratios for the interaction equations. The horizontal decking support members came the closest to failing the interaction check, at 0.98, 0.84, and 0.79, but they were still acceptable. These high interactions were mainly the results of large primary bending forces compared to compression, as the great unsupported length of the member had a greater capacity to take on large moments. Confirming that the interactions did not lead to any failures was the last step in the design process of the structural members of the bridge

5.1.6 Deflection of Each Member

One final check on the bridge design was the serviceability of the bridge. In order to confirm the serviceability of the bridge, the deflections of some of the members needed to be identified. Only the deflections of the loaded members were checked, as they would deform significantly more than any of the members that receive any forces distributed from the loaded members. These members only included the decking support members, both the angled supports and the horizontal supports at the center of the bridge. The deflections were then checked with several serviceability criteria to determine whether the bridge deflection was within reasonable limits.

Under each criterion, the members deflected below the limit. The central support members deflected the greatest under the live and dead loads as they had the longest unsupported spans. Despite the great potential for inadequate deflections, the decking members provided some resistance to ensure that the members did not deflect too much.

5.1.7 Railing Design

Though the results show that the railing design is slightly conservative, this design was considered the best design for a few reasons. First, the park's history of vandalism must be taken into account, implying that the guards may be used beyond their normal capacity. A generous factor of safety of 3 was applied for many of these members to ensure that future maintenance is minimal. The addition of the internal railing will also help to deter vandalism. Because of the possible vandalism the end railing was made slightly more robust than the intermediate ones. Additionally, the bridge is in a park, and thus might be a structure that park patrons play and climb on, which could load the bridge beyond what the code requires; this is again a reason for an over conservative estimate in the design. Appendix S: Railing Member Calculations shows the NDS design strength values for each railing member. These values, when compared to the STAAD analysis results as can be seen in the results section, were much greater. The railing members, like the bridge members, all had factors of safety of 1.5 or greater.

The internal railings will be ordered directly from a manufacturer, who will submit to calculations for approval to the professional engineer (Thompson Fabricating LLC, 2014). The information to be sent to the supplier can be found in Appendix T: Railing Information for Manufacturer.

The choice to use a manufacturer over the Worcester Technical High School is one of constructability. Though the manufacturer's railing will cost more, the time saved by going to a supplier who has manufactured these types of railings for various projects will be a benefit to the projects expansive timeline. Ordering the railings rather than constructing them allows for the construction to finish as soon as possible.

5.1.8 Connections

Once the designs for both the structure and the railing were finalized connections were chosen that could support the loading requirements outlined.

5.1.8a Superstructure Connections

Criteria for selection of the connections were based on a few factors. Initially, the selection was based on the applicability of connections to the geometric constraints of the arrangement of the bridge's members. For example, where the columns connect to the bending members, most of the joints are perpendicular. This 90° angle allows for a very wide variety of connections to be placed. Conversely, where the short compression members connect to the arch, the options were much more limited. In this case, the angles at which the members connect are particularly specific to this project. Additionally, the arch would require a flat inset to be cut within it to hold the flat plate of the connection. These geometric constraints played a substantial role in the initial selection of connections.

Aside from geometry, a few other criteria needed to be assessed. Aesthetics were important to the preliminary selection to satisfy the client's desires. It was important that the connections be discreet so as to not take away from the visual appeal of the bridge, so concealed connections were desired where possible. Some concealed connections were suspected to not be strong enough to hold the greatest of the applied loads so they were not applicable in all cases. Moisture collection was also an important issue to consider. Water from rain and melting snow pooled on flat surfaces and within the double-shear connections leading to rot at the joints. This effect highlights the importance of moisture control in this design of the bridge. Plates where water could be trapped were avoided when selecting connections, as well as the use of nails, which withdraw much easier in moist conditions. Finally, the depth of pressure treatment was an important consideration. According to Mr. Harvey, connections should not penetrate through the pressure treated layers of the bridge (S. Harvey, personal communication, February 6, 2014). If the layer is penetrated, the member may be weakened, especially if moisture gets into the non-treated layers. This will not be avoidable when using bolts but should be considered when using nails and lag screws. In conclusion, aesthetics, moisture control, and depth of pressure treatment were additional criteria to consider for selecting connections before a quantitative analysis was performed.

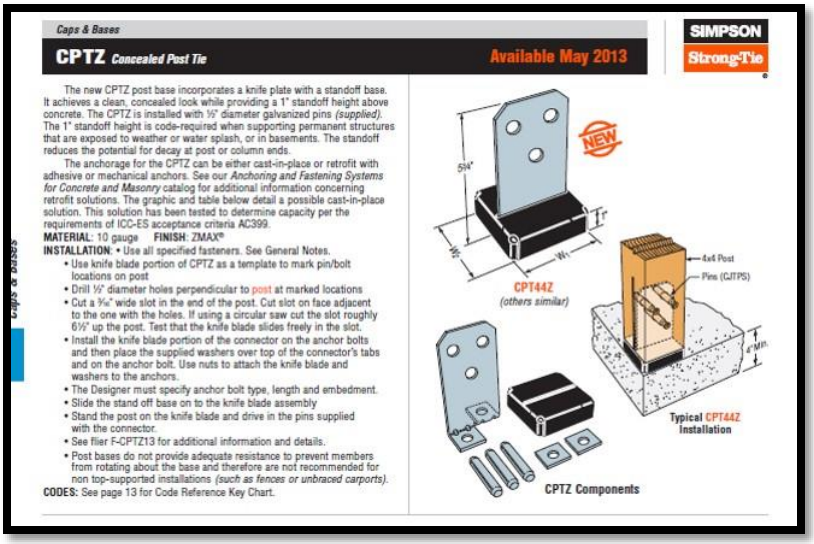
Before any quantitative analysis was performed for the connections, the type of connection first needed to be selected. From the advice of professional engineer Steve Harvey, the connections were selected from the manual of Simpson Strong-Tie. The selections were made with strength, aesthetics, and moisture control in mind. The following connections were selected for use within the superstructure:

- Concealed Post Tie (CPT66Z)
- Concealed Joist Tie (CJT3)
- Heavy Angle (HL53)
- Custom plate
- Nails

The subsequent sections describe the selection of connections and their application to the design in further detail.

5.1.8b Concealed Post Tie

For connecting the bases of the columns of the superstructure to the foundation, concealed post ties were selected. This connection satisfied each criterion for selection of connections at this joint. As with all connections, it was most desired that the connection be minimally visible to park visitors. The knife plate inserted through the center of the post would not leave any plates visible outside of the wood, only the connecting bolts. It was also important that the connection provide a means of separating the wooden members from the concrete foundation to prevent water from pooling and rotting the wood. As the connection included a shallow plastic block to place at the base of the column, a means of drainage was provided. Finally, the connections could not be cast in concrete as no more concrete was expected to be poured. This connection only required screws to attach it to the foundation, so no additional concrete was required. Aside from applying screws, the only additional work for construction of this connection is the cut through the center of the member parallel to the bridge span into which the knife plate is inserted. Overall, the concealed post tie was acceptable.


Caps & Bases

CPTZ Concealed Post Tie

Available May 2013

SIMPSON Strong-Tie

The new CPTZ post base incorporates a knife plate with a standoff base. It achieves a clean, concealed look while providing a 1" standoff height above concrete. The CPTZ is installed with 1/2" diameter galvanized pins *(supplied)*. The 1" standoff height is code-required when supporting permanent structures that are exposed to weather or water splash, or in basements. The standoff reduces the potential for decay at post or column ends.

The anchorage for the CPTZ can be either cast-in-place or retrofit with adhesive or mechanical anchors. See our *Anchoring and Fastening Systems for Concrete and Masonry* catalog for additional information concerning retrofit solutions. The graphic and table below detail a possible cast-in-place solution. This solution has been tested to determine capacity per the requirements of ICC-ES acceptance criteria AC399.

MATERIAL: 10 gauge **FINISH:** ZMAX®

INSTALLATION: • Use all specified fasteners. See General Notes.

- Use knife blade portion of CPTZ as a template to mark pin/bolt locations on post
- Drill 1/2" diameter holes perpendicular to post at marked locations
- Cut a 5/16" wide slot in the end of the post. Cut slot on face adjacent to the one with the holes. If using a circular saw cut the slot roughly 6 1/2" up the post. Test that the knife blade slides freely in the slot.
- Install the knife blade portion of the connector on the anchor bolts and then place the supplied washers over top of the connector's tabs and on the anchor bolt. Use nuts to attach the knife blade and washers to the anchors.
- The Designer must specify anchor bolt type, length and embedment.
- Slide the stand off base on to the knife blade assembly
- Stand the post on the knife blade and drive in the pins supplied with the connector.
- See flier F-CPTZ13 for additional information and details.
- Post bases do not provide adequate resistance to prevent members from rotating about the base and therefore are not recommended for non top-supported installations *(such as fences or unbraced carports)*.

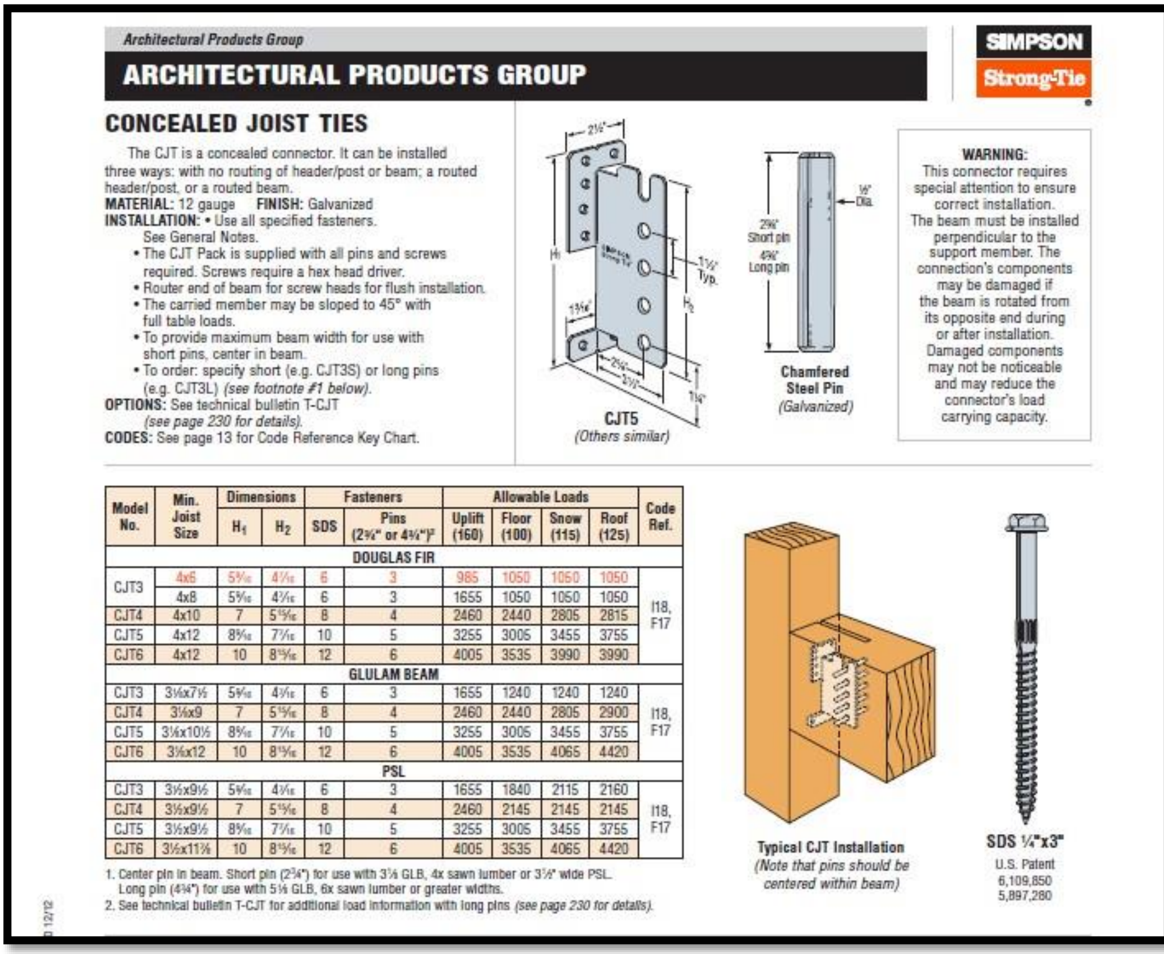
CODES: See page 13 for Code Reference Key Chart.

Figure 76: Concealed Post Tie (Simpson Strong-Tie Catalogue).

5.1.8c Concealed Joist Tie

Concealed joist ties were selected for connecting all 4x6 beams to the columns and arches. One tie would be placed on each end of each beam. As there were more of these joints than any other throughout the bridge, it was especially important that these connections were discrete. Aside from the connecting pins inserted through the beams, these connections would be fully obscured from view. This particular connection requires that members be joined perpendicularly, which may be an issue for a few of the beams connecting perpendicularly, especially to the arch. However, with careful assembly and properly cut members, this connection should be satisfactory for all connections to the columns.

One issue with this connection after construction is trapping of moisture. If the cut through the beam to fit the concealed plate goes through the member top to bottom, moisture may easily go inside through the top of the cut. If the cut can be made through from the bottom without coming fully through the member, moisture will be less likely to be trapped within the connection. However, if the beam is not flush with the adjacent columns, then water will still get trapped between the two members. An inset cut may have to be made into the column to place the portion of the connection to ensure that there is no gap between the column and beam where moisture may be trapped. This inset would be required in the arch to connect any of the adjacent 4x6s due to its curved surface. Careful construction is required for use of this connection to ensure that moisture trapping is minimized and that the strength of the connection is not compromised.



Architectural Products Group

ARCHITECTURAL PRODUCTS GROUP

SIMPSON Strong-Tie

CONCEALED JOIST TIES

The CJT is a concealed connector. It can be installed three ways: with no routing of header/post or beam; a routed header/post, or a routed beam.

MATERIAL: 12 gauge **FINISH:** Galvanized

INSTALLATION: • Use all specified fasteners. See General Notes.

- The CJT Pack is supplied with all pins and screws required. Screws require a hex head driver.
- Router end of beam for screw heads for flush installation.
- The carried member may be sloped to 45° with full table loads.
- To provide maximum beam width for use with short pins, center in beam.
- To order: specify short (e.g. CJT3S) or long pins (e.g. CJT3L) *(see footnote #1 below).*

OPTIONS: See technical bulletin T-CJT *(see page 230 for details).*

CODES: See page 13 for Code Reference Key Chart.

WARNING:
This connector requires special attention to ensure correct installation. The beam must be installed perpendicular to the support member. The connection's components may be damaged if the beam is rotated from its opposite end during or after installation. Damaged components may not be noticeable and may reduce the connector's load carrying capacity.

Model No.	Min. Joist Size	Dimensions		Fasteners		Allowable Loads				Code Ref.
		H_1	H_2	SDS	Pins (2¾" or 4¾")[2]	Uplift (160)	Floor (100)	Snow (115)	Roof (125)	
DOUGLAS FIR										
CJT3	4x6	5 9/16	4 3/16	6	3	985	1050	1060	1050	I18, F17
	4x8	5 9/16	4 3/16	6	3	1655	1050	1050	1050	
CJT4	4x10	7	5 15/16	8	4	2460	2440	2805	2815	
CJT5	4x12	8 9/16	7 3/16	10	5	3255	3005	3455	3755	
CJT6	4x12	10	8 15/16	12	6	4005	3535	3990	3990	
GLULAM BEAM										
CJT3	3⅛x7½	5 9/16	4 3/16	6	3	1655	1240	1240	1240	I18, F17
CJT4	3⅛x9	7	5 15/16	8	4	2460	2440	2805	2900	
CJT5	3⅛x10½	8 9/16	7 3/16	10	5	3255	3005	3455	3755	
CJT6	3⅛x12	10	8 15/16	12	6	4005	3535	4065	4420	
PSL										
CJT3	3½x9½	5 9/16	4 3/16	6	3	1655	1840	2115	2160	I18, F17
CJT4	3½x9½	7	5 15/16	8	4	2460	2145	2145	2145	
CJT5	3½x9½	8 9/16	7 3/16	10	5	3255	3005	3455	3755	
CJT6	3½x11⅞	10	8 15/16	12	6	4005	3535	4065	4420	

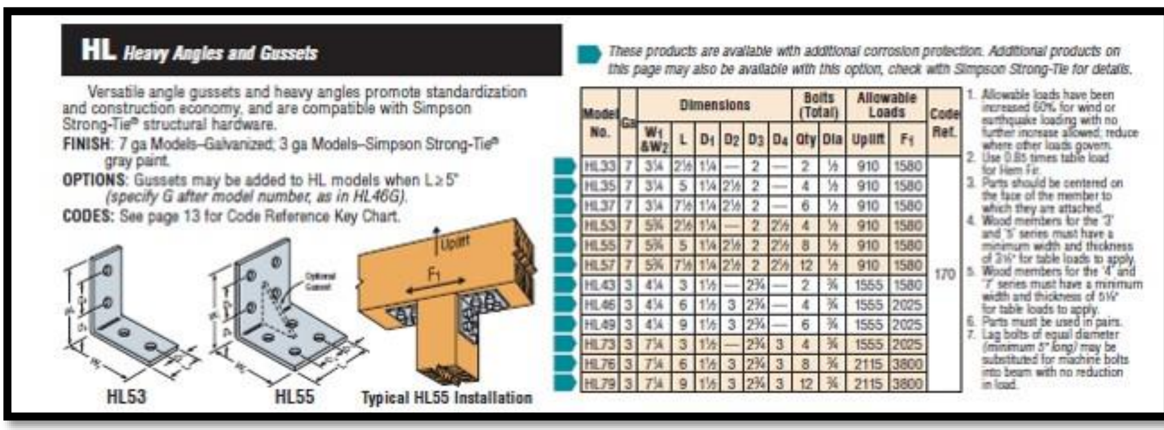
1. Center pin in beam. Short pin (2¾") for use with 3⅛ GLB, 4x sawn lumber or 3½" wide PSL. Long pin (4¾") for use with 5⅛ GLB, 6x sawn lumber or greater widths.
2. See technical bulletin T-CJT for additional load information with long pins *(see page 230 for details).*

Figure 77: Concealed Joist Tie from Simpson Strong-Tie Catalogue

5.1.8d Heavy Angle

The heavy angle was selected to connect members to the angled decking support members (12-…-17, 31-…-36, 50-…-55, 69-…-74, 88-…-93, and 107-…-112). At the top of the outermost columns, one heavy angle will be placed on the inner side of the joint with the decking support member. For the middle columns on each end of the span, one heavy angle will be placed on each side of the column at the joints. For the inner columns, adjacent to the arches, one heavy angle connection will be placed between the column and the decking support member and another between the column and the adjacent chord (15-18, 34-37, 53-56, 72-75, and 91-94). Another would be placed between that chord and the decking and finally one more would be placed on each side of the next shorter chord **(**16-19, 35-38, 54-57, 73-76, and 92-95**)** where it connects to the angled decking support member.

As opposed to the previous connections, these were selected based on different criteria. As they are to be located directly beneath the decking supports, moisture trapping within the metal plates was not as great a concern. However, the geometry of the members was more restricting here, requiring this connection. Specifically, none of the angles involved in this connection were orthogonal but this connection could be adjusted to various angles as needed. As the members are to be flush with one another, rather than offset, a bolt could not simply be driven through one column into the decking support. While a bolt could be driven down through the decking support into the column, the end-grain connection through the latter could be significantly weaker than connections perpendicular to the grain. Despite the geometric advantages of the connections, there may be difficulty in construction when assembling at tight angles such as where the innermost column and the longer chord connect. Aesthetically, these connections are more exposed than the concealed connections, but they will be covered with paint to disguise their appearance. Although the angles are more visible than other connections, they would fit the geometric needs of the joints appropriately.


HL *Heavy Angles and Gussets*

Versatile angle gussets and heavy angles promote standardization and construction economy, and are compatible with Simpson Strong-Tie® structural hardware.

FINISH: 7 ga Models–Galvanized; 3 ga Models–Simpson Strong-Tie® gray paint.

OPTIONS: Gussets may be added to HL models when L ≥ 5" *(specify G after model number, as in HL46G).*

CODES: See page 13 for Code Reference Key Chart.

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

Model No.	Ga	Dimensions						Bolts (Total)		Allowable Loads		Code Ref.
		W_1 & W_2	L	D_1	D_2	D_3	D_4	Qty	Dia	Uplift	F_1	
HL33	7	3¼	2½	1¼	—	2	—	2	½	910	1580	170
HL35	7	3¼	5	1¼	2½	2	—	4	½	910	1580	
HL37	7	3¼	7½	1¼	2½	2	—	6	½	910	1580	
HL53	7	5⅝	2½	1¼	—	2	2½	4	½	910	1580	
HL55	7	5⅝	5	1¼	2½	2	2½	8	½	910	1580	
HL57	7	5⅝	7½	1¼	2½	2	2½	12	½	910	1580	
HL43	3	4¼	3	1½	—	2¾	—	2	¾	1555	1580	
HL46	3	4¼	6	1½	3	2¾	—	4	¾	1555	2025	
HL49	3	4¼	9	1½	3	2¾	—	6	¾	1555	2025	
HL73	3	7¼	3	1½	—	2¾	3	4	¾	1555	2025	
HL76	3	7¼	6	1½	3	2¾	3	8	¾	2115	3800	
HL79	3	7¼	9	1½	3	2¾	3	12	¾	2115	3800	

1. Allowable loads have been increased 60% for wind or earthquake loading with no further increase allowed; reduce where other loads govern.
2. Use 0.85 times table load for Hem Fir.
3. Parts should be centered on the face of the member to which they are attached.
4. Wood members for the '3' and '5' series must have a minimum width and thickness of 3½" for table loads to apply.
5. Wood members for the '4' and '7' series must have a minimum width and thickness of 5½" for table loads to apply.
6. Parts must be used in pairs.
7. Lag bolts of equal diameter (minimum 5" long) may be substituted for machine bolts into beam with no reduction in load.

Figure 78: Heavy Angle connections.

5.1.8e Custom Plate

In order to fit the needs of the most complex joints of the bridge, a custom plate needed to be ordered. This joint was where the top of the arch meets the two decking support members. None of the standard beam-to-column plates were appropriate here due to the angles at which the three members meet. Bolts could be driven down from the decking supports through the arch but these could only connect two members at a time, leaving the joint less secure. Due to the horizontal decking support members being subject to such great bending moments, the joint needed to be able to resist rotations without fully distributing them onto the arch. With bolts driven laterally through the members and connected by a custom plate, the members would be fastened together to produce a cohesive joint. In terms of aesthetics, the plate would be revealed if placed on the exterior but also could be hidden by a coat of paint or hidden on the inside of the span.

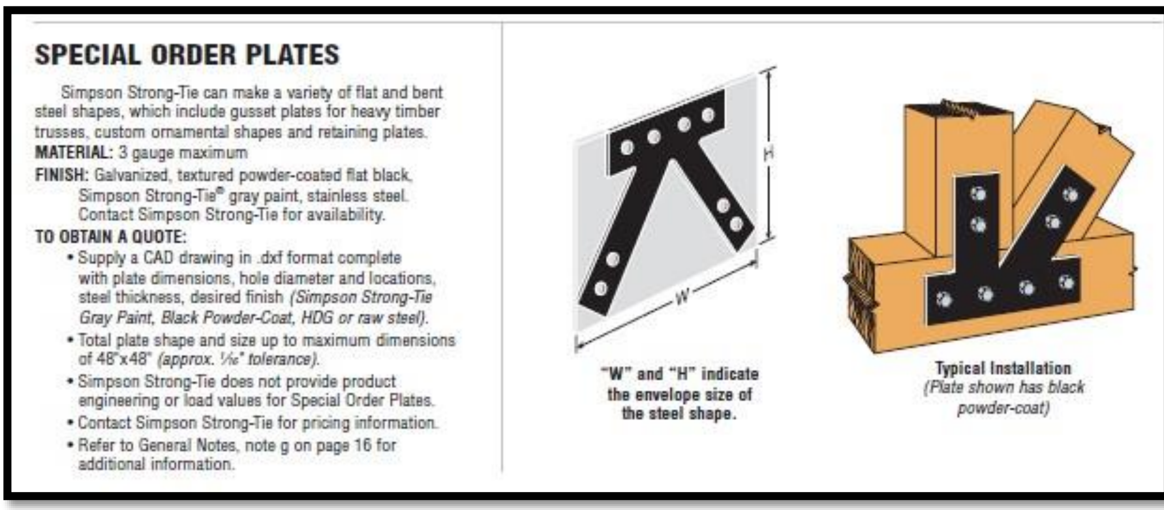


Figure 79: Example of a special order plate (Simpson Strong-Tie Catalogue)

5.1.8f Nails

The final connection needed for the bridge was nails. These were used to fasten the cross-bracing members between spans to their respective columns. As the cross bracing was intended to hold the spans together rather than carry any significant loads, stronger connections like bolts were not deemed necessary. The number and size of nails was selected according to the IBC schedule of nails with the cross bracing considered to be bridging (International Code Council, 2014). The nails would be concealed within the structure as only the head could be outside of the wood, although the cross-bracing members themselves are also relatively concealed. Finally, although nails are weakened significantly in withdrawal by moisture, the nails at the bottom of the bracings would be directed upwards so water would not be able to get into the hole at the location of the nail. The nails at the top of the bracings may be protected from water by the decking directly above them also. Nails will be applied sparingly on the bridge due to their relatively low strength but will be satisfactory for the needs of the bracing.

5.1.8g Quantitative Analysis

A quantitative analysis of the strengths of the selected connections was performed to ensure that their strengths were adequate to withstand the loads on the bridge. Each type of connection was analyzed according to NDS design procedures for finding the shear strength, *Z'*, and withdrawal strength, *W'*, where applicable. The shear strengths were determined for each potential failure method of the connections and then the weakest failure method dictated the strength of the connections. For each connection, the geometry used was defined in the Simpson Strong-Tie catalog while strengths of the wood and dowel connections were provided by the NDS. Overall, eleven different joints were considered.

- Connection of foundation to column
- Connection on beams and chords
  - Beam to column, perpendicularly
  - Beam to column, at an angle
  - Beam to arch
  - Chord to arch, two separately
- Connection on columns
  - Column to beam
  - Column to decking support
  - Column to cross-bracing
- Connection of decking support to column
- Connection of angled and horizontal decking supports to arch

Although only five different types of connections were used throughout the bridge, several connections were considered for different configurations in which the members connected were oriented differently. The geometry was influential in determining the failure method for each connection. Failure methods for each type of connection are shown below. For each of the connections at the considered joints, the number of dowels used was to be based on the shear strength of the connection at the weakest failure method.

Table 43: Failure methods for each type of connection.

Fnd'n to Col.	Col. to Beam ⊥	Col. To Beam θ	Beam to Col.	Col. To Deck Sup	Deck Sup to Col.	Arch to Beam 11	Arch to Beam 20	Arch to Beam 21	Arch to H Deck Sup	X-Sups	H-Sups	Decking
I_m	III_s	I_m	III_s	III_s	III_s	I_m	I_m	I_m	III_s	IV	IV	IV

In addition to the shear strength of the connections, the withdrawal strengths were also considered. The withdrawal strengths were applicable to a few connections, specifically those involving nails and screws; bolts will not withdraw as they are secured on both ends by the head and the nut. The withdrawal strengths were used to check if the forces in the members would be enough to wedge a screw or nail out of place.

Next, the number of bolts or nails was selected for each connection. For most connections, the number of dowels was defined in the catalog based on the geometry of the members. For example, the welded knife plate for the connection to the foundation would have three pins regardless of the size of the member to which it is to be attached, while the concealed joist tie required a number of pins dependent on the size of the connected members. Conversely, the heavy angles connecting to the decking supports can vary in the quantity of bolts that are used, as long as there are enough to carry the loads at the connection. Appendix H: Connection Calculations summarizes the results from the connections strength checks, including the shear and withdrawal strength values, the number of each connection that may be used, and the total force that may be resisted by each connection, *P*. While the hardware was designed to hold the specified number of dowels, it was important to confirm the strength of each connection.

Each connection was analyzed for the maximum load that it may take throughout the structure to identify the number of bolts or pins it may require. For the concealed post tie and concealed joist ties, it was confirmed that the standard number of dowels used in each connection would hold the applied loads. For the heavy angles, one row of two bolts on each side of the angle would be sufficient. If applied in double shear, with one bolt through a column or chord with part of an angle on each side, the connection would be especially strong enough to resist the loads. The single shear connection of the angle with the bolt going through the decking support is weaker than the double shear connections but would still carry the loads with a factor of safety. An angle with only one bolt on each side of the fold could suffice but an extra bolt was recommended to eliminate any potential rotations of the angle around the bolts. Finally, the nails on the cross bracing were also sufficient. The members do not transfer a large amount of load between the spans of the bridge so only one nail could theoretically be used safely, but typical toe-nail installation uses two nails at a connection. While the connections may have been overdesigned, the hardware provided limited the freedom to reduce the quantity of connections and a factor of safety was important to apply to ensure the stability of the structure.

After the strengths of the dowels were compared to the forces in the members to ensure that the connections were satisfactory, the design of the connections was finalized. Overall, five different types of connections were required for use in the bridge. These connections required four different types of dowel connections to fasten the members and plates together: bolts, pins, screws, and nails. The calculations for assessing the connections is shown in Appendix H: Connection Calculations. When properly fastened to the constructed members, the bridge should be able to safely resist all typical loadings applied throughout its lifetime without failure.

5.2 Foundation Analysis

The foundation analysis portion of the project was arguably one of the more ambiguous aspects due to some unknown field conditions and the unique geometric shape of the existing foundations. However, these results are very important as they dictate the foundations' ability to support the new bridge structure. From a safety standpoint, it was vital that a conservative approach for the foundation analysis be taken.

The purposes of the visual and structural evaluations were to determine the capacity of the Elm Park Red Wooden Footbridge's foundations to support the new Myra Hiatt Kraft Footbridge that would take its place. There were some challenges the project group faced as part of the analysis. Unfortunately, the original plans of the concrete foundations from 1972 could not be obtained from the City. Due to a restricted ability and lack of resources to conduct any in depth examinations of the foundations such as soundings, invasive pluming tests, and geophysical investigations, it was difficult to accurately determine the chemical makeup and proportioning of the concrete mix that was used, as well as if any reinforcement was used in the existing foundations. Furthermore, it was difficult to estimate how deep the existing foundations penetrated into the underlying soil. This made it especially problematic to estimate exactly how much weight contributed to the bearing pressure exerted by the foundation on the soil. Conservative and reasonable assumptions were made throughout the foundation analysis process to estimate a final bearing capacity for the foundations in Elm Park.

The assessment of negative phenomena, or evidence that movement had occurred after the original construction was completed, could not be conducted at the time of the evaluation due to the fact that the bridge had already been partially disassembled and relocated to the Worcester Technical High School. The bridge was analyzed initially in the park, and after it was moved to the high school.

The damage and mold that was found can most likely be credited to substantial Worcester weathering over the past 40 years, as well as the accumulation of pond water in the hazardous locations. Ponding occurs when water accumulates on horizontal structures and cannot dissipate due to limited or restricted runoff capabilities (FPA, 2013). Given the bridges proximity to the pond, it was very susceptible to ponding, especially at the mortar bearing pads. Mortar is not as strong as concrete and can fall victim to weathering over extended periods of time. The mold however can be considered purely superficial and more closely related to the aesthetic features of the foundation. Therefore it is vital that the mold be removed and the foundations be cleaned extensively before the new bridge is constructed. It would assuredly be an eye sore to have a new refurbished bridge and a dirty and seemingly neglected foundation.

Again, there was no access to any original plans of the foundations, so it was unknown if both foundations were designed to be exactly identical with equivalent measurements. It is important to note however, that the foundations were essentially symmetrical from a bird's eye view and were more than likely designed as such. Upon inspection and measurements, it was clear both the East and West foundations were at least designed to be exactly the same and the slight differences of certain measurements could be attributed to weathering over the last forty years. The only measurement that could not be acquired was the depth of the foundation blocks, as it was not feasible to dig around the foundation to its ultimate depth and would only be detrimental to the foundation itself. It was assumed that the depth was about 6 feet.

Bearing capacity was an extremely important facet of the overall existing foundation assessment as it indicates the underlying soil's ability to withstand the loads imparted by both the existing foundation and the new bridge structure that will be constructed on top of these foundations. It was important to consider the many different ways that the bearing capacity could be evaluated given the existing circumstances for the site and the foundations, as well as the different assumptions that needed to be made in order to obtain the best possible estimation of the aforementioned indicator. Considering best practices for obtaining a reasonable estimation for bearing capacity, and considering the lack of available plans for the foundations that were poured in 1972, a reasonable methodology was adopted and implemented in order to calculate appropriate bearing values.

Before conducting the bearing capacity analysis of the foundations, it was important to first investigate the subsurface conditions below the foundations themselves. The soil properties below the foundations directly dictate the value of the ultimate bearing capacity of the soil relative to the foundations. Soil borings were taken adjacent to the foundation locations and it was found that peat and clay layers of soil were located at depths below the existing foundations. These layers are inherently incapable of sustaining large bearing loads. Therefore, it was reasonable to assume that those layers beneath the foundation were excavated in 1972 and replaced with more adequate structural fill. This was an important consideration for the foundation analysis considering there was no way of determining for certain if these conditions were actually the case.

Rather than considering the foundations as massive solid concrete blocks, the analysis approach that was chosen considered the foundations as three separate but similar concrete strips, which extended about 6 feet into the underlying soil. The perpendicular strips connecting the three main strips could be considered negligible to the bearing capacity of the soil. This simplified the analysis process but was also conservative given the magnitude of the loads the foundations will be sustaining. Strips one and three produced similar pressure distributions due to the symmetrical distribution of loads imparted by the bridge structure on the foundation. Normal weight concrete was assumed for existing foundations.

The results of the foundation analysis confirm that the assumed soil type is more than capable of sustaining the structural loads imparted by the foundations and the new bridge structure. Again, it was difficult to determine the exact bearing capacity of the underlying soil because it was unknown how deep the foundations penetrated due to lack of available plans from the foundations were poured and if the underlying peat and clay layers were excavated as part of the construction process in 1972. It is recommended that additional tests be completed to assess the exact site conditions underneath the existing foundation and the conditions of the foundation itself.

5.3 Abutment Design

The abutment design portion of the project was extremely important as it served as an intermediary to connect the site design and bridge design portions. Both the external stability and internal stability of the structure needed to be adequately addressed to yield a final design that could sustain the representative loads, weather, and soil conditions in Elm Park.

There were multiple challenges and considerations the team faced as part of the abutment design process. First and foremost, it was important to design an abutment that could successfully be attached to the existing foundations. It was decided early on in the project that the existing bridge foundations would be used for the new bridge. Therefore, once the height of the new bridge changed, it necessitated an abutment be built that served a multi-purpose roll: one, to act as a retaining wall and prevent spill-over of soil into the existing foundations, and two, to accommodate for the change in elevation from the old bridge to new bridge. The group needed to research multiple options for attaching concrete structures to each other, and it ultimately became an important consideration in the design process.

Second, it was important to design an abutment that was both structurally stable and cost efficient. At the beginning of the project, it was still unknown if the new bridge would be made ADA compliant or not. Therefore, a new abutment was not considered as part of the city's plans to reconstruct the Elm Park Red Wooden Footbridge. When it was decided that the bridge would in fact be ADA compliant, it necessitated an additional abutment structure be built, and thus it would incur greater project costs. As part of the design process, it was the project team's goal to be conservative in the amount of additional material needed to design the new abutment piece. This is reflected in the final design.

The preliminary design considerations mentioned in the methods section were adapted throughout the design process for the new concrete abutment. The design for protection against external stability was emulated in a manner similar to the examples seen in *Design of Concrete Structures: Second Edition* by Arthur Nilson for design of reinforced concrete retaining walls. The external stability of the structure was checked for overturning, bearing capacity and sliding failures, as these were the most important modes of failure that the new structures could experience in Elm Park.

Upon confirming the external stability of the new concrete abutments, the internal stability was then designed in a manner that was both structurally stable and economical. The design process for the internal reinforcement of the structure was adapted from examples seen in *Design of Concrete Structures: Second Edition* by Arthur Nilson, and utilized the approach of designing for a single unit length of wall, and then replicating the internal reinforcement throughout the length and height of the walls and landing. This was a conservative approach, but acceptable given the fact that the magnitude of the loads on the landing above will be relatively small in nature compared to larger retaining wall projects.

The two design options proposed for the concrete abutment were pitched to the relevant stakeholders involved in the project, including the Worcester City Manager's office, the Worcester Parks and Recreation Department, and Steve Harvey, the structural engineer signing off on the final design documents. After extensive discussion with the aforementioned parties and consideration of their feedback, design option two was chosen as a final recommended design.

Design option two was the most economical option presented and utilizes a far less amount of concrete material than design option 1. The option could also potentially incorporate the use of helical piles for additional support, especially considering the varying and unclear soil profile beneath where the abutments will be installed. These helical piles would transfer the loads exerted on the abutment structure to the bedrock below the surface. In this way, they could serve as additional foundation supports that ensure the structural integrity of the designed abutments. However, these helical piles were not represented in the final abutment design, and should only be considered as an additional option to explore depending on actual field conditions and the final assessment of the professional engineer.

5.4 Site Design

The final conceptual design and engineering plans presented in the results section were the culmination of several iterations of designs. The earlier designs focused on solving the problem of soil erosion around the foundation of the bridge and the slopes of the approaching walkways. These designs included lengthy retaining walls that had an overall negative impact on the look of the park. The Parks Department was not pleased with designs including retaining walls and suggested that the pond edge wall be moved further into the pond to make a more gradual slope.

With permission to develop a design that included pond edge wall relocation, a new design was created. This design had discontinuities in the slopes and would not be feasible for upkeep. The next step was to figure out how to combine mowing capabilities, and erosion control on the slopes. A system of placed stones for slopes too great for mowing was put in place for erosion control and that was meshed with slopes that could be navigated by a mower.

The final design incorporated aspects from each iteration of designs from beginning to end. From the first design almost nothing was kept the same. The only thing that stayed was the way in which the walkways were graded. At 1:25 slopes they were more conservative than the 1:20 slope requirement, which addresses the potential for miscalculations in the field. The position of the pond edge wall was only moved a few feet and it stayed in this position through multiple design iterations. It was important that the wall not be moved too far out into the pond, because the area that was filled behind the wall needs to be made up somewhere else around the pond. Once the maximum slopes for placed stone as an erosion deterrent, and the slope for mowing were determined, the pond walls were immediately integrated into the design, and didn't change.

Once the final design was approved it was sent to Beal's & Thomas who then made minor alterations and created the final engineering plans. The changes were related less to building codes but more toward constructability. They produced the engineering plans and included a basic cut and fill analysis. On the plans they also indicated where several trees will need to be removed in order to meet the grading requirements. The final engineering plans and design were pitched to the city of Worcester and other relevant stake holders who expressed their approval.

5.4.1 Lighting

Though not part of the final engineering plan, part of the site design is the new lighting for the park around the bridge. Interesting and new lighting options were taken into account for the Myra Hiatt Kraft Footbridge. For better illumination at night, projectors can be placed under the bridge at the level of the surface concrete footing. This would illuminate the bridge in a discreet and visually appealing way.

Another options for lighting would be putting LED lights inside the railings of the bridge itself. These lights would shine through the bottom of the rails and light up the deck and outer parts of the bridge. This option is a viable option because it would not add a substantial amount of extra weight to the bridge; only five extra pounds would be added to each of the railings. For the structural aspect this would not change the loadings on the bridge by any significant number. Although this option is viable there is a problem with putting LED lights under the handrails. The cord that would need to attach the lights to a power source would have to somehow go through the bridge and into the ground. Another issue with this would be the fact that the lights could be tampered with easily and the maintenance for them would be very high. If one light were to be tampered with and go out it would take a lot of effort and money in order to replace the blown out lights.

The last option that was looked into was keeping the bridge the way it is and putting lighting around the ends of it. There would be a lamppost at each end of the bridge that would produce enough light to illuminate the trail and the entire bridge. This option would be the most ideal because these types of lampposts are already being used in the park. Another good thing about this type of lighting is that they are already tamper proof, and would need a lot less maintenance. Since these are already being used throughout the park the availability to plug them in and make them work would be much easier than trying to integrate an outlet for the LED lights underneath the bridge. This option would be the best for the bridge design because it gives the necessary lighting without the additional cost and maintenance.

6.0 Conclusions

This MQP has taught our group much about what is involved in a professional engineering project. As we began the process we were not aware of the time and effort a project of this magnitude requires. After a few slow weeks, the team learned how to effectively work in a group, play to everyone's strengths, and challenge each other to do the best work we could. This bridge is a prominent feature in Worcester's Elm Park and Worcester as a whole, and as such the corresponding effort put into this project was more than substantial.

The need for this project was established early in 2013 when the Elm Park Red Wooden Bridge was deemed unsuitable for pedestrian traffic. At this point the group was formed and work began immediately. After the initial conditions assessment, the problem areas on the previous design were located and the team worked diligently to improve these aspects.

The two major external factors that needed to be considered in the new bridge design were accessibility and historical integrity, as well as the ever-present strength and safety design aspects. After extensive research, meetings and presentations the team was able to establish the most important aspects of the redesign in the eyes of relevant stakeholders. The importance of maintaining the historic appearance as well as adherence to ADA regulations helped the team establish its final architectural design which ultimately lead to the final engineered design.

The bridge was engineered using preliminary hand calculations to find member dimensions. Then using the STAAD finite element analysis software all forces felt by members and connections were calculated, compiled, and compared to allowable strength values as set by the NDS. Ultimately, the final bridge strength results as well as the connections results have been sent to Francis Steven Harvey Jr. for a final check and implementation.

As a result of the redesign of the bridge, it mandated additional site work in Elm Park. In this way, not only would the bridge be accessible, but the surrounding landscape would be designed to be accessible as well, allowing for a fully functional park system. The group worked with the Worcester Parks and Recreation Department to develop a new site design that fit the needs of the both the Parks and Recreation team and other stakeholders concerned with the historically aesthetic features of the park. It was at this point in the project that the group was able to work with an outside engineering firm to complete the final site plans. This alone was a tremendous experience as it allowed the group to not only witness the inner-workings of a landscape engineering firm on a daily basis, but also opened our eyes to the amount of detail that is required in order to gain approval for a specific design. Ultimately, the site design in Elm Park was approved by the Worcester Conservation Commission and will be implemented in the park in the summer of 2014.

This project has served not only as an academic requirement for our graduation, but also as a headfirst dive into a real engineering project. We had to meet both academic timelines and real world timelines of city organizations in order to complete the project in the most efficient manner. We had to meet with professors and industry professionals to navigate some of our design flaws and create a final design that could be safely implemented in Elm Park. Ultimately, we have produced a structural bridge design that will meet the needs of all those who have been involved in the project, and will represent Elm Park and the city of Worcester as a sculptural ornament for many years to come.

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Appendix A: Settlement Excel Spreadsheet

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Schmertmann Method

Date	March 12, 2014
Identification	Example 7.6

Input

Units	E	E or SI
Shape	RE	SQ, CI, CO, or RE
B =	2	ft
L =	17.625	ft
D =	7	ft
P =	90	k
Dw =	5	ft
gamma =	120	lb/ft^3
t =	50	yr

Results

q =	3478	lb/ft^2
delta =	17.92	in

Depth to Soil Layer						
Top (ft)	Bottom (ft)	Es (lb/ft^2)	zf (ft)	I epsilon	strain (%)	delta (in)
0.0	7.0					
7.0	8.0	4902	0.5	0.330	19.1161	2.2939
8.0	9.0	4902	1.5	0.564	32.6003	3.9120
9.0	10.0	4902	2.5	0.588	34.0007	4.0801
10.0	11.0	4902	3.5	0.459	26.5346	3.1842
11.0	12.0	4902	4.5	0.345	19.9513	2.3942
12.0	13.0	7353	5.5	0.246	9.5006	1.1401
13.0	14.0	7353	6.5	0.148	5.7004	0.6840
14.0	15.0	7353	7.5	0.049	1.9001	0.2280
15.0	16.0	7353	8.5	0.000	0.0000	0.0000
16.0	17.0	7353	9.5	0.000	0.0000	0.0000
17.0	18.0	7353	10.5	0.000	0.0000	0.0000
18.0	19.0	7353	11.5	0.000	0.0000	0.0000
19.0	20.0	7353	12.5	0.000	0.0000	0.0000
20.0	21.0	7353	13.5	0.000	0.0000	0.0000

Appendix B: Soil Boring Logs

TEST BORING LOG

SHEET 1

Soil Exploration Corp. Geotechnical Drilling Groundwater Monitor Well 148 Pioneer Drive Leominster, MA 01453 978 840-0391	**City of Worcester DPW & Parks** **Site: Parks Division** **Elm Park** **Worcester, MA**	BORING B-1 PROJECT NO. 14-0101 DATE: January 9, 2014

Ground Elevation:
Date Started: January 6, 2014
Date Finished: January 6, 2014
Driller: GG
Soil Engineer/Geologist:

GROUNDWATER OBSERVATIONS

DATE	DEPTH	CASING	STABILIZATION

Depth Ft.	Casing bl/ft	Sample No.	Pen/Rec	Sample Depth	Blows/6"	Strata	Visual Identification of Soil and / or Rock Sample
1		1	20"	0-2'0"	37-31-21-21		Frozen fine to coarse sand and gravel, trace cobbles, medium fill.
						3'0"	
5		2	24"	5'0"-7'0"	1-1-1-1		Soft moist peat organics.
10		3	24"	10'0"-12'0"	1-1-1-1	10'0"	Soft, wet blue clay, some organics.
						14'0"	
15		4	12"	15'0"-17'0"	3-3-3-3		
20		5	15"	20'0"-22'0"	3-4-5-5		Loose to medium dense, wet, fine to coarse sand.
25		6	18"	25'0"-27'0"	6-11-15-15		
						27'0"	
30							End of boring at 27'0". Water encountered at 4'0" upon completion.
35							
39							

Notes: Hollow Stem Auger Size - 4-1/4"

Cohesionless: 0 - 4 V. Loose, 4 - 10 Loose,	Trace	0 to 10%		CASING	SAMPLE	CORE TYPE
10 -30 M Dense, 30 -50 Dense, 50+ V Dense.	Little	10 to 20%	ID SIZE (IN)		SS	
Cohesive: 0 -2 V Soft, 2 -4 Soft, 4 -8 M Stiff	Some	20 to 35%	HAMMER WGT (LB)		140 lb.	
8 -15 Stiff, 15 -30 V. Stiff, 30 + Hard.	And	35% to 50%	HAMMER FALL (IN)		30"	

TEST BORING LOG

SHEET 6

Soil Exploration Corp. Geotechnical Drilling Groundwater Monitor Well 148 Pioneer Drive Leominster, MA 01453 978 840-0391	City of Worcester DPW & Parks Site: Parks Division Elm Park Worcester, MA	BORING B-6 PROJECT NO. 14-0101 DATE: January 9, 2014

Ground Elevation:
Date Started: January 7, 2014
Date Finished: January 7, 2014
Driller: GG
Soil Engineer/Geologist:

GROUNDWATER OBSERVATIONS

DATE	DEPTH	CASING	STABILIZATION

Depth Ft.	Casing bl/ft	Sample No.	Pen/Rec	Sample Depth	Blows/6"	Strata	Visual Identification of Soil and / or Rock Sample
1		1	12"	0-2'0"	36-21-20-14		Dense, frozen, topsoil, some fine to coarse sand and gravel, medium fill.
						3'0"	
5		2	24"	5'0"-7'0"	1-1-1-1		Soft wet peat organics.
10		3	24"	10'0"-12'0"	1-1-1-1	10'0"	
							Soft, wet, some clay and organics.
15		4	14"	15'0"-17'0"	4-5-4-4	15'0"	
20		5	16"	20'0"-22'0"	5-5-6-5		Loose to medium dense, wet, fine to coarse sand, trace fine to medium gravel and silt.
25		6	18"	25'0"-27'0"	4-5-5-5		
						27'0"	
30							End of boring at 7'0". Water encountered at 4'0" upon completion.
35							
39							

Notes: Hollow Stem Auger Size - 4-1/4"

					CASING	SAMPLE	CORE TYPE
Cohesionless: 0 - 4 V. Loose, 4 - 10 Loose,	Trace	0 to 10%					
10 -30 M Dense, 30 -50 Dense, 50+ V Dense.	Little	10 to 20%	ID SIZE (IN)			SS	
Cohesive: 0 -2 V Soft, 2 -4 Soft, 4 -8 M Stiff	Some	20 to 35%	HAMMER WGT (LB)			140 lb.	
8 -15 Stiff, 15 -30 V. Stiff, 30 + Hard.	And	35% to 50%	HAMMER FALL (IN)			30"	

Appendix C: Bearing Capacity Excel Spreadsheet

BEARING CAPACITY OF SHALLOW FOUNDATIONS

Terzaghi and Vesic Methods

Date	April 1, 2014						
Identification	Example 6.4						
Input			**Results**				
Units of Measurement				Terzaghi		Vesic	
	E	SI or E	Bearing Capacity				
			q ult =	n/a	lb/ft^2	10,406	lb/ft^2
Foundation Information			q a =	n/a	lb/ft^2	5,203	lb/ft^2
Shape	RE	SQ, CI, CO, or RE					
B =	2	ft	Allowable Column Load				
L =	17.625	ft	P =	#VALUE!	k	183	k
D =	3	ft					
Soil Information							
c =	0	lb/ft^2					
phi =	35	deg					
gamma =	120	lb/ft^3					
Dw =	0	ft					
Factor of Safety							
F =	2						

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Unit conve	1000
Gamma w	62.4
phi (radiar	0.61087
Terzaghi Computations	
a theta =	4.20367
Nc =	57.75
Nq =	41.44
N gamma	47.28
gamma' =	57.6
coefficient	0
coefficient	0
sigma zD'	172.8
Vesic Computation	
Nc =	46.12
sc =	1.08
dc =	1.39
Nq =	33.30
sq =	1.08
dq =	1.25
N gamma	48.03
s gamma	0.95
d gamma	1.00
B/L =	0.11348
k =	0.98279
W sub f	0

Appendix D: Reinforced Concrete Design Tables

TABLE 17.1
Unit weights w, effective angles of internal friction ϕ, and coefficients of friction with concrete f

Soil	Unit Weight w, pcf	ϕ, deg	f
1. Sand or gravel without fine particles, highly permeable	110–120	33–40	0.5–0.6
2. Sand or gravel with silt mixture, low permeability	120–130	25–35	0.4–0.5
3. Silty sand, sand and gravel with high clay content	110–120	23–30	0.3–0.4
4. Medium or stiff clay	100–120	25–35[a]	0.2–0.4
5. Soft clay, silt	90–110	20–25[a]	0.2–0.3

[a] For saturated conditions, ϕ for clays and silts may be close to zero.

TABLE 5.1
Simplified tension development length in bar diameters according to the ACI Code

	No. 6 (No. 19) and Smaller Bars and Deformed Wires†	No. 7 (No. 22) and Larger Bars
Clear spacing of bars being developed or spliced $\geq d_b$, clear cover $\geq d_b$, and stirrups or ties throughout l_d not less than the Code minimum	$l_d = \left(\frac{f_y \psi_t \psi_e}{25\lambda\sqrt{f'_c}}\right) d_b$	$l_d = \left(\frac{f_y \psi_t \psi_e}{20\lambda\sqrt{f'_c}}\right) d_b$
Clear spacing of bars being developed or spliced $\geq 2d_b$, and clear cover $\geq d_b$	Same as above	Same as above
Other cases	$l_d = \left(\frac{3f_y \psi_t \psi_e}{50\lambda\sqrt{f'_c}}\right) d_b$	$l_d = \left(\frac{3f_y \psi_t \psi_e}{40\lambda\sqrt{f'_c}}\right) d_b$

† For reasons discussed in Section 5.3a, ACI Committee 408 recommends that l_d for No. 7 (No. 22) and larger bars be used for all bar sizes.

ψ_e = coating factor

Epoxy-coated bars or wires with cover less than $3d_b$ or clear spacing less than $6d_b$:	1.5
All other epoxy-coated bars or wires:	1.2
Uncoated and zinc-coated (galvanized) reinforcement:	1.0

However, the product of $\psi_t \psi_e$ need not be taken greater than 1.7.

ψ_s = reinforcement size factor

No. 6 (No. 19) and smaller bars and deformed wires:	0.8†
No. 7 (No. 22) and larger bars:	1.0

λ = lightweight aggregate concrete factor

When lightweight aggregate concrete is used:	0.75
However, when f_{ct} is specified, $\lambda = f_{ct}/(6.7\sqrt{f'_c}) \leq 1.0$.	
When normalweight concrete is used:	1.0

c = spacing or cover dimension, in.
Use the smaller of either the distance from the center of the bar to the nearest concrete surface or one-half the center-to-center spacing of the bars being developed.

Appendix E: Architectural Bridge Design Iterations Not Chosen

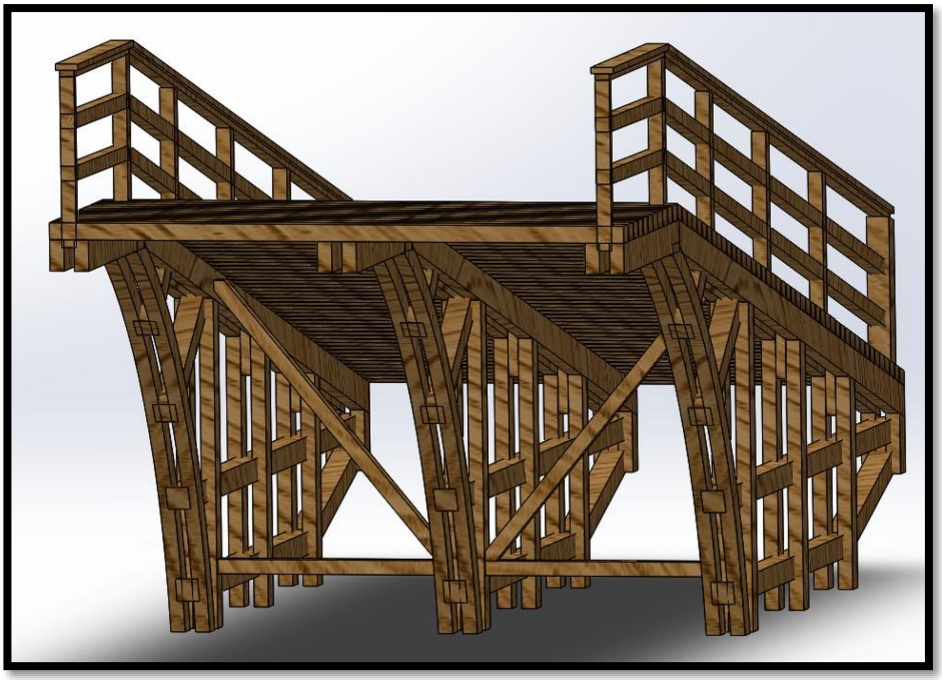


Figure 80: Replication Design 3D section cut

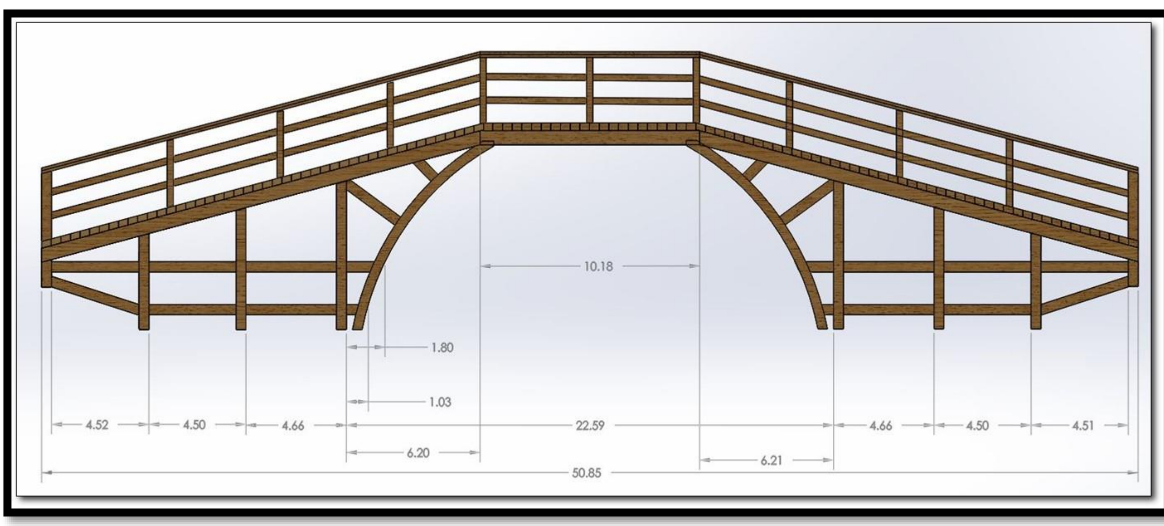


Figure 81: Replication design 2D CAD drawing with dimensions in feet

Figure 82: Switchback addition for replication design, 2D side view

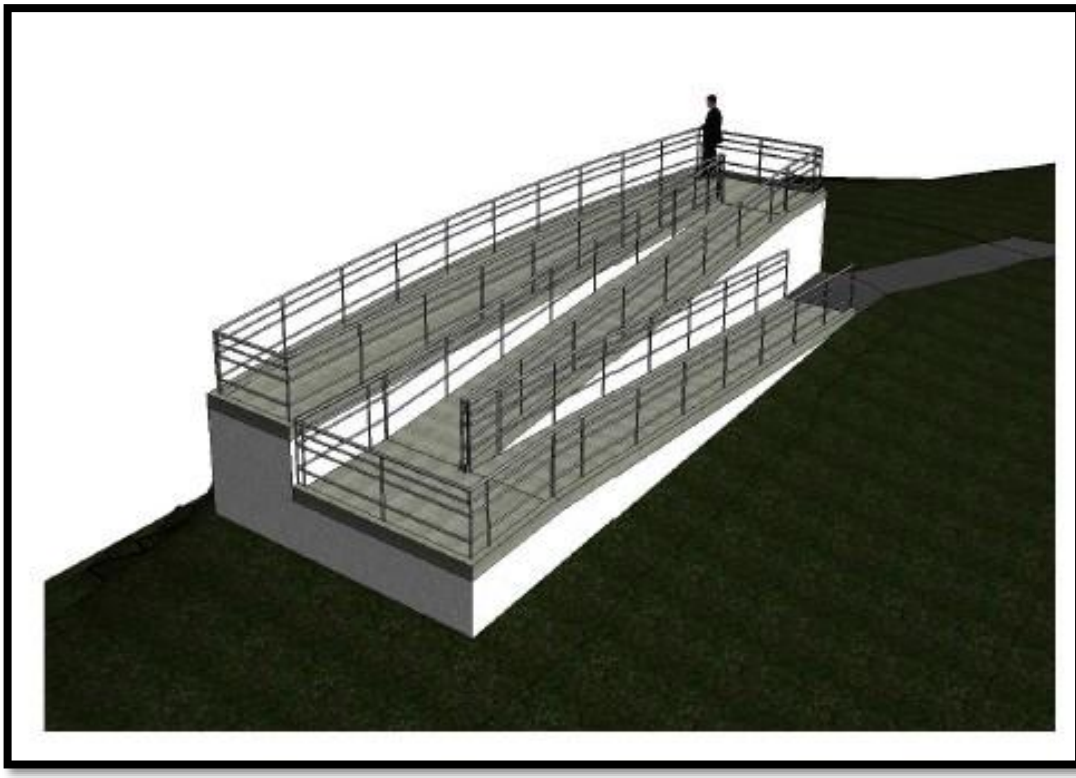


Figure 83: switchback addition for replication design, 3D isometric view

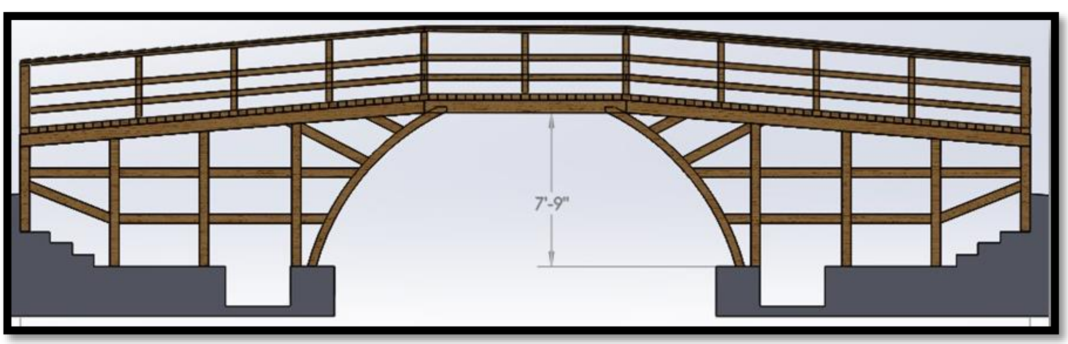


Figure 84: First ADA-compliant design iteration

Appendix F: Conditions Assessment

A complete condition assessment of the bridge was conducted November 25, 2013. In general, the structure was in poor condition with several major problems. A significant number of members were found to be missing, decayed, or cracked. These problems were due to extreme weather conditions, mediocre connection joints, vandalism, and lack of maintenance.

Deck

The deck was generally in poor condition with a significant number of timber members missing or worn down. Potential causes may have been due to frequent pedestrian usage and immediate exposure to extreme weather conditions, which also explained the chipped paint on the surface. The deck boards were of a different wood specimen since they had been replaced at one point and were therefore not part of the original material design. The lumber used for the deck was pressure treated southern yellow pine, but the rest of the structure was of Douglas Fir-Larch. Of all the bridge sections, the deck was definitely the one in the direst condition due to missing elements.

Superstructure

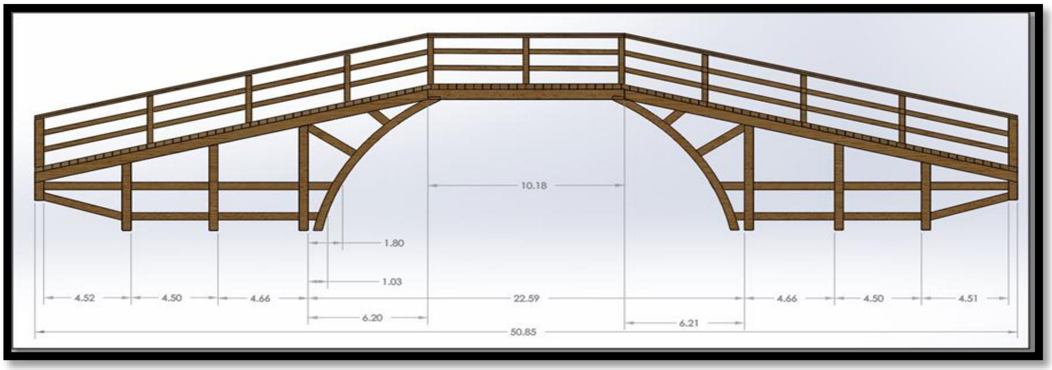
The superstructure had several structural deficiencies. The connection joints were rotten. Pick test results demonstrated that a significant number of timber members were decayed. The paint on each of the members had been chipped away or turned into a mossy green color, as the wood had succumbed to moisture over time. The joints that connected most of the members were rotted to the extent that the bolts were showing through the wood. Also most of the wood members had split down the middle causing moisture to seep through and decay them further.

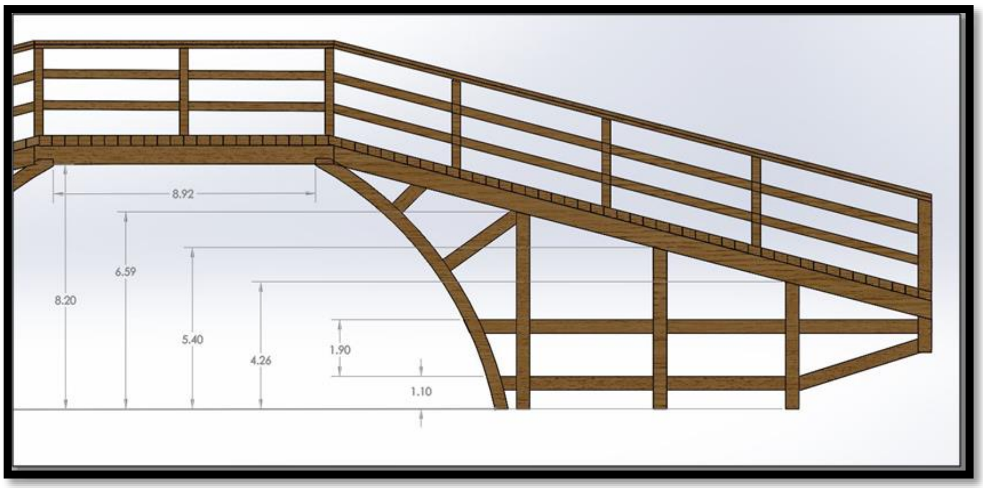
Beams were missing in quite a few places and extreme rotting had occurred in other areas. Rotting had taken place mostly in the bottom near the connection with the footing of the bridge. The weakening of these members had caused the bridge itself to become structurally unsound and dangerous for pedestrian usage. Using a screwdriver for the pick test, the wood was so weak that even with a slight jab the timber members immediately splintered, especially at the connection joints. A couple members were able to withstand the drive of stabbing with a screw driver but others were so rotten that they just broke into dust.

Substructure

The footings of the bridge were also examined for structural integrity. The footings have been in place since the redesign of the bridge in the 1960s. After an examination of the footings, they appeared to have minimal wear as a result of constant exposure to water, but visually still seemed to be adequately able to sustain the weight of the bridge. To ensure the structural integrity of the footing, a professional was recruited to analyze it and found adequate results (R. Antonelli, personal communication, Sept. 9, 2013). The footings will not need to be replaced. **Figure 85** shows, evidence of wear below the typical high water mark is apparent in addition to the growth of algae that has developed on the concrete at the polls of standing water from the pond. However, the bridge footing would still be able to withstand the minor wear.

Figure 85a, b, c, and d: Images of bridge's failure points. Top Left: missing bolts. Top right: missing beams. Bottom left: missing cross-bracing. Bottom right: rotting connections.





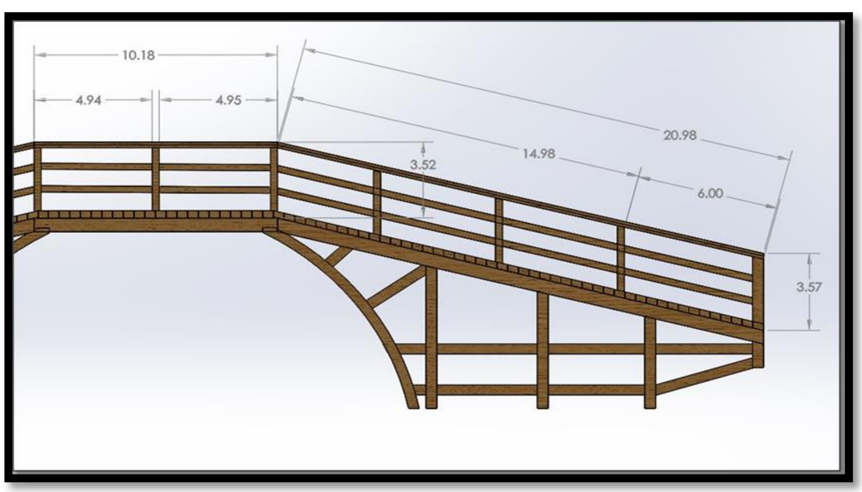


Figure 86a, b, and c: CAD drawing pictures. Top: all horizontal lengths of support members. Middle: vertical heights of support members. Bottom: railing heights and lengths.

Figure 87a and b: Images of the bridge's footings. Left: wide view of multiple piers. Right: close up view of a single pier.

Figure 88: Pick test result indicating decayed wood

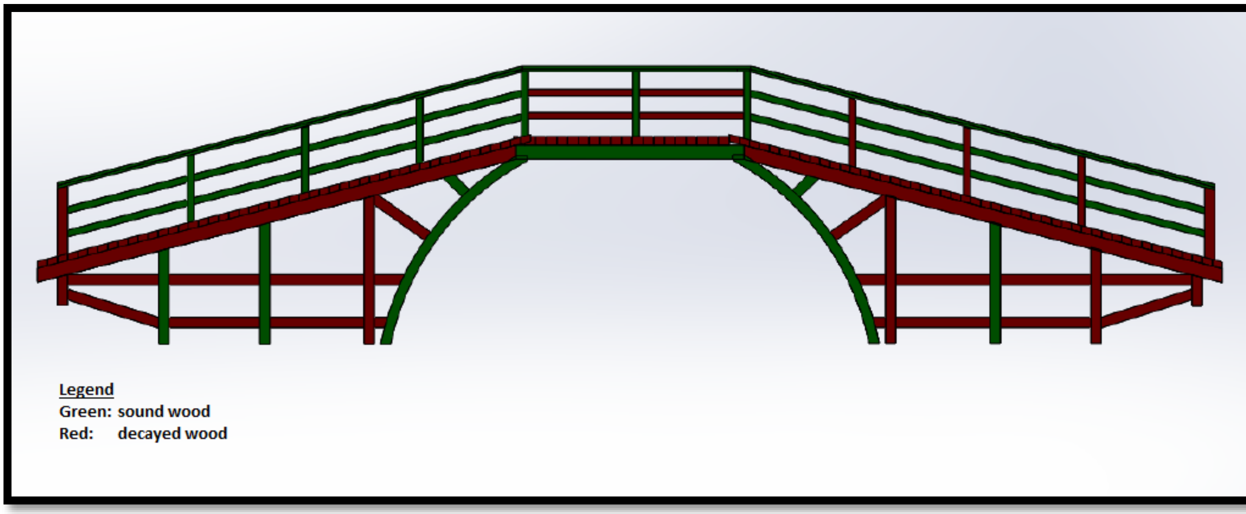


Figure 89: Front side Condition Assessment Result (South Mere View)

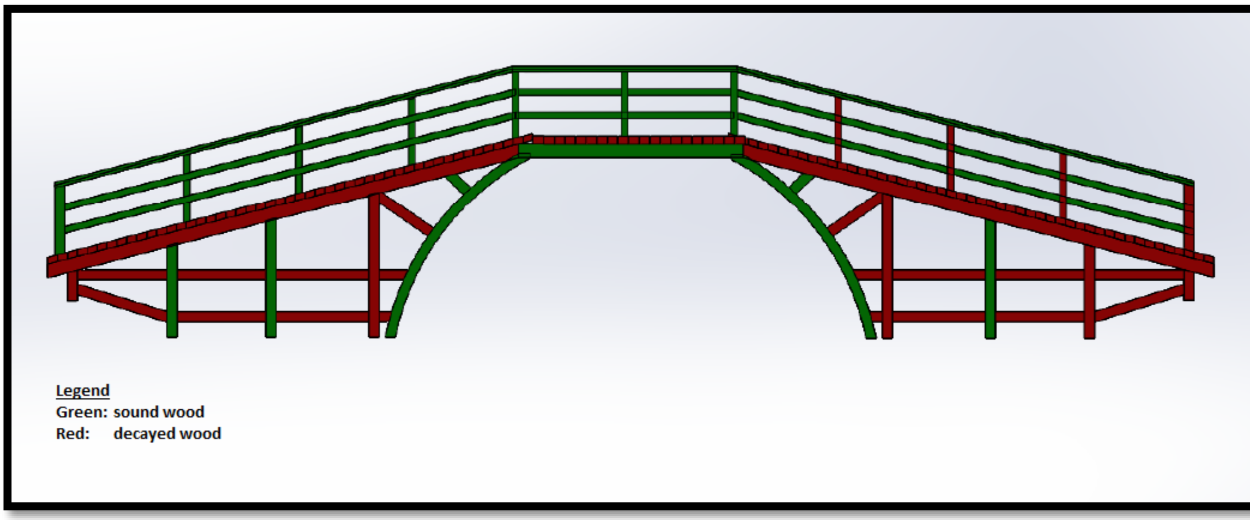


Figure 90: Backside Condition Assessment Result (North Mere View)

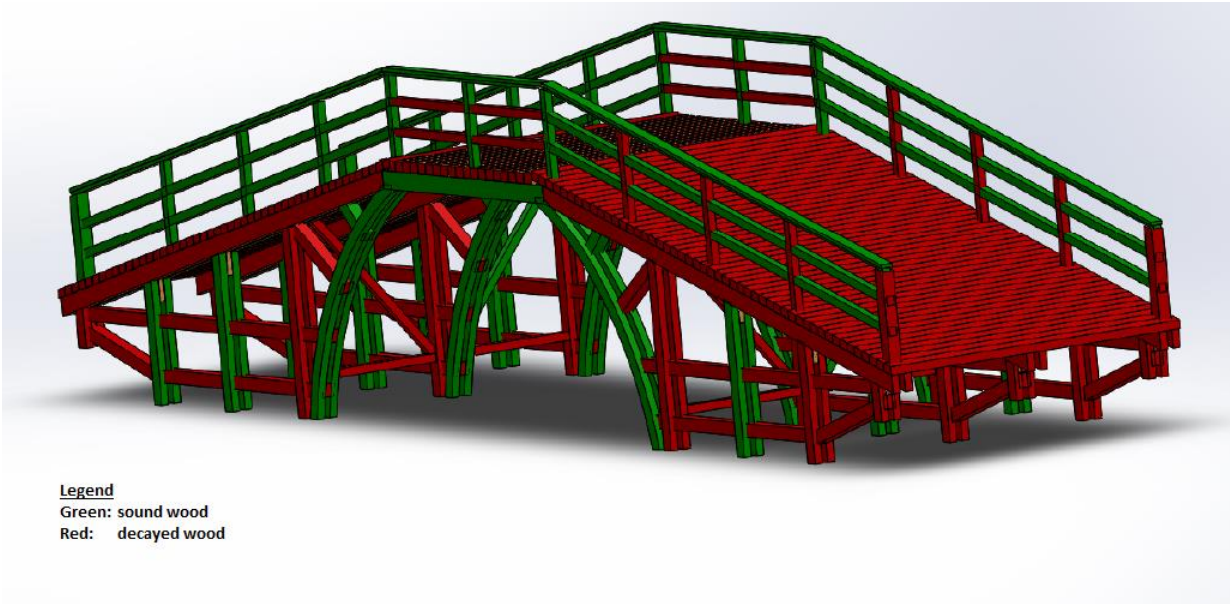


Figure 91: Left Interior Condition Assessment Result (South Mere View)

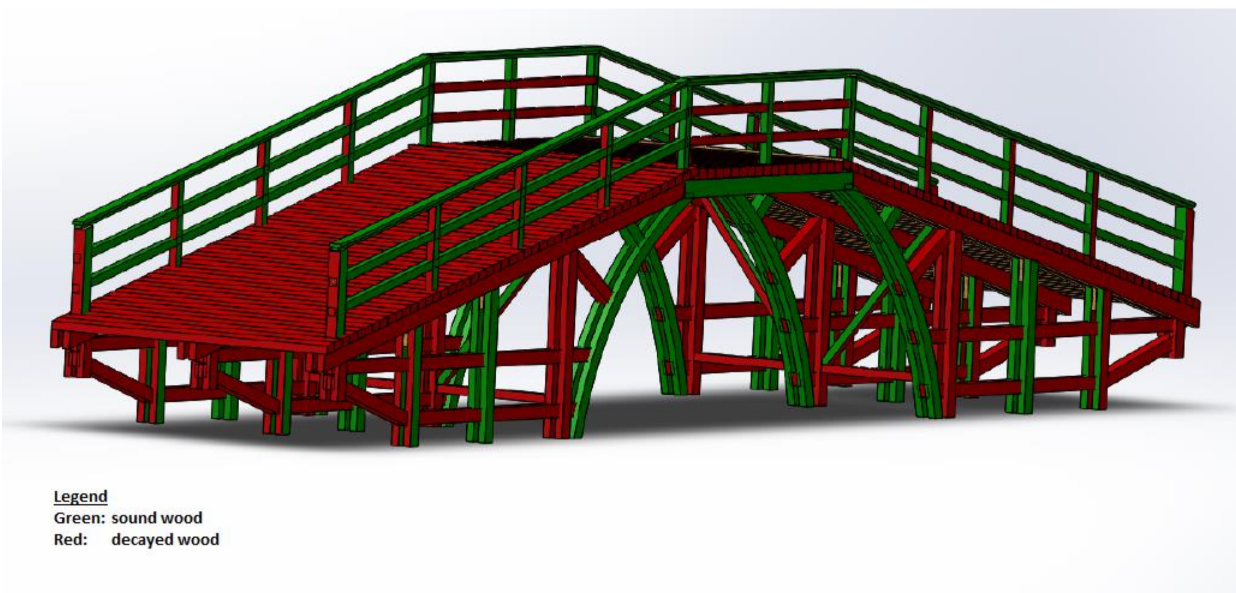


Figure 92: Right Interior Condition Assessment Result (South Mere View)

Results

A thorough condition assessment of the Myra Hiatt Kraft Bridge was important for designing a new bridge that achieves more effective structural stability and longevity. Figures 5 through 8 demonstrate color-coded drawings from different perspectives of the footbridge indicating areas of sound wood and decayed wood. In general, poor connection joints were the reason for failure, especially in areas near the Elm Park pond water level. Also, the decking boards were worn out or missing due to scour and extreme weather conditions. On the other hand, most of the railing members were in decent conditions. In order to mitigate these negative effects in the future, the new bridge design would incorporate better connection materials.

In addition to these points of error, the bridge was also experiencing a collection of snow in the connection areas. The beams' horizontal placement allowed snow and rain to easily collect in the creases. Besides these interfaces, the area near the water level also experienced a high level of rotting due to moisture. This was a result of water being too close to the wood and a lack of protection from moisture near the connections. Since the bridge stood directly on the concrete itself, the water could freely penetrate into the connections and remain in any hollow spaces.

For the new bridge design, different types of connections were studied for the base plates. These would allow the water to be drained away from the connections and therefore keep the wood from rotting as easily. With the new connections and decking, the wood would be pressure-treated and maintained every few years to help prevent weathering due to natural causes, thus prolonging the lifespan of the bridge.

Appendix F1: Strength test of Original Bridge

Table 44: Results and interpretation of compression test on wood sample.

		Load, *P*	Area, *A*	Length, *L*	Delta, *δ*	Stress, *σ*	Strain, *ε*	Young's Mod., *E*
		lb	in^2	in	in	lb/in^2	in/in	lb/in^2
comp	1	7215	3.7308	6	0.1094	1933.902	0.018233	106064
	2	16500	4.0477	6	0.15	4076.389	0.025	163056

Table 45: Results and interpretation of three-point bending test on wood sample.

		Load, *P*	Length, *L*	Width, *b*	Depth, h	Delta, *δ*	Stress, *σ*	Strain, *ε*	Young's Mod., *E*	Bend Mod., *E*
		lb	in	in	in	in	lb/in^2	in/in	lb/in^2	lb/in^2
bend	1	917	16	1.904	1.921	0.315	3132.268	0.019688	159099	220856.3
	2	1175	16	2.072	2.051	0.15	3235.4	0.009375	345109	448703.9

Appendix G: Results of STAAD Analysis

Appendix G1: Numbering Schemes

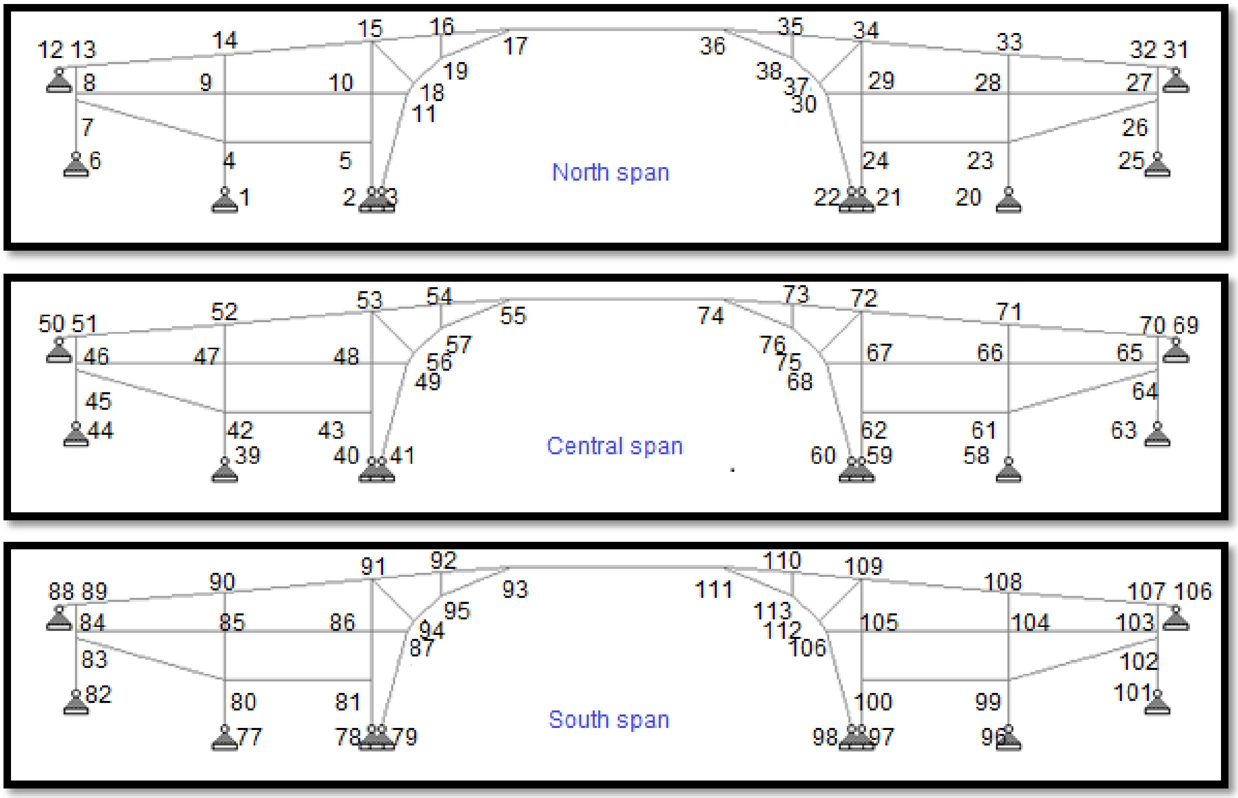


Figure 93: Nodal numbering scheme used for reference in spreadsheets.

Appendix G2: Full STAAD Results

C1	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	1468	211.4	309.5	208.0
$F'_{b,y}$	1469	0.0	0.0	0.0
F'_{t}	0	0.0	0.0	0.0
$f_{v,y}$	285	8.3	12.2	4.6
$f_{v,z}$	285	0.0	0.0	0.0
F'_{c}	893	174.2	326.2	174.2

C2	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	1468	176.1	269.2	173.0
$F'_{b,y}$	1469	11.5	0.6	10.6
F'_{t}	950	29.6	0.0	28.8
$f_{v,y}$	285	5.3	8.1	5.2
$f_{v,z}$	285	0.4	0.0	0.4
F'_{c}	886	0.0	40.7	0.0

C3	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	1468	513.3	727.9	506.4
$F'_{b,y}$	1469	0.0	0.0	0.0
F'_{t}	0	0.0	0.0	0.0
$f_{v,y}$	285	4.1	41.0	28.5
$f_{v,z}$	285	10.1	0.0	0.0
F'_{c}	890	113.6	307.0	190.8

B4	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	3322	195.1	401.2	195.1
$F'_{b,y}$	3370	0.0	0.0	0.0
F'_{t}	1853	0.0	7.4	3.0
$f_{v,y}$	302	1.6	27.0	13.1
$f_{v,z}$	302	13.5	0.0	0.0
F'_{c}	880	3.6	0.0	0.0

B5	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	4082	0.9	0.9	0.9
$F'_{b,y}$	4156	0.0	0.0	0.0
F'_{t}	2359	0.0	0.0	0.0
$f_{v,y}$	302	0.2	0.2	0.2
$f_{v,z}$	302	0.0	0.0	0.0
F'_{c}	873	101.9	151.4	100.5

B1	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	4086	195.1	401.2	195.1
$F'_{b,y}$	4156	0.0	0.0	0.0
F'_{t}	2359	0.0	0.0	0.0
$f_{v,y}$	302	13.1	27.0	13.1
$f_{v,z}$	302	0.0	0.0	0.0
F'_{c}	945	72.7	104.9	71.8

B2	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	4086	195.1	401.2	195.1
$F'_{b,y}$	4156	0.0	0.0	0.0
F'_{t}	2359	0.0	0.0	0.0
$f_{v,y}$	302	13.1	27.0	13.1
$f_{v,z}$	302	0.0	0.0	0.0
F'_{c}	945	89.7	130.3	88.5

B3	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	4143	11.4	11.4	11.4
$F'_{b,y}$	4156	0.0	0.0	0.0
F'_{t}	2359	6.9	11.8	6.8
$f_{v,y}$	302	0.7	0.7	0.7
$f_{v,z}$	302	0.0	0.0	0.0
F'_{c}	3383	0.0	0.0	0.0

S1	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	1463	604.1	947.6	604.4
$F'_{b,y}$	1469	40.0	0.3	39.6
F'_{t}	950	0.0	0.0	0.0
$f_{v,y}$	285	86.7	120.8	86.6
$f_{v,z}$	285	4.8	0.0	4.8
F'_{c}	712	78.6	98.6	78.5

B6	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	4132	0.0	1.6	1.6
$F'_{b,y}$	4156	0.0	0.0	0.0
F'_{t}	0	0.0	0.0	0.0
$f_{v,y}$	302	0.3	0.3	0.3
$f_{v,z}$	302	0.0	0.0	0.0
F'_{c}	2989	312.1	407.7	308.5

B7	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	4143	0.0	0.0	0.0
$F'_{b,y}$	4156	0.0	0.0	0.0
F'_{t}	0	0.0	0.0	0.0
$f_{v,y}$	302	0.0	0.0	0.0
$f_{v,z}$	302	0.0	0.0	0.0
F'_{c}	3381	142.2	260.4	139.5

A1	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	1468	577.9	758.2	571.8
$F'_{b,y}$	1469	0.0	0.0	0.0
F'_{t}	0	0.0	0.0	0.0
$f_{v,y}$	285	105.8	133.7	104.8
$f_{v,z}$	285	0.0	0.0	0.0
F'_{c}	893	428.2	627.5	422.3

S2	Allow	Ext1	Int	Ext 2
$F'_{b,z}$	1457	908.8	985.2	907.7
$F'_{b,y}$	1469	40.0	0.2	3.6
F'_{t}	0	0.0	0.0	0.0
$f_{v,y}$	285	72.5	102.1	72.3
$f_{v,z}$	285	3.8	0.0	0.4
F'_{c}	631	150.6	218.4	149.5

D1	Allow	Calc	D2	Allow	Calc	DM	Allow	Calc
$F'_{b,z}$	4116	1268.2	$F'_{b,z}$	4114	89.4	$F'_{b,z}$	4027	1839.7
$F'_{b,y}$	4156	105.1	$F'_{b,y}$	4156	216.3	$F'_{b,y}$	3538	8.0
F'_{t}	2359	0.0	F'_{t}	2359	0.0	F'_{t}	0	0.0
$f_{v,y}$	302	35.0	$f_{v,y}$	302	1.6	$f_{v,y}$	302	37.5
$f_{v,z}$	302	1.6	$f_{v,z}$	302	2.9	$f_{v,z}$	302	0.5
F'_{c}	410	14.2	F'_{c}	375	27.3	F'_{c}	1377	4.7

Appendix G3: STAAD Results factors of safety

C1	Ext1	Int	Ext 2
$F'_{b,z}$	6.9	4.7	7.1
$F'_{b,y}$	0.0	0.0	0.0
F'_{t}	0.0	0.0	0.0
$f_{v,y}$	34.2	23.4	62.0
$f_{v,z}$	0.0	0.0	0.0
F'_{c}	5.1	2.7	5.1

C2	Ext1	Int	Ext 2
$F'_{b,z}$	8.3	5.5	8.5
$F'_{b,y}$	127.3	2395.8	138.5
F'_{t}	32.1	0.0	33.0
$f_{v,y}$	53.6	35.1	54.6
$f_{v,z}$	718.7	8624.9	784.1
F'_{c}	0.0	21.8	0.0

C3	Ext1	Int	Ext 2
$F'_{b,z}$	2.9	2.0	2.9
$F'_{b,y}$	0.0	0.0	0.0
F'_{t}	0.0	0.0	0.0
$f_{v,y}$	69.5	7.0	10.0
$f_{v,z}$	28.2	0.0	0.0
F'_{c}	7.8	2.9	4.7

B4	Ext1	Int	Ext 2
$F'_{b,z}$	17.0	8.3	17.0
$F'_{b,y}$	0.0	0.0	0.0
F'_{t}	0.0	249.5	615.1
$f_{v,y}$	189.0	11.2	23.0
$f_{v,z}$	22.4	0.0	0.0
F'_{c}	244.4	0.0	0.0

B5	Ext1	Int	Ext 2
$F'_{b,z}$	4716.3	4716.3	4716.3
$F'_{b,y}$	0.0	0.0	0.0
F'_{t}	0.0	0.0	0.0
$f_{v,y}$	1455.3	1455.3	1455.3
$f_{v,z}$	0.0	0.0	0.0
F'_{c}	8.6	5.8	8.7

B1	Ext1	Int	Ext 2
$F'_{b,z}$	20.9	10.2	20.9
$F'_{b,y}$	0.0	0.0	0.0
F'_{t}	0.0	0.0	0.0
$f_{v,y}$	23.0	11.2	23.0
$f_{v,z}$	0.0	0.0	0.0
F'_{c}	13.0	9.0	13.2

B2	Ext1	Int	Ext 2
$F'_{b,z}$	20.9	10.2	20.9
$F'_{b,y}$	0.0	0.0	0.0
F'_{t}	0.0	0.0	0.0
$f_{v,y}$	23.0	11.2	23.0
$f_{v,z}$	0.0	0.0	0.0
F'_{c}	10.5	7.2	10.7

B3	Ext1	Int	Ext 2
$F'_{b,z}$	364.7	364.7	364.7
$F'_{b,y}$	0.0	0.0	0.0
F'_{t}	344.0	199.1	346.6
$f_{v,y}$	415.8	415.8	415.8
$f_{v,z}$	0.0	0.0	0.0
F'_{c}	0.0	0.0	0.0

S1	Ext1	Int	Ext 2
$F'_{b,z}$	2.4	1.5	2.4
$F'_{b,y}$	36.8	4525.4	37.1
F'_{t}	0.0	0.0	0.0
$f_{v,y}$	3.3	2.4	3.3
$f_{v,z}$	58.8	5880.6	59.7
F'_{c}	9.1	7.2	9.1

B6	Ext1	Int	Ext 2
$F'_{b,z}$	0.0	2546.3	2546.3
$F'_{b,y}$	0.0	0.0	0.0
F'_{t}	0.0	0.0	0.0
$f_{v,y}$	1164.2	1164.2	1164.2
$f_{v,z}$	0.0	0.0	0.0
F'_{c}	9.6	7.3	9.7

B7	Ext1	Int	Ext 2
$F'_{b,z}$	0.0	0.0	0.0
$F'_{b,y}$	0.0	0.0	0.0
F'_{t}	0.0	0.0	0.0
$f_{v,y}$	0.0	0.0	0.0
$f_{v,z}$	0.0	0.0	0.0
F'_{c}	23.8	13.0	24.2

A1	Ext1	Int	Ext 2
$F'_{b,z}$			
$F'_{b,y}$			
F'_{t}			
$f_{v,y}$			
$f_{v,z}$			
F'_{c}			

S2	Ext1	Int	Ext 2
$F'_{b,z}$	1.6	1.5	1.6
$F'_{b,y}$	36.8	6788.1	403.3
F'_{t}	0.0	0.0	0.0
$f_{v,y}$	3.9	2.8	3.9
$f_{v,z}$	74.5	14897.5	709.4
F'_{c}	4.2	2.9	4.2

D1	Calc	D2	Calc
$F'_{b,z}$	3.2	$F'_{b,z}$	46.0
$F'_{b,y}$	39.5	$F'_{b,y}$	19.2
F'_{t}	0.0	F'_{t}	0.0
$f_{v,y}$	8.6	$f_{v,y}$	185.2
$f_{v,z}$	185.2	$f_{v,z}$	105.8
F'_{c}	28.9	F'_{c}	13.7

DM	Calc
$F'_{b,z}$	2.2
$F'_{b,y}$	443.9
F'_{t}	0.0
$f_{v,y}$	8.1
$f_{v,z}$	646.8
F'_{c}	291.2

Appendix H: Connection Calculations

Appendix H1: Yield strength parameters

$$k_1 = \frac{\sqrt{R_e + 2R_e^2\left(1 + R_t + R_t^2\right) + R_t^2 R_e^3} - R_e(1 + R_t)}{1 + R_e}$$

$$k_2 = -1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}}$$

$$k_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_m^2}}$$

$$R_e = \frac{F_{em}}{F_{es}}$$

$$R_t = \frac{l_m}{l_s}$$

$$R_d = f(K_\theta)$$

$$K_\theta = 1 + .25(\theta/90°)$$

Appendix H2: NDS Design Values for Bridge Connections

For supporting calculated for all work in this appendix, please see attached file: NDS Design Values – Connections.xlsx

Modified yield and withdrawal strengths *Z'* and *W'* of each major connection on the bridge structure. Additionally, the net strengths for the total amount of dowels used, P_W and P_Z, are provided, along with member loads to be resisted, F_W and F_Z.

Design Values		Fnd'n to Col.	Col. to Beam ⊥	Col. To Beam θ	Beam to Col.	Col. To Deck Sup	Deck Sup to Col.	Arch to Beam 11	Arch to Beam 20	Arch to Beam 21	Arch to H Deck Sup	X-Sups	H-Sups	Decking	
	Failure	I_m	III_s	I_m	III_s	III_s	III_s	I_m	I_m	I_m	III_s	IV	IV	IV	
	Shear	2	2	2	1	2	1	2	2	2	2	1	1	1	
	Z'	**4399.4**	**2618.5**	**2804.8**	**450.3**	**2553.4**	**2208.3**	**3464.6**	**2933.5**	**2640.2**	**1489.4**	**190.0**	**190.0**	**229.0**	lb
	W'				**105.2**							**135.5**	**135.5**	**132.7**	lb
	n	3	3	*3*	6	4	4	3	*3*	*3*	8	3	3	2	fasteners
	P_Z	13198.1	7855.6	8414.5	2701.7	10213.8	8833.4	10393.8	8800.6	7920.5	11914.9	570.1	570.1	457.9	total lb
	P_W				631.2							406.5	406.5	265.4	total lb
Applied	**F_Z**	9876	2818	2818	747	6843	6836	178	7952	4956	11398	328	328	76	lb
	F_W				3							326	326	0	lb

Appendix H3: NDS Design Values for Railing Connections

For supporting calculated for all work in this appendix, please see attached file:

<u>NDS Design Values – Rail Connections.xlsx</u>

Modified yield and withdrawal strengths *Z'* and *W'* of each major connection on the railings. Additionally, the net strengths for the total amount of dowels used, P_W and P_Z, are provided, along with member loads to be resisted, F_W and F_Z.

Design Values		Support to End Post	Support to Inner Post	Post to X	Mid Hold to X	Post to Upper Hold	Post to Mid Hold	
	Failure	I_m	III_s	I_m	III_s	III_s	III_s	
	Shear	2	2	2	1	2	1	
	Z'	**1010.8**	**1010.8**	**190.0**	**190.0**	**153.4**	**190.0**	lb
	W'			**108.0**	**71.2**	**177.0**	**93.1**	
	n	2	2	2	2	2	2	fasteners
	P_Z	2021.7	2021.7	380.1	380.1	306.8	380.1	total lb
	P_W			215.9	142.5	353.9	186.2	total lb
Applied	**F_Z**	194.0	194.0	16.0	16.0	150.0	16	lb
	F_W			36	36.0	30	28	lb
		bolt	bolt	nails	nails	nails	nails	
TOTAL	**n**	8	56	240	240	64	120	fasteners
		64	bolts	664	nails			

Appendix I: Bearing Capacity Hand Calculations

Bearing Capacity

(1) Initial Assumptions:

* Assume 3 separate combined, rectangular, strip footings connected with perpendicular inter connection pieces that do not contribute to structural support or bearing capacity calcs.
* Assume a total depth of 6 feet from 492' elevation.
* Assume that peat + clay material were excavated in 1972 as these soil types are unsuitable for supporting structural components.
* Assume Peat + Clay were replaced with soil similar to soil found @ 15' depth adjacent to foundations.
* Evaluate all three rectangular strip footings separately with resultant force acting from 3 column loads subjected on each strip by bridge.
* Assume 3 feet of foundation showing above pond floor.
* Base column loads off Staad Analysis design values.

(2) Initial Parameters:

$B = 2'$

$L = 211.50'' = 17.625'$

$\gamma_w = 62.4\ lb/ft^3$

GW table = 0 feet

Soil Properties:

$c' = 0$

$\phi = 35°$

$\gamma = 120\ lb/ft^3$

Use Vesic's Method

* Only consider shape, depth, bearing capacity factors, rest = 1

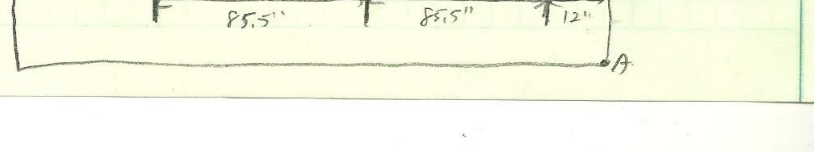
$\therefore q_{ult} = \underbrace{\cancel{c'N_c s_c d_c}}_{c'=0} + \sigma_{zD}' N_q s_q d_q + 0.5\gamma' B N_\gamma s_\gamma d_\gamma$

$\therefore q_{ult} = \sigma_{zD}' N_q s_q d_q + 0.5\gamma' B N_\gamma s_\gamma d_\gamma$

(3) Determine Pressure of foundation strips:

Strip 1: Closest to South Mere

Total Load = 14.656k



1) Determine location of resultant

Take moment around front edge

$\Sigma M_A = 14.636\,x = 7.219(12") + 3.980(97.5) + 3.437(183")$

$14.636\,\bar{x} = 1103.649$

$\bar{x} = 14.636\,k$ @ $75.406"$ ⟹ Middle third of foundation good ✓

2) Determine weight of foundation @ 6 feet

*Assume Normal Wt. Concr = $150\ lb/ft^3$

$W_F = 150\ lb/ft^3 \times 2' \times 17.625' \times 6' = 31,725\ lbs = 31.725\ k.$

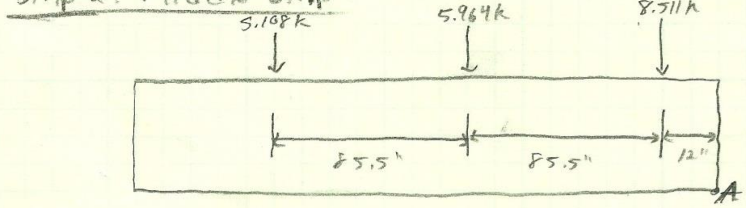
3) Determine total Pressure at bottom of footing

$$\frac{W_F + R}{\text{square Area of Strip}} = \frac{14.636 + 31.725\ k}{2' \times 17.625'} = \underline{1.32\ ksf}$$

(compare to allowable bearing capacity)

Strip 2: Middle Strip

Total Load = 19.583 K



1) Determine location of Resultant

$\Sigma M_A = 19.583\,\bar{x} = 8.511(12) + 5.964(97.5) + 5.108(183)$

$19.583\,\bar{x} = 1618.37$

$\bar{x} = 82.64"$ ⟹ middle third of foundation good ✓

2) Determine weight of foundation @ 8'

$W_F = 31,725\ lbs = 31.725\ k$

3) Determine total Pressure @ bottom of footing

$$\frac{19.583 + 31.725}{2' \times 17.625'} = \underline{1.45\ Ksf}$$ (compare to allowable bearing capacity)

Strip 3: Closest to North Mere

* Based on stand, same pressure distribution as Strip 1:

∴ Total Pressure = $\underline{1.32\ ksf}$

(4) Bearing Capacity of Strips 1+3

σ_{zD}' @ depth 6' (3' above ground level)

$\sigma_{zD}' = \Sigma \gamma h - u = (120\ lb/ft^3)(3') - (62.4)(3')$

$\sigma_{zD}' = 172.8\ lb/ft^2$

$\gamma' (D_w < D) = \gamma_b = \gamma - \gamma_w = 120\ lb/ft^3 - 62.4\ lb/ft^3 = 57.6\ lb/ft^3$

From table 6.1: Foundation Design: ($\phi = 35°$)

$N_q = 33.3$

$N_\gamma = 48.0$

Other factors: (Foundation Design Second edition)

$s_q = 1 + \left(\frac{B}{L}\right) \tan\phi' = 1 + \left(\frac{2}{17.625}\right) \tan 35 = 1.079$

$s_\gamma = 1 - 0.4\left(B/L\right) = 1 - 0.4(2/17.625) = .955$

$d_q = 1 + 2k \tan\phi'(1 - \sin\phi')^2 = 1 + 2(.9827) \tan 35(1 - \sin 35)^2 = 1.25$

$k = \tan^{-1}\left(\frac{3}{2}\right) = .9827$

$d_\gamma = 1$

$\therefore q_{ult} = (172.8\ lb/ft^2)(33.3)(1.079)(1.25) + 0.5(57.6\ lb/ft^3)(2')(48.0)(.955)(1)$

$q_{ult} = 10{,}401.41\ lb/ft^2$

* Assume Factor of Safety of 2 given uncertain soil parameters below foundation

$q_a = \frac{q_{ult}}{F} = \frac{10{,}401.41\ lb/ft^3}{2} = 5200.71\ lb/ft^2$

$q_a = 5.2\ ksf$

$q_a > P_{1+3}$

$5.2\ ksf > 1.32\ ksf$ ✓

* Soil can withstand loads imparted by strips 1+3 with adequate factor of safety.

(5) Bearing Capacity of Strip 2:

From previous calculations

$\sigma'_{zD} = 172.8\ lb/ft^2$

$\gamma' = 57.6\ lb/ft^3$

$N_q = 33.3$

$N_\gamma = 48.0$

$s_q = 1.079$

$s_\gamma = .955$

$d_q = 1.25$

$d_\gamma = 1$

∴ Factors are same, q_{ult} + q_a will be the same

∴ $q_a = 5.2\ ksf$

$q_a > P_2$

$5.2\ ksf > 1.45\ ksf$ ✓

* Soil can withstand loads imparted by strip 2 with adequate factor of safety

* Any loads imparted by interconnection pieces should be considered negligible.

Appendix J: Hilti Adhesive Chart

Adhesive	Size: Minimum Size Diameter	Size: Maximum Size Diameter	Hole Cleaning Option[14]: Standard Cleaning (blow, brush, blow)	Hole Cleaning Option[14]: Self Cleaning with TE-CD/YD Hollow Drill Bit[11]	Hole Cleaning Option[14]: Non Cleaning with specialty HIT-Z Rod	Head Type: Stud (Externally Threaded)	Head Type: Flush or Internally Threaded	Head Type: Rebar	Head Type: Hex Bolt	Head Type: Round/Mushroom	Base Material: Concrete	Base Material: Lightweight Concrete	Base Material: Grout-filled Block	Base Material: Hollow Block/Brick[12]	Corrosion Resistance: Hot-dip Galvanized Steel	Corrosion Resistance: 304 Stainless Steel	Corrosion Resistance: 316 Stainless Steel	Features: Seismic	Features: Cracked Concrete (per ICC-ES)[13]	Approvals (1: FM; 2: UL; 3a: ICC Concrete; 3b: ICC Grouted Masonry; 3c: ICC Hollow Masonry; 3d: ICC Unreinforced Masonry; 4: COLA; 5: Miami-Dade; 6: NSF 61; 7: LEED)
HIT HY 200-A [8, 9] Hybrid Adhesive	3/8"	1-1/4"	■	■	■	■	■	■			■	■			■	■	■	■	■	3a, 4*, 6, 7
HIT HY 200-R [8, 9] Hybrid Adhesive	3/8"	1-1/4"	■	■	■	■	■	■			■	■			■	■	■	■	■	3a, 4*, 6, 7
HIT-HY 150 MAX-SD [8, 9] Hybrid Adhesive	3/8"	1"	■			■		■			■	■			■	■	■	■	■	3a, 4, 6, 7
HIT-HY 150 MAX [8, 9] Hybrid Adhesive	3/8"	1-1/4"	■			■	■	■			■	■	■		■	■	■	■ A-B		3a, 3b, 4, 5, 6, 7
HIT-RE 500-SD [8, 9] Epoxy Adhesive	3/8"	1-1/4"	■			■	■	■			■	■			■	■	■	■	■	3a, 4, 6, 7
HIT-RE 500 Epoxy Adhesive	3/8"	>1-1/4"	■			■	■	■			■	□			■	■	■			6, 7
HIT-HY 70 [8, 9] Hybrid Adhesive	1/4"	3/4"	■			■	■	■					■	■	■	■	■	■		3b*, 3c*, 3d, 4, 7
HVU Adhesive Capsule	3/8"	1-1/4"	■			■	■	■			■	□	□		■	■	■			7
HIT-ICE Cold Weather Adhesive	3/8"	1-1/4"	■			■	■	■			■	■	■		■	■	■			7
HIT-HY 10 PLUS Hybrid Adhesive	3/8"	3/4"	■			■	■	■			■	□	■	■	■	■	■			7

Appendix K: Bridge Member Calculations

Appendix K1: NDS Design Values for Bridge Support Structure

For supporting calculated for all work in this appendix, please see attached file: NDS Design Values – Bridge.xlsx

Modified design strength of sawn lumber members of the bridge.

LRFD						8-9-10-11											
	1-4-9-14	2-5-10-15	6-7-8-13	4-5	4-7	8-9	9-10	10-11	12-...-17	15-18	16-19	5-53	15-43	17-36	5-15	decking	
$F'_{b,z}$	1468	1468	1468	3322	4082	4086	4086	4143	1463	4132	4143	4116	4114	1457	4127	4027	psi
$F'_{b,y}$	1469	1469	1469	3370	4156	4156	4156	4156	1469	4156	4156	4156	4156	1469	4156	3538	psi
F'_{t}		950	950	1853	2359	2359	2359	2359	950		2359	2359	2359				psi
F'_{v}	285	285	285	302	302	302	302	302	285	302	302	302	302	285	302	302	psi
$F'_{c\perp}$	703	703	703	1059	1059	1059	1059	1059	703	1059	1059	1059	1059	703	1059	1059	psi
F'_{c}	893	886	890	880	873	945	945	3383	712	2989	3381	410	375	631	679	1377	psi
E'	1140000	1140000	1140000	1520000	1615000	1615000	1615000	1615000	1140000	1615000	1615000	1615000	1615000	1140000	1615000	1520000	psi
E'_{min}	627000	627000	627000	826500	883500	883500	883500	883500	627000	883500	883500	883500	883500	627000	883500	826500	psi

Modified design strength of glulam arch members of the bridge.

	arch	
$F'_{b,z}$	3460	psi
$F'_{b,y}$	4328	psi
F'_{t}	3024	psi
$F'_{v,z}$	648	psi
$F'_{v,y}$	562	psi
$F'_{c\perp}$	1388	psi
F'_{c}	4026	psi
E'	1700000	psi
E'_{min}	1320000	psi
F'_{rt}	216	psi

Modification factors used to adjust design strength. Lines highlighted in red were not used.

Adjustment factors		1-4-9-14	2-5-10-15	6-7-8-13	4-5	4-7	8-9-10-11			12-...-17	15-18	16-19	5-53	15-43	17-36	5-15	decking	*arch*
Load duration	C_D	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15
Wet service	C_M	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Temperature	C_t	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Beam stability	$C_{L,z}$	0.999354	0.999225	0.99929	0.985982	0.982233	0.983199	0.983199	0.99697	0.996025	0.994308	0.99694	0.990474	*0.989943*	*0.992138*	*0.992952*	*0.989753*	*0.998206*
	$C_{L,y}$	0.999354	0.999225	0.99929	0.996948	0.996283	0.996449	0.996449	0.999249	0.998497	0.998636	0.999242	0.990474	*0.989943*	*0.998629*	*0.992952*	*0.997674*	*0.998663*
Size	C_F	1.0	1.0	1.0	1.3	1.3	1.3	1.3	1.3	1.0	1.3	1.3	1.3	1.3	1.0	1.3	1.3	
		1.0	1.0	1.0	1.3	1.3	1.3	1.3	1.3	1.0	1.3	1.3	1.3	1.3	1.0	1.3	1.3	
		1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.0	1.1	1.1	1.1	1.1	1.0	1.1	1.1	
Flat use	$C_{fu,z}$	1	1	1	1	1	1	1	1	1	1	1	1	1	1	*1*	*1.05*	1
	$C_{fu,y}$	1	1	1	1	1	1	1	1	1	1	1	1	1	1	*1*	*1.05*	1
Incising factor	C_i	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
		1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
		0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	
Repetitive	C_r	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1.15	
Column stability	C_P	0.98424	0.976988	0.980829	0.280538	0.248152	0.268596	0.268596	0.962172	0.785167	0.849911	0.961402	0.116541	0.106509	0.695615	0.193223	0.438917	0.847322
Buckling stiffness	C_T	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	
Bearing area	C_b	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1.0
Volume	C_V																	1.003097
Curvature	C_c	1	1	1	1	1	1			1	1	1	1	1	*1*	1		1
Format conversion and Resistance	ϕK_f	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16
		1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875
		1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
		1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Time effect	λ	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Appendix K2: Comparison of Design Values to Analysis

For the full comparison of each individual member and stress in the bridge, please see attached file:

<u>NDS Max stresses and interactions.xlsx</u>

Interaction equations for members in bending and tension. Only for members in which the cells are white are these equations applied. For each three lines of X_{bzt} or X_{byt}, the first line represent the span of members closest to the wind source, the second to the middle span, and the third to the span farthest from the wind source. Where the cell is green, the member passed the interaction test and was therefore acceptable.

Interaction Equations
Bending and Axial Tension

$$X_{bt1} = \frac{f_t}{F_t'} + \frac{f_b}{F_b^*} \leq 1.0 \qquad X_{bt2} = \frac{f_b - f_t}{F_b^{**}} \leq 1.0$$

		1-4-9-14	2-5-10-15	6-7-8-13	4-5	4-7	8-9 (8-9-10-11)	9-10 (8-9-10-11)	10-11 (8-9-10-11)	12-...-17	15-18	16-19	*arch*	5-53	15-43	17-36	5-43	decking
LRFD	$F_{b,z}$*	1469	1469	1469	3370	4156	4156	4156	4156	1469	4156	4156	3467	4156	4156	1469	4156	
	$F_{b,y}$*	1470	1470	1470	3380	4171	4171	4171	4159	1471	4162	4159	4333	4196	4198	1471	4185	

$F_b^* = F_b'/C_L$

		1-4-9-14	2-5-10-15	6-7-8-13	4-5	4-7	8-9 (8-9-10-11)	9-10 (8-9-10-11)	10-11 (8-9-10-11)	12-...-17	15-18	16-19	arch	5-53	15-43	17-36	5-15	decking
LRFD	$F_{b,z}$**	1468	1468	1468	3322	4082	4086	4086	4143	1463	4132	4143	0	4116	4114	1457	4127	
	$F_{b,y}$**	1469	1469	1469	3370	4156	4156	4156	4156	1469	4156	4156	0	4156	4156	1469	4156	

$F_b^{**} = F_b'/C_v$

		1-4-9-14	2-5-10-15	6-7-8-13	4-5	4-7	8-9 (8-9-10-11)	9-10 (8-9-10-11)	10-11 (8-9-10-11)	12-...-17	15-18	16-19	arch	5-53	15-43	17-36	5-15	decking
LRFD	X_{bzt1}		0.09		0.06		0.05		0.01				#DIV/0!	0.06				
			0.07	0.21	0.12	0.00	0.10		0.01	0.66			#DIV/0!					
			0.09	0.14	0.06		0.05	0.05	0.01	0.43		0.00	#DIV/0!		0.06			
	X_{byt1}		0.05		0.00		0.00		0.00				#DIV/0!	0.03				
			0.00	0.00	0.01	0.00	0.00		0.01	0.00			#DIV/0!					
			0.05	0.00	0.00		0.00	0.00	0.00	0.03			#DIV/0!		0.03			

		1-4-9-14	2-5-10-15	6-7-8-13	4-5	4-7	8-9 (8-9-10-11)	9-10 (8-9-10-11)	10-11 (8-9-10-11)	12-...-17	15-18	16-19	arch	5-53	15-43	17-36	17-36	decking
LRFD	X_{bzt2}		0.01		0.06		0.05		0.00				#DIV/0!	0.06				
			0.07	0.21	0.12	0.00	0.10		0.00	0.66			#DIV/0!					
			0.01		0.06		0.05	0.05	0.00	0.43			#DIV/0!		0.06			
	X_{byt2}		-0.02		0.00		0.00		0.00				*#DIV/0!*	0.03				
			0.00	0.00	0.00	0.00	0.00		0.00	0.00			*#DIV/0!*					
			-0.02		0.00		0.00	0.00	0.00	0.03		0.00	*#DIV/0!*		0.03			

Interaction equations for members in bending and compression. Only for members in which the cells are white are these equations applied. For each three lines of X_{bzt} or X_{byt}, the first line represent the span of members closest to the wind source, the second to the middle span, and the third to the span farthest from the wind source. Where the cell is green, the member passed the interaction test and was therefore acceptable. Where yellow, the members passed but only within about twenty percent, and were thus the limiting members.

Interaction Equations

Bending and Axial Compression

$$X_{bc} = \left[\frac{f_c}{F_c'}\right]^2 + \frac{f_{bz}}{F_{bz}'[1-(f_c/F_{cEz})]} + \frac{f_{by}}{F_{by}'\left[1-\left(f_c/F_{cEy}\right)-(f_{bz}/F_{bE})^2\right]} \leq 1.0$$

		1-4-9-14	2-5-10-15	6-7-8-13	4-5	4-7	8-9-10-11			12-...-17	15-18	16-19	arch	5-53	~~15-43~~	17-36	5-43	decking
LRFD	**F_{cEz}**	12046	8412	9995	948	930	1014	1014	20595	1233	6374	20222	6261	421	~~383~~	919	712	1592

$$f_c < F_{cEz} = \frac{.822E'_{min,z}}{(l_e/d)_z^2}$$

		1-4-9-14	2-5-10-15	6-7-8-13	4-5	4-7	8-9-10-11			12-...-17	15-18	16-19	arch	5-53	~~15-43~~	17-36	5-15	decking
LRFD	**F_{cEy}**	12046	8412	9995	384	377	411	411	8340	663	2581	8189	6664	421	~~383~~	308	712	645

$$f_c < F_{cEy} = \frac{.822E'_{min,y}}{(l_e/d)_y^2}$$

		1-4-9-14	2-5-10-15	6-7-8-13	4-5	4-7	8-9-10-11			12-...-17	15-18	16-19	arch	5-53	~~15-43~~	17-36	5-15	decking
Quotients	**1**	0.039703477	0	0.0162991	1.6741E-05	0.001776455	0.0012083	0.001403	0	0.0099779	0.0098399	0.0027342	0.014298	0.000351	~~0.0175608~~	0.0525687	~~0.000128~~	0.00002
		0.137453666	0.0005539	0.118967	0	0.004517265	0.0026957	0.003259	0	0.0150617	0.0155185	0.0094636	0.030972			0.10786		
		0.039703477	0	0.045682	0	0.001716747	0.0011741	0.001362	0	0.0100255	0.0096375	0.0094636	0.013909	~~0.014668~~	~~0.0175608~~	0.0517928	0.000655	
	2	0.047978727	0.0424355	0.1459101	0.058936485	0.00022076	0.0493373	0.049466	0.002742	0.453149	0	0	0.297807	0.313774	~~0.066502~~	0.7403116	~~0.310629~~	0.458364
		0.069750941	0.0676912	0.2182512	0.120747096	0.0002263	0.1031703	0.103695	0.002742	0.7138127	0.0004449	0	0.367984			0.8722609		
		0.047230618	0.0417966	0.1448206	0.058712787	0.000220606	0.0493138	0.049439	0.002742	0.4532564	0.0004391	0	0.294401	~~0.065605~~	~~0.066502~~	0.7383954	0.019884	
	3	0	0.0068748	0	0	0	0	0	0	0.0301232	0	0	0	0.019364	~~0.0401928~~	0.0517415	~~0.019581~~	0.001959
		0	0.0004184	0	0	0	0	0	0	0.0002264	0	0	0			0.0004655		
		0	0.0062855	0	0	0	0	0	0	0.0297155	0	0	0	~~0.039671~~	~~0.0401924~~	0.0046042	0.045436	
LRFD	**X_{bc}**	0.09	0.05	0.16	0.06	0.00	0.05	0.05	0.00	0.49	0.01	0.00	0.31	0.33	~~0.12~~	0.84	~~0.33~~	0.46
		0.21	0.07	0.34	0.12	0.00	0.11	0.11	0.00	0.73	0.02	0.01	0.40			0.98		
		0.09	0.05	0.19	0.06	0.00	0.05	0.05	0.00	0.49	0.01	0.01	0.31	~~0.12~~	~~0.12~~	0.79	0.07	

Appendix K3: Deflection Calculations

Calculations used to find deflections of members loaded on the outer spans (1 and 3) and inner span (2). Deflections were calculated for three different loading cases. All loaded members passed in each of the cases considered.

Span 1, 3		12-...-17						
		12-13	13-14	14-15	15-16	16-17	17-36	
	w_L	29.43	29.43	29.43	29.43	29.43	29.43	lb/in
	w_D	4.77	4.77	4.77	4.77	4.77	4.77	lb/in
	w_T	50.38	50.38	50.38	50.38	50.38	50.38	lb/in
	$L_{unsuppd}$	*6.12*	85.79	85.79	*28.76*	*47.01*	120.50	in
	E	1.14E+06	1.14E+06	1.14E+06	1.14E+06	1.14E+06	1.14E+06	lb/in^2
	I	193.36	193.36	193.36	193.36	193.36	392.9635	in^4
	K	0.50	0.50	0.50	0.50	0.50	0.50	
	Δ_L	0.000023	0.0942	0.0942	0.0012	0.0085	**0.1804**	in
	Δ_{KD+L}	0.000025	0.1018	0.1018	0.0013	0.0092	**0.1950**	in
	Δ_T	0.000040	0.1612	0.1612	0.0020	0.0145	**0.3087**	in
max	Δ_L	0.017002	0.238301	0.238301	0.07988	0.130578	0.334722	in
max	Δ_{KD+L}	0.025502	0.357452	0.357452	0.11982	0.195867	0.502083	in
max	Δ_T	0.025502	0.357452	0.357452	0.11982	0.195867	0.502083	in

Span 2		50-...-55						
		50-51	51-52	52-53	53-54	54-55	55-74	
	w_L	46.01	46.01	46.01	46.01	46.01	46.01	lb/in
	w_D	4.78	4.78	4.78	4.78	4.78	4.78	lb/in
	w_T	76.09	76.09	76.09	76.09	76.09	76.09	lb/in
	$L_{unsuppd}$	*6.12*	85.79	85.79	*28.76*	*47.01*	120.50	in
	E	1.14E+06	1.14E+06	1.14E+06	1.14E+06	1.14E+06	1.14E+06	lb/in^2
	I	193.36	193.36	193.36	193.36	193.36	392.9635	in^4
	K	0.50	0.50	0.50	0.50	0.50	0.50	
	Δ_L	0.000037	0.1472	0.1472	0.0019	0.0133	**0.2820**	in
	Δ_{KD+L}	0.000039	0.1549	0.1549	0.0020	0.0140	**0.2966**	in
	Δ_T	0.000061	0.2435	0.2435	0.0031	0.0219	**0.4663**	in
max	Δ_L	0.017002	0.238301	0.238301	0.07988	0.130578	0.334722	in
max	Δ_{KD+L}	0.025502	0.357452	0.357452	0.11982	0.195867	0.502083	in
max	Δ_T	0.025502	0.357452	0.357452	0.11982	0.195867	0.502083	in

Appendix L: Foundation Assessment Letter of Approval

HARVEY & TRACY ASSOCIATES, Inc.
143 DEWEY STREET
WORCESTER, MA 01610

Tel: 508 757 1354

December 5, 2013

Mr. Robert Antonelli
Assistant Commissioner
Department of Public Works and Parks
City of Worcester
50 Skyline Drive
Worcester, MA 01604

Re: Reconstruction of Wood Bridge
Elm Park
City of Worcester
Worcester, MA

Dear Mr. Antonelli,

On November 22, 2013, I met Tim Boucher at Elm Park so that I could review and give him an assessment of the conditions of the foundations at the two ends of the previously removed wood bridge.

My assessment at that time, based on visual observation, was that the foundations appear in sound condition and capable of supporting the re-fabricated bridge.

It also appears that the existing anchor bolts are in sound condition and can be utilized to anchor the support posts of the re-fabricated bridge.

However it appears that the grout around these anchor bolts should be carefully removed so as to not damage either the anchor bolts or the concrete foundation assemblies.

The question remains as to how to best prepare bearing surfaces to receive each of the wood posts.

To accomplish this, I suggest that you put me in contact with appropriate people at WPI and Worcester Technical High School so that collectively we can address post supports in a way comfortable for all.

Reconstruction of Wood Bridge
Elm Park
City of Worcester

December 5, 2013

Page 1 of 2

HARVEY & TRACY ASSOCIATES, Inc.
143 DEWEY STREET
WORCESTER, MA 01610

Tel: 508 757 1354

Should you have any questions, please feel free to contact me at your convenience.

Respectfully submitted,
HARVEY & TRACY ASSOCIATES, Inc.

Francis S. Harvey, Jr., P.E.

Appendix M: Abutment Design Hand Calculations

Retaining Wall (Abutment) Design 1

Preliminary Design Considerations:

Check: overturning, sliding, foundation bearing pressure.

Factor of Safety: 1.5

Active Pressure: 30 psf / foot of depth

Assumptions on backfill soil

Well-graded, clean gravel; gravel/sand mix

Assumed Parameters per Mass DOT E 305.30

$\phi = 37^\circ$

$\gamma = 120 pcf$

δ (wall friction angle) $= 22^\circ$

$f'_c = 4000$ psi

Foundation Soil properties:

q_f = factored bearing capacity = 9400 psf

Friction factor = .57

Φ sliding = .80

$F_y = 60000$ psi

Normal wt. Concrete: 150 pcf.

Landing Dimensions

5 ft by 13.0625 ft (156.75 in) Area = 65.3125 ft^2

Height from Top of decking to old foundation w/o old abutment

Roughly 56.57" or 4.714 ft

Max Surcharge load on landing: (Per IBC Section 1607) & mass DOT Standards

Mass DOT: 240 psf. (Uniform)

IBC: Snow Load Mass: 50 psf

Dead load slab: 150 pcf × 32.65 ft³ = 4898 lbs/65.3 = 75 psf

Live Load: Public rooms serving people: 100 psf

Total load = 225 psf so 240 psf is reasonable

Min Thickness of one way Slabs to avoid deflection calculations

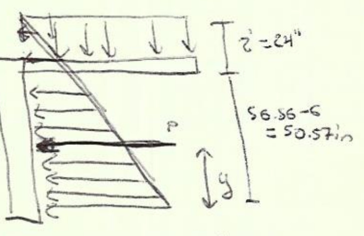
Solid one way slabs: Cantilever $L/10 = 56.57/10 = 5.657$ in

Assume a thickness of 6 in all around.

Design

$$K_{ah} = \frac{1-\sin\Phi}{1+\sin\Phi} = \frac{1-\sin 37}{1+\sin 37} = .2485$$

$$h' \text{ (equivalent height of surcharge)} = \frac{S}{\omega} = \frac{240 psf}{120 pcf} = 2'$$



$$y = \frac{h^2 + 3hh'}{3(h+2h')} = \frac{50.57^2 + 3(50.57)(24)}{3(50.57 + 2(24))} = 20.96"$$

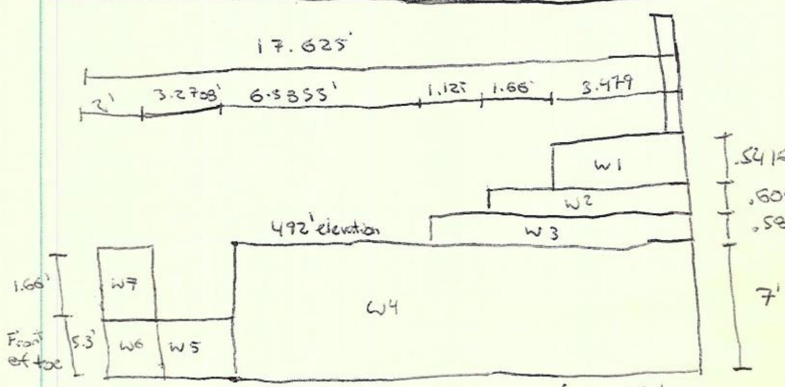
$$P = \tfrac{1}{2} k_{ah} \omega h (h+2h') = \tfrac{1}{2}(.2485)(120 pcf)(4.214 in)(4.214 + 2(2))$$

$$P = 516.122 \text{ lbs}$$

Moment overturning

$M_O = 516.122$ lb × 1.74666 ft = 901.4586 ft·lbs

Components For moment restoring on foundation



* Assume Depth of foundation = 7 feet from 492'

☆ Assume voids filled w/ soil have same unit weight: Concrete

$W_1 = .5416\text{ft} \times 3.479\text{ft} \times 150\text{ psf} = 283\text{ lbs}$
acting $17.625 - (3.479/2) = 15.8855$ feet away

$W_2 = .60416 \times 4.6456 \times 150 = 421\text{ lbs}$
acting $17.625 - (4.6456/2) = 15.3022$ feet away

$W_3 = .5833 \times 5.7706 \times 150 = 504\text{ lbs}$
acting $17.625 - (5.7706/2) = 14.739'$ feet away

$W_4 = 7 \times 12.3539 \times 150 = 12{,}971\text{ lbs}$
acting $17.625 - (12.359/2) = 11.445$ feet away

$W_5 = 3.2708 \times 5.333 \times 150 = 2616.47\text{ lbs}$
acting $5.2708 - (3.2708/2) = 3.635'$ feet away

$W_6 = 2 \times 5.333 \times 150 = 1600\text{ lbs}$
acting 1 foot away

$W_7 = 2 \times 1.656 \times 150 = 500\text{ lbs}$
acting 1 foot away

$W_8 = .5 \times 4.714 \times 150 = 353\text{ lbs}$
acting $17.625 - .25 = 17.375$ feet away

MR about front edge of foundation:

Component weights	w (lb)	x (ft)	Mr = xw (ft·lb)
W_1	283	15.8855	4496
W_2	421	15.8855	6443
W_3	504	14.739	7428
W_4	12,971	11.445	148,459
W_5	2616	3.635	9510
W_6	1600	1	1600
W_7	500	1	500
W_8	353	17.375	6133
	19 248		184,572

3

Distance Resultant from front edge:

$$a = \frac{184.572 - 901.458}{19,248} = 9.54$$ feet from frond edge (middle third of footing)

Factor of Safety against overturning

$184,572./901.458 = 204.75$ (ample)

From Fig. 17.5a (Resultant in middle third):

Max Soil pressure

$$q_1 = (4L - 6a)\left(\frac{Rv}{L^2}\right) = \left(4(17.625) - 6(9.54)\right)\left(\frac{19.248}{(17.625)^2}\right) = 821.58 \text{ psf}$$

$$q_2 = (6a - 2L)\left(\frac{Rv}{L^2}\right) = 6(9.54) - 2(17.625)\left(\frac{19.248}{(17.625)^2}\right) = 1362.5 \text{ psf}$$

Factor of Safety = 5200/1362.5psf = 3.81 ✓ good

Sliding factor (Assume no hydrostatic or seismic forces)

$f = .57 \times 19,248 = 10,971.36$ lb

Factor of Safety against sliding
= 10,971.36 / 901.45 = 12.17 ✓ good

External Stability ✓

Internal Stability

1

Initial Assumptions:

$f'_c = 4000$ psi $\quad f_y = 60{,}000$ psi $\qquad$ Table 1.3 gives $\phi_T = .9$
h (thickness) $= 6$ in
L (height) $= 56.57$ in

Evaluate for unit thickness, $b = 12$in

Factored Load Combination

$w_u = 1.6H = 1.6\,(516.122\text{lb}) = 825.79$ lbs

$M_u = 825.79\text{lbs} \times 1.746\text{ft} = 1443$ ft-lbs @bottom of wall

Effective Depth

Assuming $h = 6$in; $d = h - 2.5 = 6 - 2.5 = 3.5"$ ← Reasonable guess for 1.5" cover, .5d_b distance

$\rho_{min} = 3\frac{\sqrt{f'c}}{f_y} = .0033$

$\rho_{max}\,(\varepsilon_T = .005) = .0181$ (Per table A.4)

$\phi M_n = \phi \rho f_y b d^2 \left(1 - .59\frac{\rho f_y}{f'c}\right)$

$1443 = .9\,\rho\,60{,}000\,(12)(3.5)^2\left[1 - .59\,\frac{\rho\,60{,}000}{4000}\right]$

$\rho = .0022 < \rho_{min}$ or $\rho = .1107 > \rho_{max}$ $\qquad$ use $\rho_{min} = .0033$ to be economical

Check Shear

$P = \frac{1}{2}(.2485)(120\text{pcf})(4\text{ft})(8\text{ft}) = 477.12$ lb

$V_u = 1.6 \times 477.12\text{lb} = 763.392$ lb

$\phi V_c = \phi 2 \lambda \sqrt{f'c}\, bd = (.75)(2)(1)\sqrt{4000}\,(12)(3.5) = 3984$ lb

Factor of Safety

$\frac{3984\text{lb}}{763.39} = 5.22$ ✓ good

∴ Calculate required As @ bottom of footing

$A_s = \rho bd = .0033 * (12\text{in})(3.5\text{in}) = .1386\text{ in}^2$

Spacing of bars cannot exceed $3 \times h$ or 18in; so use 18in
To be conservative; place #5 bars @ 10in on center

Temp + Shrinkage Reinforcement (Horizontal) (ACI 14.3.3)

$b = 12$in
Min ratio (Table 13.2) = .0018
ACI Code 14.3.3:
.0020, bars not larger than #5

$A_s = \rho A_c = .0020(12\text{in})(6\text{in}) = .1296\text{ in}^2/\text{ft}$

Use #4 bars @ 14in on center

2

<u>Development Length for dowels Per ACI code 12.2.3</u>

$$l_d = \left(\frac{f_y \psi_t \psi_e}{25 \lambda \sqrt{f'_c}}\right) .625 \quad (\text{Eqn. } 5.5)$$

$\psi_t = 1.0$; $\psi_e = 1.0$; $\lambda = 1.0$; $f'_c = 4000$ psi ; $f_y = 60{,}000$ psi ; d_b = #5 bars = .625 in

$\therefore l_d = 24'' > 12''$ so good

<u>Development Length for Hooked Bars</u>

$$l_{dh} = \left(\frac{0.02 \ \psi_e f_y}{\lambda \sqrt{f'_c}}\right) d_b > 12 d_b \quad (\text{Eqn. } 5.6)$$

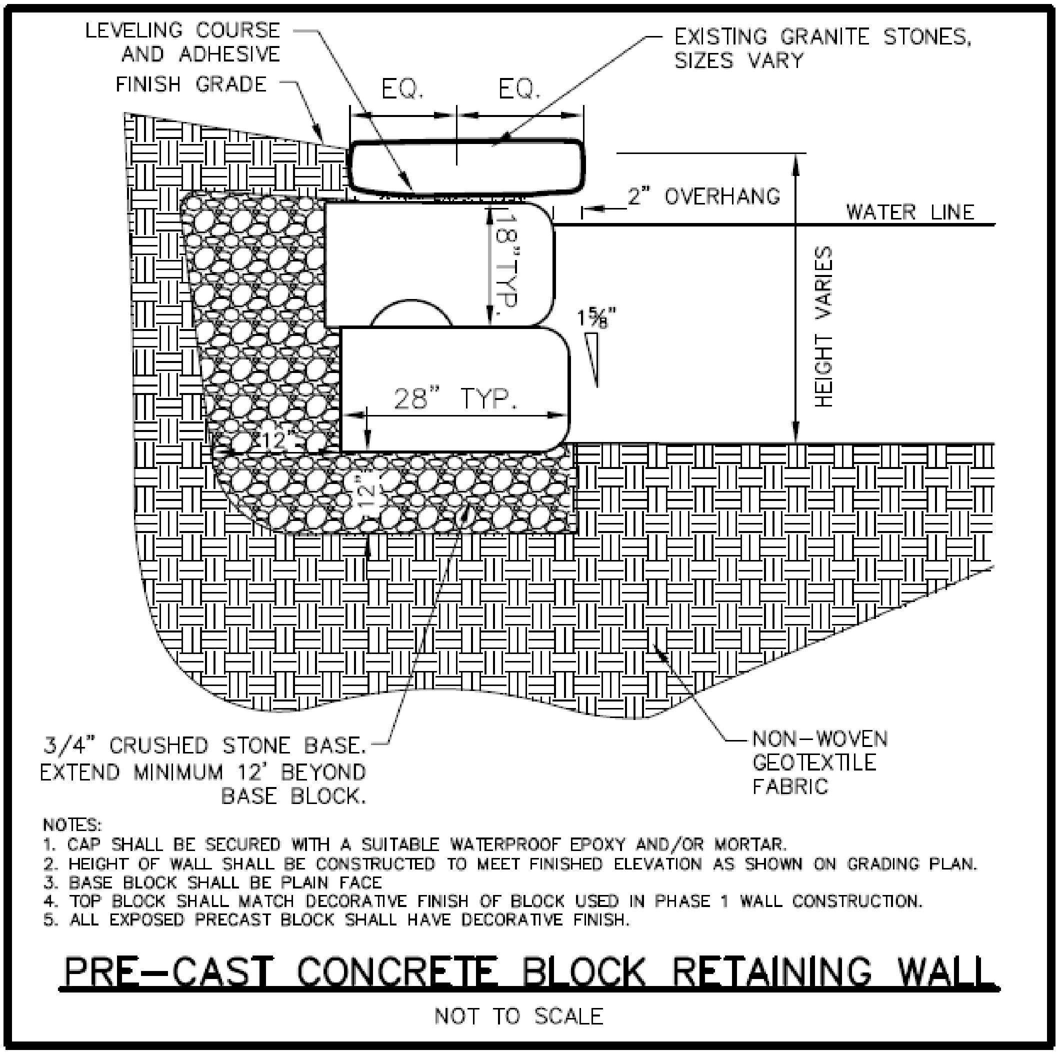
$\psi_e = 1.0$ $f_y = 60{,}000$ psi $\lambda = 1.0$ $f'_c = 4000$ psi $d_b = .5$ in

$$l_{dh} = \left(\frac{.02 \cdot 60{,}000}{\sqrt{4000}}\right) .5 = 9.48\text{in} > 6\text{in}$$ <u>use 10in</u>

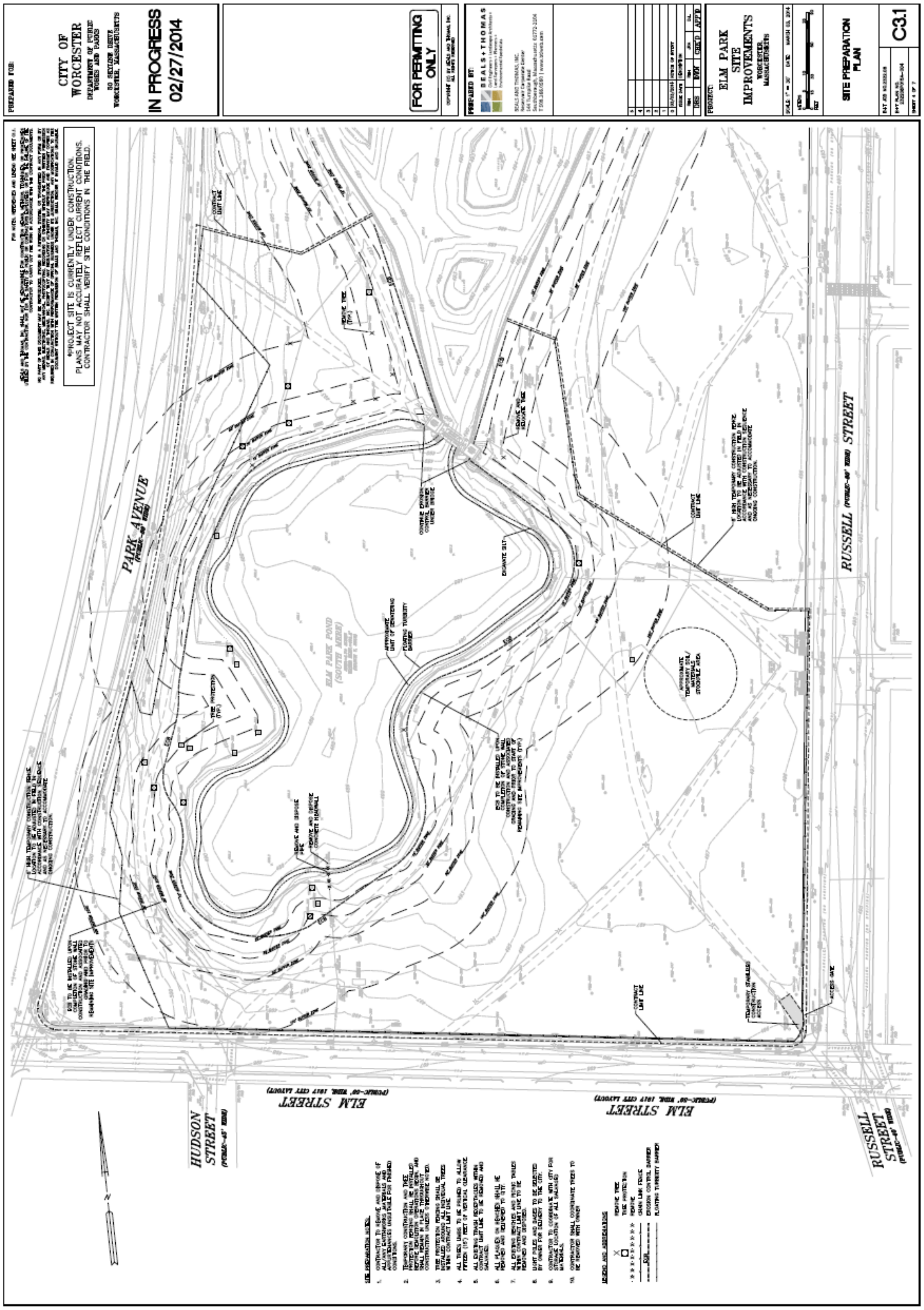
---

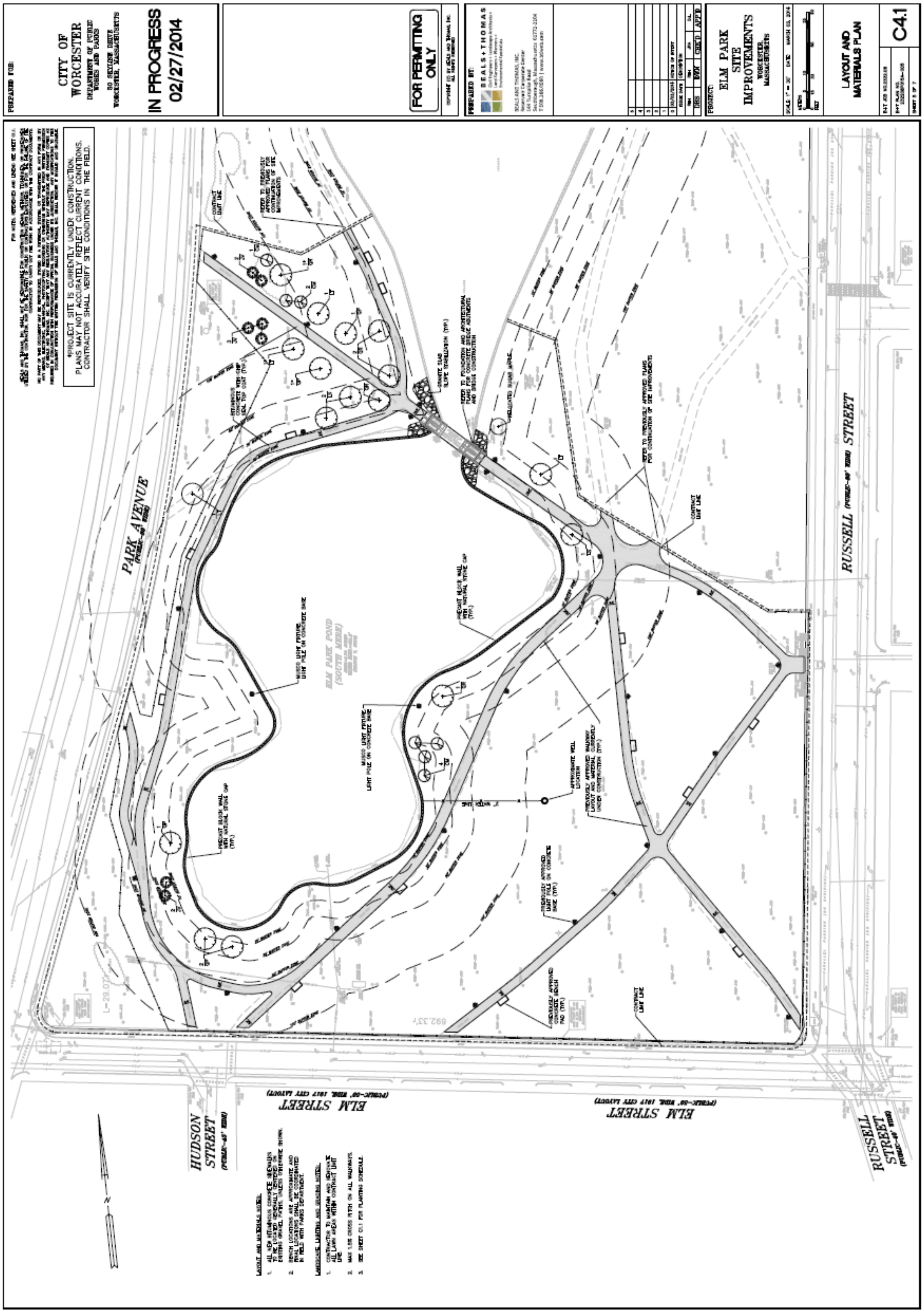
* Maintain 1½ in cover everywhere in abutment (ACI code 7.7)

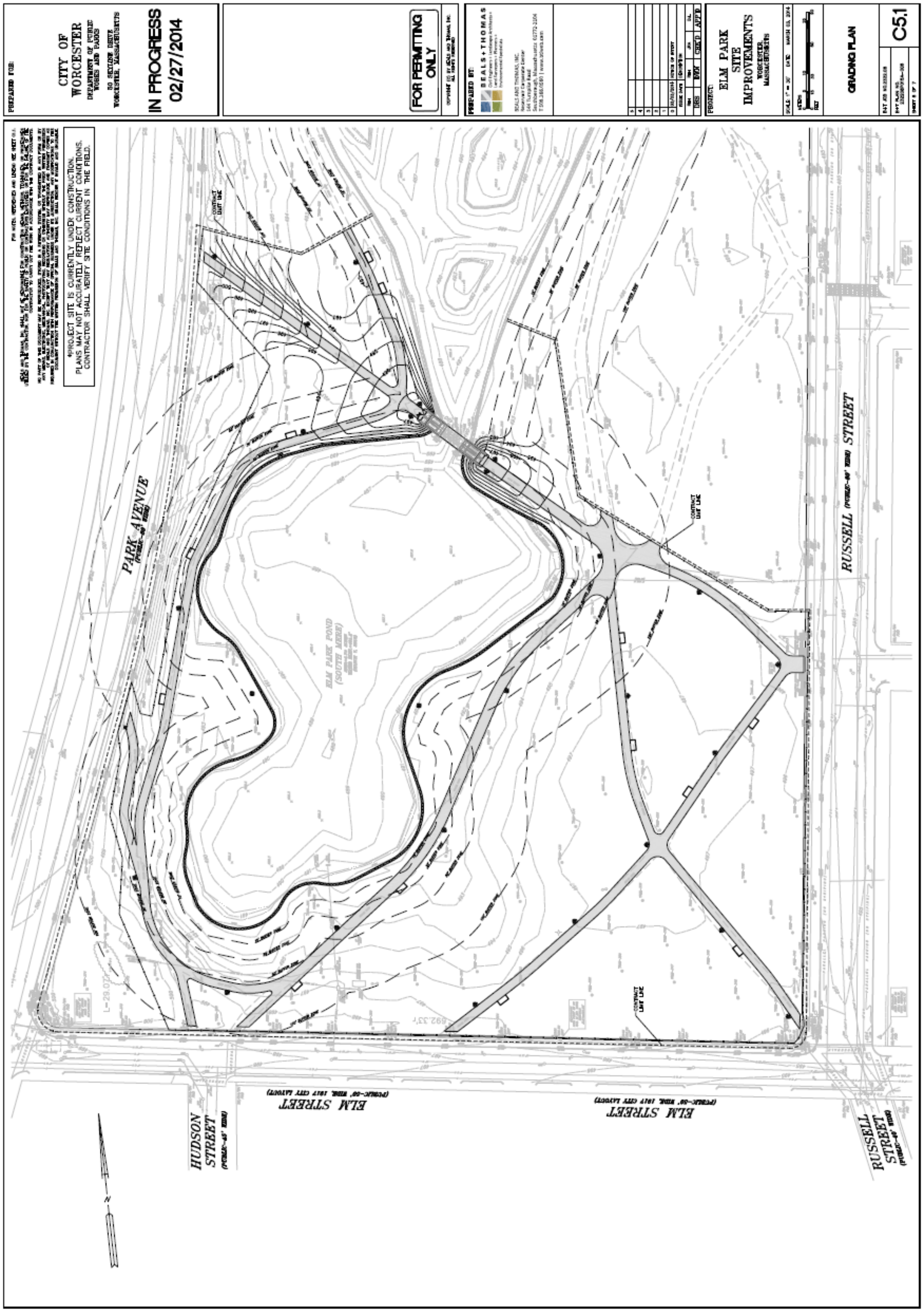
Appendix N: Pond Wall Detail Drawing



Appendix O: Elm Park Final Site Plans




PREPARED FOR:
CITY OF WORCESTER
DEPARTMENT OF PUBLIC WORKS AND PARKS
IN PROGRESS 02/27/2014
FOR PERMITTING ONLY
PREPARED BY:
BEALS + THOMAS
PROJECT:
ELM PARK SITE IMPROVEMENTS
WORCESTER, MASSACHUSETTS
LAYOUT AND MATERIALS PLAN
C4.1
PARK AVENUE
RUSSELL STREET
ELM STREET
ELM STREET
HUDSON STREET
RUSSELL STREET
ELM PARK POND (SOUTH MEER)


PREPARED FOR:
CITY OF WORCESTER
DEPARTMENT OF PUBLIC WORKS AND PARKS
IN PROGRESS 02/27/2014
FOR PERMITTING ONLY
PREPARED BY:
BEALS + THOMAS
PROJECT:
ELM PARK SITE IMPROVEMENTS
WORCESTER, MASSACHUSETTS
GRADING PLAN
C5.1
PROJECT SITE IS CURRENTLY UNDER CONSTRUCTION. PLANS MAY NOT ACCURATELY REFLECT CURRENT CONDITIONS. CONTRACTOR SHALL VERIFY SITE CONDITIONS IN THE FIELD.
PARK AVENUE
RUSSELL STREET
ELM STREET
HUDSON STREET
ELM PARK POND (SOUTH MERE)
CONTRACT LIMIT LINE

Appendix P: Site and Abutment Cost Analysis

Site Material Costs

Volume of Steel and Length in liner ft

#4 Bar $\phi = .5''$ $\phi/2 = .25''$

#5 Bar $\phi = .625''$ $\phi/2 = .3125''$

$V_s = \pi(\phi/2)^2 \cdot \text{Length}$ $\quad L = \dfrac{\#\text{ of Bars} \times \text{Length(in)}}{12\text{in}}$

#4 Bar V_s

Sides: $2[7\text{Bars} \times \pi(.25'')^2 \times 68''] = 186.925\text{ in}^3$

Wall: $4\text{Bars}[\pi(.25'')^2 \times 153.75''] = 120.755\text{ in}^3$

#4 Total Volume = 307.68 in^3

#4 Bar L.F.

Sides: $2[7\text{Bars} \times 68''] = 952\text{ in}$

Wall: $[4\text{Bars} \times 153.75''] = 615\text{ in}$

#4 Total L. inches = 1567 $\quad \therefore$ L.F. = 130.58 ft. of #4

#5 Bar V_s

Sides: $2[6\text{ Bars} \times \pi(.3125'')^2 \times 110''] = 404.971\text{ in}^3$

Wall: $16\text{Bars}[\pi(.3125'')^2 \times 48''] + 16\text{Bars}[\pi(.3125'')^2 \times 42.08''] = 442.179\text{ in}^3$

Landing: $6\text{Bars}[\pi(.3125'')^2 \times 150.625''] + 16\text{Bars}[\pi(.3125'')^2 \times 51''] = 527.613\text{ in}^3$

#4 Total Volume = 1374.76 in^3

#5 Bar L.F.

Sides: $2[6\text{Bars} \times 110''] = 1320\text{ in}$

Wall: $[16\text{Bars} \times 48''] + [16\text{Bars} \times 42.08''] = 1441.28\text{ in}$

Landing: $[6\text{Bars} \times 150.625''] + [16\text{Bars} \times 51''] = 1719.75\text{ in}$

#5 Total L. inches = 4481.03 in $\quad \therefore$ L.F. = 373.41 ft of #5

Total Volume of Steel

$[\#4\ V_s = 307.68\text{ in}^3] + [\#5\ V_s = 1374.76\text{ in}^3]$

$= 1682.44\text{ in}^3 = .973$ C.F. of Steel (1/2 Abutment)

Volume of Concrete

Sides : $2[98.57" \times 57" \times 6"] = 67421.9 in^3$

Wall : $[156.75" \times 50.57" \times 6"] - [156.75" \times 5.5" \times 3"] = 44974.7 in^3$

Landing : $[156.75" \times 60" \times 6"] = 56430 in^3$

Total = $168827 in^3$ = 97.70 C.F.

Volume of Concrete = Concrete − Steel

Total Concrete = 97.70 C.F. − .973 C.F. = 96.72 C.F. of Concrete

Convert C.F. ~~of~~ of Concrete to S.F. with 6" depth

96.72 C.F. → 193.45 S.F. (1/2 Abutment)

Volume of Fill/cut

	Fill C.Y.	Cut C.Y.
AoI 1	1448.91	250.13
AoI 2	359.73	4.82
Total	1808.64	(−) 254.95

Total = 1553.69 C.Y. of Fill

Cost of Materials

Concrete — \$2.03 / S.F.
#4 Rebar — \$0.71 / L.F.
#5 Rebar — \$1.09 / L.F.
Fill — \$28.00 / C.Y.

Concrete : 193.46 S.F. × \$2.03/SF = \$392.72

Steel : #4 130.58 L.F. × \$0.71/LF = \$92.71
#5 373.41 L.F. × \$1.09/LF = \$407.02
Total = \$92.71 + \$407.02 = \$499.73

* Costs above are for one abutment; total abutment costs:

2[\$392.72 + \$499.73] = \$1784.91

Fill : 1553.69 C.Y. × \$28.00/C.Y. = \$43,503.30

Total Site Material Costs

(Abutment = \$1784.91) + (Fill = \$43,503.30) = \$45,288.20

Summary of Materials / costs

Abutment total Cost : $1,784.91

→ Abutment Concrete costs : $785.44 for 386.93 S.F. of 6" Concrete

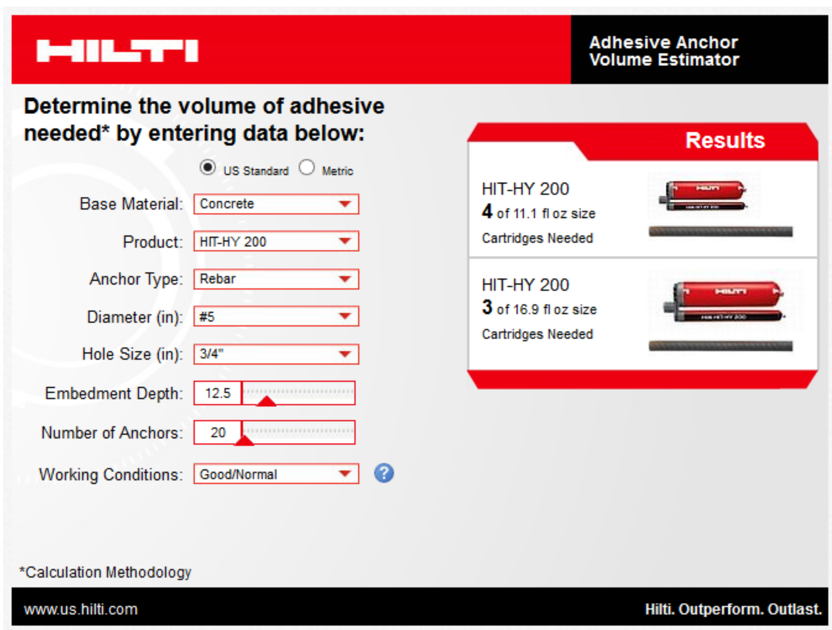
→ Abutment Steel Costs : $999.46

→ #4 Rebar cost : $185.42 for 261.16 L.F of #4

→ #5 Rebar Cost : $814.04 for 746.82 L.F of #5

Fill total Cost : $43,503.30 for 1553.69 C.Y. of fill

Total Material Cost : $45,288.20



Appendix Q: Site Design Cut and Fill Data

Cut inside of wall, bottom of wall at 490.7

Total Volumes Report
232208PD001

Beals and Thomas, Inc.
144 Turnpike Road
Southborough, MA

February 26, 2014
11:25 AM

233208PD001: Existing vs. Proposed

Excavation Volumes	Bank Cut (cu yd)	Bank Fill (cu yd)	Cut Swell (%)	Fill Shrink (%)	Adjusted Cut (cu yd)	Adjusted Fill (cu yd)
AOI - Inside of Wall	437.30	7.02	0.00	0.00	437.30	7.02

Excavation Areas	Cut Area (sq ft)	Fill Area (sq ft)	Total Area (sq ft)
AOI - Inside of Wall	24030.00	1234.00	93060.00

Project Totals

Excavation Volumes	Bank Cut (cu yd)	Bank Fill (cu yd)	Cut Swell (%)	Fill Shrink (%)	Adjusted Cut (cu yd)	Adjusted Fill (cu yd)
233208PD001 : AOI - Inside of Wall	437.30	7.02	0.00	0.00	437.30	7.02
Project Totals:	437.30	7.02	0.00	0.00	437.30	7.02
Export:	430.27				430.27	

Excavation Areas	Cut Area (sq ft)	Fill Area (sq ft)	Total Area (sq ft)
233208PD001 : AOI - Inside of Wall	24030.00	1234.00	93060.00
Project Totals:	24030.00	1234.00	93060.00

Removal Quantities	Boundary (ft)	Area (sq ft)	Volume (cu yd)
Project Totals:	0.00	0.00	0.00

* Only Drawing Master AOI's or single selected Drawing AOI's are used for Project Excavation Totals.

* Removal Quantities are clipped to Drawing Master AOI or the individually selected AOI.

Total Volumes Report

Beals and Thomas, Inc.
144 Turnpike Road
Southborough, MA

February 21, 2014
11:37 AM

233208PD002: Existing vs. Proposed

Excavation Volumes	Bank Cut (cu yd)	Bank Fill (cu yd)	Cut Swell (%)	Fill Shrink (%)	Adjusted Cut (cu yd)	Adjusted Fill (cu yd)
AOI Bridge 1 of 2	250.13	1448.91	0.00	0.00	250.13	1448.91
AOI Bridge 2 of 2	4.82	359.73	0.00	0.00	4.82	359.73
		1808.64 total				

Excavation Areas	Cut Area (sq ft)	Fill Area (sq ft)	Total Area (sq ft)
AOI Bridge 1 of 2	5924.59	26427.41	32352.00
AOI Bridge 2 of 2	656.10	5527.90	6184.00

Project Totals

Excavation Volumes	Bank Cut (cu yd)	Bank Fill (cu yd)	Cut Swell (%)	Fill Shrink (%)	Adjusted Cut (cu yd)	Adjusted Fill (cu yd)
Project Totals:	0.00	0.00	0.00	0.00	0.00	0.00

Excavation Areas	Cut Area (sq ft)	Fill Area (sq ft)	Total Area (sq ft)
Project Totals:	0.00	0.00	0.00

Removal Quantities	Boundary (ft)	Area (sq ft)	Volume (cu yd)
Project Totals:	0.00	0.00	0.00

* Only Drawing Master AOI's or single selected Drawing AOI's are used for Project Excavation Totals.

* Removal Quantities are clipped to Drawing Master AOI or the individually selected AOI.

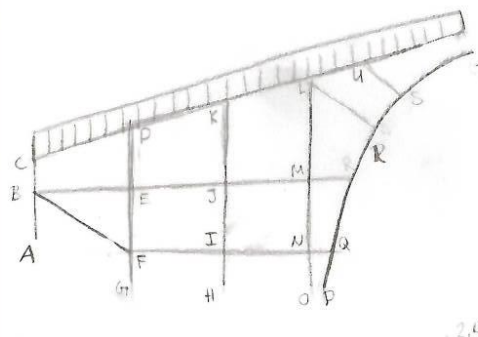
Appendix R: Hand Calculation Results

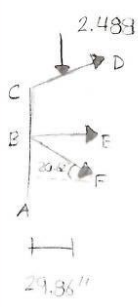
Appendix R1: Initial Full Span Hand Calculations

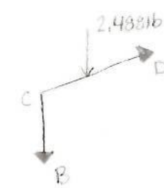
Courtney's

Pg. 1

$1\,plf \cdot \frac{244.8\,in}{12\,in} = 20.4\,ft$



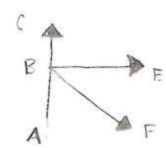




$\Sigma F_y = -2.488lb + CB$

$CB = \frac{2.488lb}{2} = 1.244\,lb = 1724.184lb$

$\Sigma F_x = 0$



$\Sigma F_y = -(CB \cdot 2) + BF \sin 20.62 + BA$

$BA = 2.448lb - .8517lb$

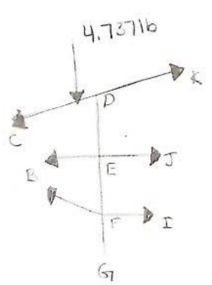
$BA = \frac{1.591b}{2} = .798lb = 1106.028$

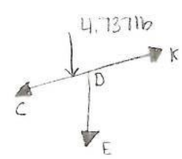
$\circlearrowleft M_B = 2.488lb\,(29.86/12\,ft) - BF \cos 20.62\,(32.82/12')$

$BF = 2.418\,lb = 3351.348lb$

$\Sigma F_x = BF \cos 20.62 - BE$

$BE = 2.263\,lb = 3136.52\,lb$





$\Sigma F_y = DE - 4.737lb$

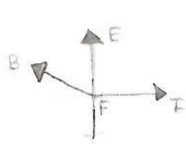
$DE = \frac{4.737lb}{2} = 2.369lb = 3282.041b$

$\Sigma F_y = -DE + EF$

$EF = 2.368lb = 3282.0431b$

$\Sigma F_x = BE - EJ$

$EJ = 2.263lb = 3136.518lb$



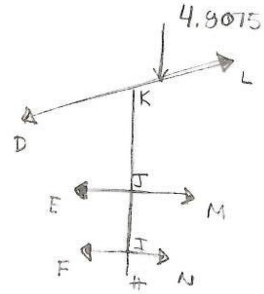
$\Sigma F_y = (2.368\,lb \cdot 2) + (2.418lb \sin 20.62) - FG$

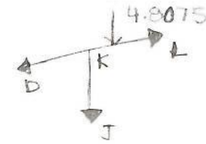
$FG = \frac{5.589lb}{2} = 2.794\,lb = 3872.484$

$\Sigma F_x = 2.418lb \cos 20.62 - FI$

$FI = 2.263lb = 3136.518lb$

pg. 2





$\Sigma F_y = KJ - 4.8075 lb$

$KJ = \frac{4.8075 lb}{2} = 2.403 lb = 3330.558 lb$

$\Sigma F_y = -KJ + JI$

$JI = 2.403 lb = 3330.558 lb$

$\Sigma F_y = -EJ + JM$

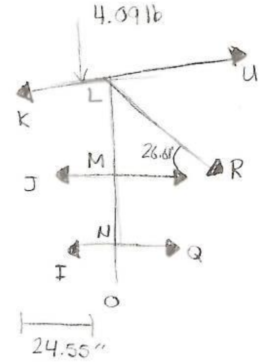
$JM = 2.263 lb = 3136.518$

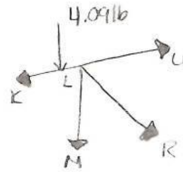
$\Sigma F_y = -JI + IH$

$IH = 2.403 lb = 3330.558$

$\Sigma F_x = -FI + IN$

$IN = 2.263 lb = 3136.518$



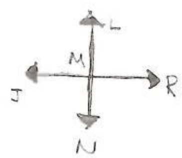


$\Sigma F_y = -4.09 lb + LM + LR \sin 26.61°$

$LM = \frac{3.607 lb}{2} = 1.803 lb = 2498.958$

$\circlearrowleft M_L = -4.09 lb(.5125 ft) + LR \cos 26.61°(2.1775 ft)$

$LR = 1.076 lb = 1491.336 lb$

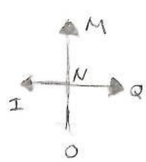


$\Sigma F_y = -LM + MN$

$MN = 1.803 lb = 2498.958 lb$

$\Sigma F_x = -JM + MR$

$MR = 2.263 lb = 3136.518 lb$

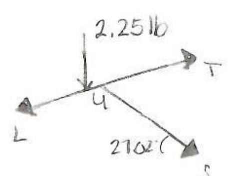


$\Sigma F_y = -MN + NO$

$NO = 1.803 lb = 2498.958 lb$

$\Sigma F_x = -IN + NQ$

$NQ = 2.263 lb = 3136.656 lb$



$\Sigma F_y = -2.25 lb + US \sin 27.02°$

$US = 4.95 lb = 6860.7 lb$

Pg 3

CT

$$m = \frac{wL^2}{8} = \frac{(693\ lb/ft)\left(\frac{1\ ft}{12\ in}\right)(244.8\ in)}{8} = 1767.15\ in\text{-}lb$$

$$C = \frac{7.5\ in}{2} = 3.75\ in$$

$$I = \frac{(3.5\ in)(7.5\ in)^3}{12} = 123.04\ in^4$$

$$F_b = \frac{1767.15\ in\text{-}lb\ (3.75\ in)}{123.04\ in^4} = 53.86\ psi$$

$V = (693\ lb/ft)(20.4\ ft/2) = 7068.6\ lb$

$Q = Ay = 26.25\ in^2 \cdot 3.75\ in = 98.43\ in^3$

$A = 7.5\ in \cdot 3.5\ in = 26.25\ in^2$

$y = 3.75\ in$

$I = 123.04\ in^4$

$b = 3.5\ in$

$$F_v = \frac{(7068.6\ lb)(98.43\ in^3)}{(123.04\ in^4)(3.5\ in)} = 1615.64\ psi$$

center

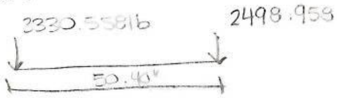
$$m = \frac{wL^2}{8} = \frac{(693\ lb/ft)(1/12\ in)(122.16\ in)}{8} = 881.84\ in\text{-}lb$$

$$F_b = \frac{(881.84\ in\text{-}lb)(3.75\ in)}{123.04\ in^4} = 26.87\ psi$$

$V = (693\ lb/ft)(5.09\ ft) = 3527.37\ lb$

$$F_v = \frac{(3527.37\ lb)(98.43\ in^3)}{(123.04\ in^4)(3.5\ in)} = 806.23\ psi$$

JM



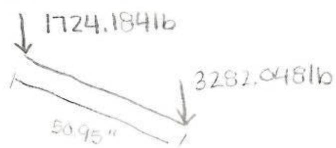
$\circlearrowleft M = -3330.558\,lb\,(25.2\,in) + 2499.958\,lb\,(25.2\,in)$

$M = -20956.32\ in\text{-}lb$

$$F_b = \frac{(20956.32\,in\text{-}lb)(2.75\,in)}{76.25\,in^4} = 755.801\,psi$$

$$F_v = \frac{(3330.558\,lb)(83.1875\,in^3)}{(76.25\,in^4)(5.5\,in)} = 684.47\,psi$$

BF =



$\circlearrowleft M = -1724.184\,lb\,(25.475\,in) + 3282.048\,lb\,(25.475\,in)$

$M = 39686.58\ in\text{-}lb$

$$F_b = \frac{(39686.58\,in\text{-}lb)(2.75\,in)}{76.25\,in^4} = 1431.32\,psi$$

$$F_v = \frac{(3282.048\,lb)(83.1875\,in^3)}{(76.25\,in^4)(5.5\,in)} = 651.029\,psi$$

FI = EJ

IN = JM

CA

$$F = \frac{\pi^2 EI}{(KL)^2} = \frac{\pi^2 \cdot (620\,000\,psi)(48.53\,in^4)}{(1 \cdot 48.15\,in)^2} = 128088.206\,lb \leq 1724.184\,lb$$

$$I = \frac{(3.5\,in)(5.5\,in)^3}{12} = 48.53\,in^4$$

DG

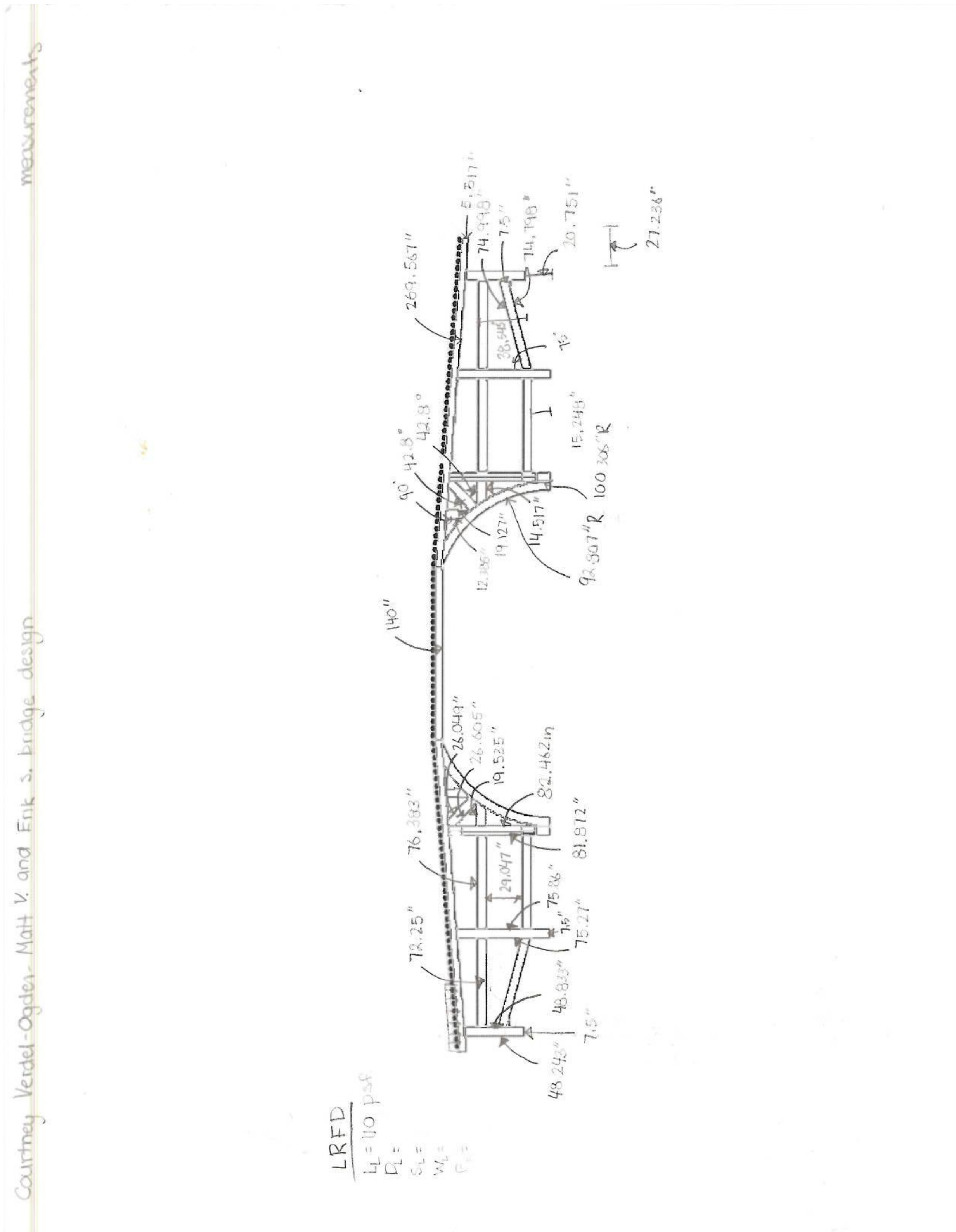
$$F = \frac{\pi^2 EI}{(KI)^2} = \frac{\pi^2 (620000\,psi)(48.53\,in^4)}{(1 \cdot 73.24\,in)^2} = 55{,}361.14\,lb \leq 3872.494\,lb$$

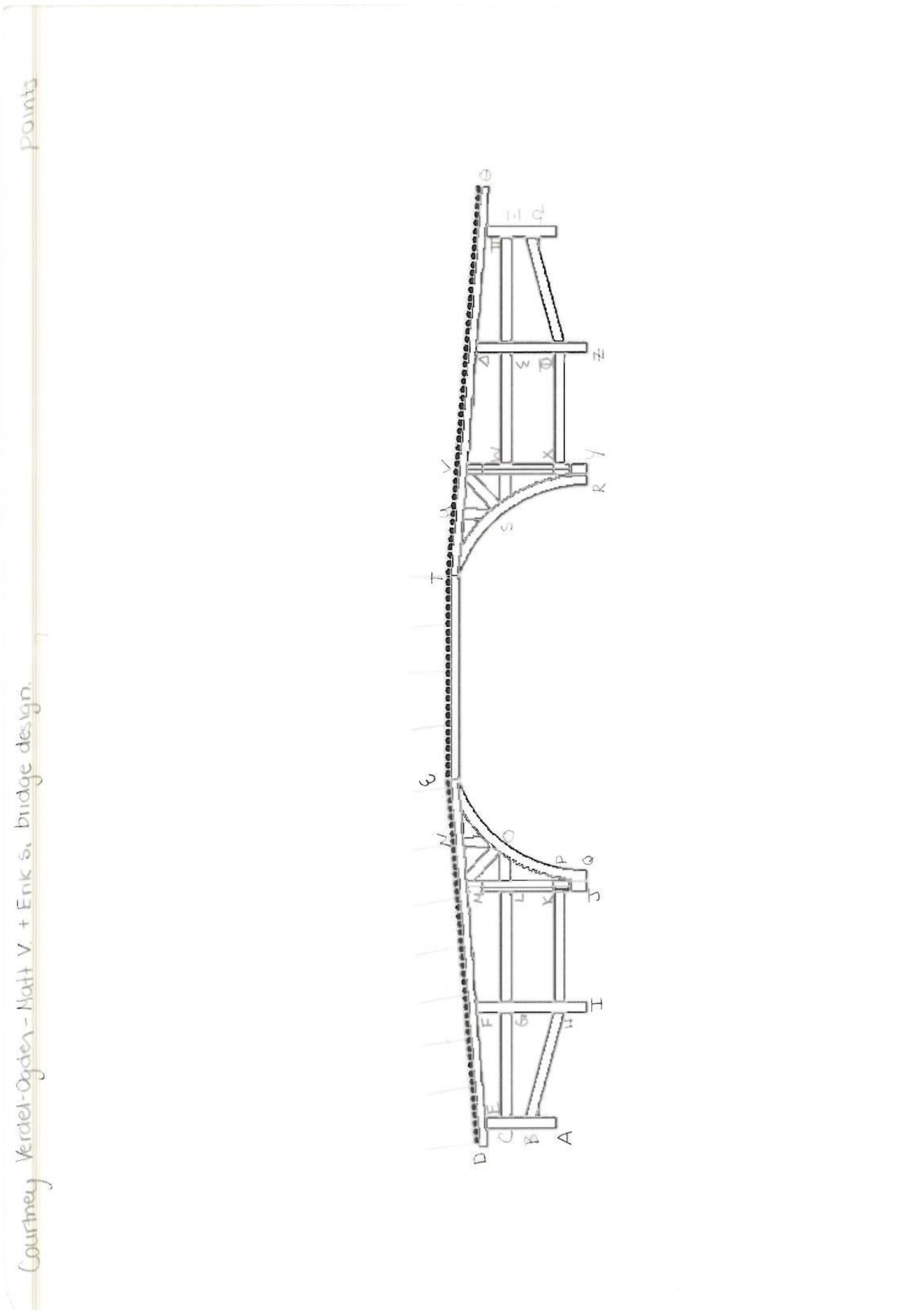
KH

$$F = \frac{\pi^2 (620000\,psi)(48.53\,in^4)}{(1 \cdot 77.56\,in)^2} = 49{,}365.79\,lb \leq 3330.55\,lb$$

LO

$$F = \frac{\pi^2 (620000\,psi)(48.53\,in^4)}{(1 \cdot 82.03\,in)} = 44132.27\,lb \leq 2498.95\,lb$$


Courtney Verdel-Ogden, Matt V. and Erik S. bridge design
measurements
LRFD
L_L = 110 psf
D_L =
S_L =
W_L =
P_L =
48.243"
7.5"
48.833"
72.25"
7.5"
75.27"
75.86"
29.047"
76.383"
81.812"
82.462in
19.525"
26.605"
26.049"
140"
12.386"
19.127"
14.517"
92.807"R
100.306"R
90°
42.8°
42.8°
15.248"
269.567"
38.545
75°
74.998"
7.5"
74.798"
5.317"
20.751"
21.236"


Courtney Verdel-Ogden - Matt V. + Erik S. bridge design.
Points

Loads

Fir - 48 lb/ft^3

279.53 in + 279.53 in + 140 in = 58.255 ft (4in/12) = 19.398 ft^2 walkways

58.255 ft (4in/12)(5.5in/12) = 8.9 ft^3 - supports

19(.44 ft^3) + (58.255ft · .11 ft^2) + (58.255ft · .027ft^2) + 36 (.477 ft^3) = 33.51 ft^3 - rails

(.390625 ft^2)(16.8ft + 7.204ft + 8.202 ft + 4.069ft + 6.322ft + 6.8225 ft + 9.567ft) = 23.04 ft^3

(23.04 ft^3 + 33.51ft^3 + 8.9 ft^3 + 116.388 ft^3)(48 lb/ft^3) = 8727.744 lb → 150.184 lb/ft

1.4D = 1.4 (150 lb/ft) = 210 lb/ft

1.2D + 1.6L + .5S = 1.2 (150 lb/ft) + 1.6(660 lb/ft) + .5 (300 lb/ft) = 1386 lb/ft

1.2D + 1.6S + .5L = 1.2 (150 lb/ft) + 1.6 (300 lb/ft) + .5 (660 lb/ft) = 990 lb/ft

1.2D + 1.3W + .5L + .5S = 1.2 (150 lb/ft) + 1.3 (43.67 lb/ft) + .5 (660 lb/ft) + .5(300 lb/ft) = 716.771 lb

.9D - 1.3W = .9 (150 lb/ft) - 1.3 (43.67 lb/ft) = 191.771 lb/ft

w = 1386 lb/ft

$F_b = \frac{Mc}{I}$ (beam) $F_v = \frac{VQ}{Ib}$ $Q = Ay$

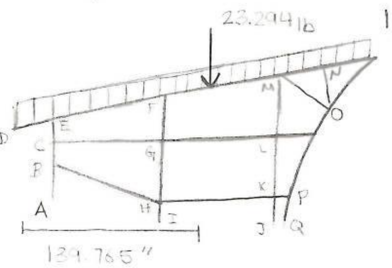
$y = .5y$

Courtney Verdel-Ogden - Erik S. + Matt V. design calculations pg. 1

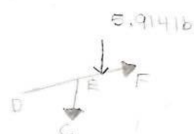
$1.4D$
$1.2D + 1.6L + .5(L_r \text{ or } S \text{ or } R)$
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + .5(L) \text{ or } .8(W)$
$1.2D + 1.3W + .5L + .5(L_r \text{ or } S \text{ or } R)$
$.9D - (1.3W \text{ or } 1.0E)$

D - dead load
L - live load
L_r - live roof load
S - snow load
R - rain load
W - wind load
E - earthquake load

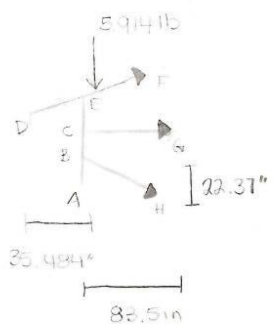
*assuming 1 plf to achieve ratio



$1\,\text{plf} \cdot \frac{279.53''}{12\,\text{in/ft}} = 23.294\,\text{lb}$



$\Sigma F_y = -5.914\,\text{lb} + EC$
$EC = 5.914\,\text{lb} = 8196.804\,\text{lb}$
$\Sigma F_x = 0$

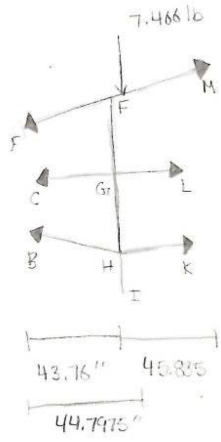


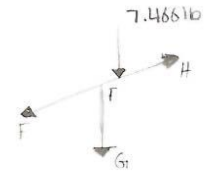
$S\frac{O}{H} \quad C\frac{A}{H} \quad T\frac{O}{A}$

$\Sigma F_y = -EC + CB$
$CB = 5.914\,\text{lb} = 8196.804\,\text{lb}$
$\Sigma F_x = CG$
$CG = 0$

$\Sigma F_y = -CB + BA + BH\cos 75$
$BA + BH\cos 75° = 5.914\,\text{lb}$
$BA = 5.887\,\text{lb} = 8159.382\,\text{lb}$

$\circlearrowright M_C = 5.914\,\text{lb}(.062\,\text{ft}) - BH\sin 75°(3.671\,\text{ft})$
$BH = .103\,\text{lb} = 142.753\,\text{lb}$

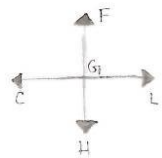




$\Sigma F_y = -7.466 lb + FG$

$FG = 7.466 lb = 10348.015 lb$

$\Sigma F_x = 0$



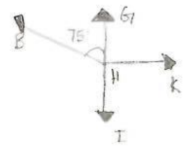
$\Sigma F_y = -FG + GH$

$GH = 7.466 lb = 10347.8761 lb$

$\Sigma F_x = -CG + GL$

$GL = 0$

$\circlearrowright M_G = 0$



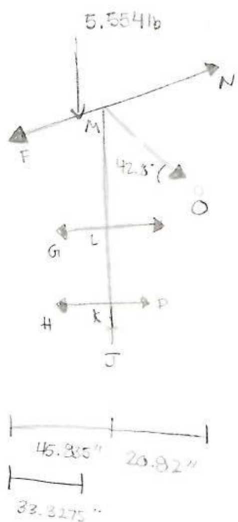
$\Sigma F_y = -GH + HI - BH\cos 75°$

$7.466 lb = HI - BH\cos 75°$

$HI = 7.49 lb = 10381.14$

$\Sigma F_x = BH(\sin 75) - HK$

$HK = .099 lb = 137.2141 lb$



$\Sigma F_y = -5.554 lb + ML + MO\sin 42.8°$

$5.514 lb = ML + MO\sin 42.80°$

$ML = 3.269 lb = 4530.834$

$\circlearrowright M_M = -5.554 lb(1.0422 ft) + MO\cos 42.8(2.389 ft)$

$MO = 3.304 lb = 4579.344 lb$

$\Sigma F_y = -ML + LK$

$LK = 3.269 lb = 4530.834$

$\Sigma F_x = GL - LO$

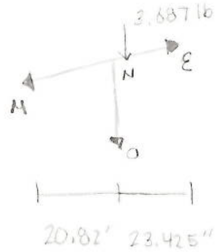
$LO = 0$

$\Sigma F_y = -LK + KJ$

$KJ = 3.269 lb = 4530.834$

$\Sigma F_x = HK - KP$

$KP = .099 lb = 137.214 lb$



$\Sigma F_y = -3.687 + NO$

$NO = 3.687\text{lb} = 5110.182\text{lb}$

$\Sigma F_x = 0$

moment

DE

E - Fir $1.29 \cdot 10^6$ psi

$$M = \frac{wL^2}{8} = \frac{1386\text{ lb/ft}\left(\frac{1\text{ft}}{12\text{in}}\right)(279.53\text{ in})^2}{8} = 4035.714\text{ in-lb}$$

$C = 5.517\text{ in}/2 = 2.758\text{ in}$

$$I = \frac{bh^3}{12} = \frac{(7.5\text{in})(5.517\text{in})^3}{12} = 104.95\text{ in}^4$$

$$F_b = \frac{(4035.714\text{ in-lb})(2.758\text{in})}{104.95\text{in}^4} = 106.05\text{ psi}$$

$V = (1386\text{ lb/ft})(23.294\text{ ft}/2) = 16139.97\text{ lb}$

$Q = Ay = 41.377\text{in}^2 \cdot 2.758\text{in}^2 = 114.139\text{ in}^3$

$A = 7.5\text{in} \cdot 5.517\text{in} = 41.377\text{in}^2$

$y = 5.517\text{in}/2 = 2.758$

$I = 104.95\text{ in}^4$

$b = 7.5\text{in}$

$$F_v = \frac{(16139.97\text{lb})(114.139\text{in}^3)}{(104.95\text{in}^4)(7.5\text{in})} = 2340.41\text{ psi}$$

ET

$$M = \frac{wL^2}{8} = \frac{(1386 \text{ lb/ft})(1\text{ft}/12\text{in})(140 \text{ in})^2}{8} = 2021.25 \text{ in-ft}$$

$C = 2.758 \text{ in}$

$I = 104.95 \text{ in}^4$

$$F_b = \frac{(2021.25 \text{ in-ft})(2.758 \text{ in})}{104.95 \text{ in}^4} = 53.116 \text{ psi}$$

$V = (1386 \text{ lb/ft})(140 \text{ in} / 12 \text{ in/ft} \div 2) = 48510 \text{ lb}$

$Q = 114.139 \text{ in}^3$

$I = 104.95 \text{ in}^4$

$b = 7.5 \text{ in}$

$$F_v = \frac{(48510 \text{ lb})(114.139 \text{ in}^3)}{(104.95 \text{ in}^4)(7.5 \text{ in})} = 7034.31 \text{ psi}$$

GG

$\circlearrowleft$ $M = -8196.804 \text{ lb} \cdot (72.25 \text{ in}/2) + 10348.015 \text{ lb} \, (72.25 \text{ in}/2) = 77712.497 \text{ in-lb}$

$C = 7.5 \text{ in}/2 = 3.75 \text{ in}$

$$I = \frac{7.5 \text{ in}^4}{12} = 263.67 \text{ in}^4$$

$$F_b = \frac{77712.497 \text{ in-lb} \, (3.75 \text{ in})}{263.67 \text{ in}^4} = 1105.25 \text{ psi}$$

$V = 10348.015 \text{ lb}$

$Q = 56.25 \text{ in}^2 \cdot 3.75 \text{ in} = 210.937 \text{ in}^3$

$A = 7.5^2 \text{ in}^2 = 56.25 \text{ in}^2$

$y = 7.5/2 \text{ in} = 3.75 \text{ in}$

$I = 263.67 \text{ in}^4$

$b = 7.5 \text{ in}$

$$F_v = \frac{(10348.015 \text{ lb})(210.937 \text{ in}^3)}{(263.67 \text{ in}^4)(7.5 \text{ in})}$$

$F_v = 1103.79 \text{ psi}$

Pg 5

GL

$\circlearrowleft M = -10348.015\text{lb}(76.383/2\text{in}) + 4530.834\text{lb}(76.383/2\text{in})$

$= -222166.86 \text{ in-lb}$

$C = 3.75\text{in}$

$I = 263.67\text{in}^4$

$$F_b = \frac{(-222166.86\text{in-lb})(3.75\text{in})}{263.67\text{in}^4} = -3159.72 \text{ psi}$$

$V = 10348.015\text{lb}$

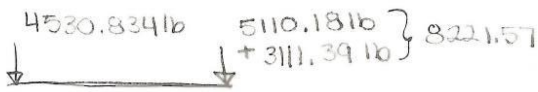
$Q = 210.937\text{in}^3$

$I = 263.67\text{in}^4$

$b = 7.5\text{in}$

$$F_v = \frac{(10348.015\text{lb})(210.937\text{in}^3)}{(263.67\text{in}^4)(7.5\text{in})} = 1103.79 \text{ psi}$$

LO



$\circlearrowleft M = -4530.83\text{lb}(27.03/2\text{ in}) + 8221.57\text{lb}(27.03/2\text{in})$

$= 49880.35\text{in-lb}$

$C = 3.75\text{in}$

$I = 263.67\text{in}^4$

$$F_b = \frac{(49880.35\text{in-lb})(3.75\text{in})}{263.67\text{in}^4} = 709.41 \text{ psi}$$

$V = 8221.57\text{lb}$

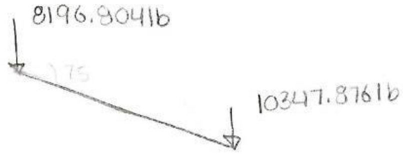
$Q = 210.937\text{in}^3$

$I = 263.67\text{in}^4$

$b = 7.5\text{in}$

$$F_v = \frac{(8221.57\text{lb})(210.937\text{in}^3)}{(263.67\text{in}^4)(7.5\text{in})} = 876.97 \text{psi}$$

BH



$$\circlearrowleft M = (-8196.804\text{lb} \cdot 72.34/2\text{in}) + (10347.876\text{lb} \cdot 72.34/2\text{in})$$
$$= 77808.028\text{in-lb}$$

$C = 3.75\text{in}$

$I = 263.67\text{in}^4$

$$F_b = \frac{(77808.028\text{in-lb})(3.75\text{in})}{263.67\text{in}^4} = 1106.61\text{psi}$$

$V = 10347.87\text{lb}$

$Q = 210.937\text{in}^3$

$I = 263.67\text{in}^4$

$b = 7.5\text{in}$

$$F_v = \frac{(10347.87\text{lb})(210.937\text{in}^3)}{(263.67\text{in}^4)(7.5\text{in})} = 1103.77\text{psi}$$

HK & GL

EA

$$F = \frac{\pi^2 EI}{(KL)^2} = \frac{\pi^2(620000\text{psi})(263.67\text{in}^4)}{(1 \cdot 48.833\text{in})^2} = 676589.63\text{lb} \leq 8196.804\text{lb}$$

FI

$$F = \frac{\pi^2(620000\text{psi})(263.67\text{in}^4)}{(1 \cdot 75.86\text{in})^2} = 280366.73\text{lb} \leq 10348.015\text{lb}$$

MJ

$$F = \frac{\pi^2(620000\text{psi})(263.67\text{in}^4)}{(1 \cdot 82.872\text{in})^2} = 234928.93\text{lb} \leq 4530.834\text{lb}$$

Appendix R2: Hand Calculated Spot Checks

Outer Support Stuctures		
w(d,decking)	37.10	lb/ft
w(d,railing)	20.56	lb/ft
w(d,total)	**57.657**	**lb/ft**
Member Density	35.6	pcf
Tributary Length	42.875	in
Decking Depth	3.5	in
w(l)	**357.292**	**lb/ft**
LL	100	psf
Trib Length	42.875	in
w(s)	**196.510**	**lb/ft**
SL	55	psf
Trib Length	42.875	in
Load Combo	#2: 1.2D + 1.6L + 0.5S	
w(total)	**739.111**	**lb/ft**

Center Support Stucture		
w(d,decking)	57.32	lb/ft
w(d,total)	**57.325**	**lb/ft**
Member Density	35.6	pcf
Tributary Length	66.25	in
Decking Depth	3.5	in
w(l)	**552.0833**	**lb/ft**
LL	100	psf
Trib Length	66.25	in
w(s)	**303.6458**	**lb/ft**
SL	55	psf
Trib Length	66.25	in
Load Combo	#2: 1.2D + 1.6L + 0.5S	
w(total)	**1103.946**	**lb/ft**

Column Loading

#1: Internal		
Trib Length	51	in
w(d)	46.0	lb/in
w(l)	25.3	lb/in
w(s)	4.8	lb/in
w(T)	92.0	lb/in
wt. support	43.3	lb
force	4764.1	lb
wt. member	29.0	lb
d	5.5	in
b	5.5	in
l	46.5	in
Fc	**157.5**	**psi**

#2: Internal		
Trib Length	85.5	in
w(d)	46.0	lb/in
w(l)	25.3	lb/in
w(s)	4.8	lb/in
w(T)	92.0	lb/in
wt. Support	72.7	lb
force	7984.6	lb
wt. member	46.3	lb
d	5.5	in
b	5.5	in
l	74.3	
Fc	**264.0**	**psi**

#3: Internal		
Trib Length	120	in
w(d)	46.0	lb/in
w(l)	25.3	lb/in
w(s)	4.8	lb/in
w(T)	92.0	lb/in
wt. support	102.0	lb
force	11192.1	lb
wt. member	50.7	lb
d	5.5	in
b	5.5	in
l	81.3	
Fc	**370.0**	**psi**

#1: External		
Trib Length	51	in
w(d)	29.8	lb/in
w(l)	4.8	lb/in
w(s)	16.4	lb/in
w(T)	61.6	lb/in
wt. support	43.3	lb
force	3213.5	lb
wt. member	29.0	lb
d	5.5	in
b	5.5	in
l	46.5	in
Fc	106.2	psi

#2: External		
Trib Length	85.5	in
w(d)	29.8	lb/in
w(l)	4.8	lb/in
w(s)	16.4	lb/in
w(T)	61.6	lb/in
wt. Support	72.7	lb
Force	5385.1	lb
wt. member	46.3	lb
d	5.5	in
b	5.5	in
l	74.3	
Fc	178.0	psi

#3: External		
Trib Length	120	in
w(d)	29.8	lb/in
w(l)	4.8	lb/in
w(s)	16.4	lb/in
w(T)	61.6	lb/in
wt. support	102.0	lb
force	7543.8	lb
wt. member	50.7	lb
d	5.5	in
b	5.5	in
l	81.3	
Fc	249.4	psi

Decking Support Members

Decking Support				Bending Moment				Deflection	(.5LL)		
	Internal			w(max)	92.0	lb/in		w(max)	23.0	lb/in	
w(l)	552.1	lb/ft		L	120	in		L	120.0	in	
w(s)	303.6	lb/ft		M(max)	165592	lb-in		E	1425600	psi	
w(d)	57.3	lb/ft						I	392.96	in^4	Allow
w(T)	1103.9	lb/ft						**Δ(max)**	**0.11**	**in**	0.333
	92.00	lb/in		Smin	66.24	in^3		Deflection	(.5LL+DL)		
Fb'=	~2500			b	5.5			w(max)	27.8	lb/in	
				d	9.5			L	120.0	in	
				S	82.73			E	1425600	psi	
				Fb	**2002**	**psi**		I	392.96	in^4	Allow
								Δ(max)	**0.13**	**in**	0.5
Decking Support				Bending Moment				Deflection	(.5LL)		
	External			w(max)	50.0	lb/in		w(max)	14.9	lb/in	
w(l)	357.3	lb/ft		L	120	in		L	120.00	in	
w(s)	57.7	lb/ft		M(max)	90074	lb-in		E	1425600	psi	
w(d)	196.5	lb/ft						I	392.96	in^4	Allow
w(T)	600.5	lb/ft						**Δ(max)**	**0.07**	**in**	0.333
	50.0	lb/in		Smin	16.38	in^3		Deflection	(.5LL+DL)		
Fb'=	~2500			b	5.5			w(max)	31.3	lb/in	
				d	9.5			L	120.0	in	
				S	82.73			E	1425600	psi	
				Fb	**1089**	**psi**		I	392.96	in^4	Allow
								Δ(max)	**0.15**	**in**	0.5

Decking Support				Shear		
	Internal			w(max)	92.0	lb/in
w(l)	552.1	lb/ft		L	120	in
w(s)	303.6	lb/ft		V(max)	5519.73	lb
w(d)	57.3	lb/ft				
w(T)	1103.9	lb/ft		b	5.5	
	92.00	lb/in		d	9.5	
				Fv	**158.46**	**psi**
Decking Support				Shear		
	External			w(max)	61.6	lb/in
w(l)	357.3	lb/ft		L	120	in
w(s)	57.7	lb/ft		V(max)	3695.55	lb-in
w(d)	196.5	lb/ft				
w(T)	739.1	lb/ft		b	5.5	
	61.59	lb/in		d	9.5	
				Fv	**106.09**	**psi**

Decking Members

Decking Members				Bending Moment			Deflection	(.5LL)		
				w(max)	15.9	lb/in	w(max)	4.2	lb/in	
w(l)	100.0	lb/ft		L	66.25	in	L	66.25	in	
w(s)	50.0	lb/ft		M(max)	8719	lb-in	E	1425600	psi	
w(d)	4.8	lb/ft					I	19.65	in^4	Allow
w(T)	190.7	lb/ft					**Δ(max)**	**0.04**	**in**	0.184
	15.9	lb/in		Smin	1.59	in^3	Deflection	(.5LL+DL)		
Fb'=	~5500			b	5.5		w(max)	4.6	lb/in	
				d	3.5		L	66.25	in	
				S	11.23		E	1425600	psi	
				Fb	**776**	**psi**	I	19.65	in^4	Allow
							Δ(max)	**0.04**	**in**	0.276

Decking Members				Shear		
				w(max)	15.9	lb/in
w(l)	100.0	lb/ft		L	66.75	in
w(s)	50.0	lb/ft		V(max)	132.60	lb-in
w(d)	4.8	lb/ft				
w(T)	190.7	lb/ft		b	5.5	
	15.9	lb/in		d	3.5	
				Fv	**6.89**	**psi**

Appendix S: Railing Member Calculations

Appendix S1: NDS Design Values

For supporting calculated for all work in this appendix, please see attached file: NDS Design Values – Railings.xlsx

Table 46: Modified design strength of each wooden member of the railings.

LRFD						Mid		Upper		
	End Post	Flat Post	Angle Post	Flat X	Angle X	Flat Hold	Angle Hold	Flat Hold	Angle Hold	
$F'_{b,z}$	1467	6375	4146	3866	3837	3277	5065	3278	5065	psi
$F'_{b,y}$	1467	6375	4146	3489	3515	2834	4375	2794	4209	psi
F'_{t}	950	3041	1996	1814	1814	3024	2419	3024	2419	psi
F'_{v}	285	302	302	302	302	302	302	302	302	psi
$F'_{c\perp}$	703	1059	1059	1059	1059	1059	1059	1059	1059	psi
F'_{c}	883	3095	2998	1650	2974	1894	2638	775	613	psi
E'	1140000	1710000	1615000	1615000	1615000	1615000	1710000	1615000	1710000	psi
E'_{min}	627000	940500	883500	883500	883500	883500	940500	883500	940500	psi
F'_{rt}										psi

Table 47: Modification factors used to adjust design strength. Lines highlighted in red were not used.

Adjustment factors		End Post	Flat Post	Angle Post	Flat X	Angle X	Mid Flat Hold	Mid Angle Hold	Upper Flat Hold	Upper Angle Hold	*note*
Load duration	C_D	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	1.15	
Wet service	C_M	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
Temperature	C_t	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
Beam stability	$C_{L,z}$	0.999101	0.99576	0.9975204	0.999446	0.99205	0.999571	0.99949154	0.999697	0.99948176	
	$C_{L,y}$	0.999101	0.99576	0.9975204	0.99205	0.999446	0.99402	0.99277294	0.979921	0.95527357	
Size	C_F	1.0	1.3	1.3	1.1	1.1	1.0	1.0	1.0	1.0	F_b
		1.0	1.3	1.3	1.0	1.0	1.0	1.0	1.0	1.0	F_t
		1.0	1.1	1.1	1.0	1.0	1.0	1.0	1.0	1.0	F_c
Flat use	$C_{fu,z}$	1	1	1	1.1	1.1	*1.15*	*1.15*	*1.15*	*1.15*	
	$C_{fu,y}$	1	1	1	1	1	*1*	*1*	*1*	*1*	
Incising factor	C_i	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	F_b, F_t, F_v, F_c
		1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	$F_{c\perp}$
		0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	E, E_{min}
Repetitive	C_r	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
Column stability	C_P	0.973771	0.775355	0.8525844	0.516042	0.930182	0.626454	0.72696943	0.256346	0.16905583	F_c, $F_{c\perp}$
Buckling stiffness	C_T	*1.0*	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	E_{min}
Bearing area	C_b	1	1	1	1	1	1	1	1	1	$F_{c\perp}$
Volume	C_V	1.00654	1.10456	1.1198193	1.256896	1.256896	1.189182	1.21358633	1.032223	0.94765567	F_b
Curvature	C_c	1	1	1	1	1	1	1	1	1	
Format conversion and Resistance	ϕK_f	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	2.16	F_b, F_t, F_v, F_c, F_{rt}
		1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	1.875	$F_{c\perp}$
		1	1	1	1	1	1	1	1	1	E
		1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	E_{min}
Time effect	λ	1	1	1	1	1	1	1	1	1	

Appendix S2: Comparison of Design Values to Analysis

For the full comparison of each individual member and stress in the bridge, please see attached file:

NDS Max stresses and interactions railings.xlsx

Interaction equations for members in bending and compression. Only for members in which the cells are white are these equations applied. For each three lines of X_{bzt} or X_{byt}, the first line represent the span of members closest to the wind source, the second to the middle span, and the third to the span farthest from the wind source.

Interaction Equations

Bending and Axial Compression

$$X_{bc} = \left[\frac{f_c}{F_c'}\right]^2 + \frac{f_{bz}}{F_{bz}'[1-(f_c/F_{cEz})]} + \frac{f_{by}}{F_{by}'[1-(f_c/F_{cEy})-(f_{bz}/F_{bE})^2]} \le 1.0$$

							Mid		Upper	
		End Post	Flat Post	Angle Post	Flat X	Angle X	Flat Hold	Angle Hold	Flat Hold	Angle Hold
LRFD	**F_{cEz}**	8838	5648	6980	14234	2614	18809	30059	15214	11722

$$f_c < F_{cEz} = \frac{.822E'_{min,z}}{(l_e/d)_z^2}$$

							Mid		Upper	
		End Post	Flat Post	Angle Post	Flat X	Angle X	Flat Hold	Angle Hold	Flat Hold	Angle Hold
LRFD	**F_{cEy}**	8838	5648	6980	2614	14234	3455	5521	1132	872

$$f_c < F_{cEy} = \frac{.822E'_{min,y}}{(l_e/d)_y^2}$$

							Mid		Upper	
		End Post	Flat Post	Angle Post	Flat X	Angle X	Flat Hold	Angle Hold	Flat Hold	Angle Hold
	1	1.42847E-05	2.35533E-05	2.50999E-05	1.73E-05	5.31765E-06	3.20971E-06	1.6552E-06	1.35846E-05	2.16909E-05
Quotients	2	0.242325076	0.168023816	0.256739705	0.224976	0.229236978	0.001686018	0.001090432	0.110706851	0.071691505
	3	0.013854066	0.011064967	0.017025785	4.02E-05	5.47773E-05	0.005651408	0.003660179	0.103739924	0.068909632
LRFD	**X_{bc}**	0.26	0.18	0.27	0.23	0.23	0.01	0.00	0.21	0.14

Appendix T: Railing Information for Manufacturer

Upward diagonal of 268 inches attached to a horizontal segment of 120.5 inches followed by a downward diagonal of 268 inches.

Upward and downward diagonal shall be at 4.7 degrees.

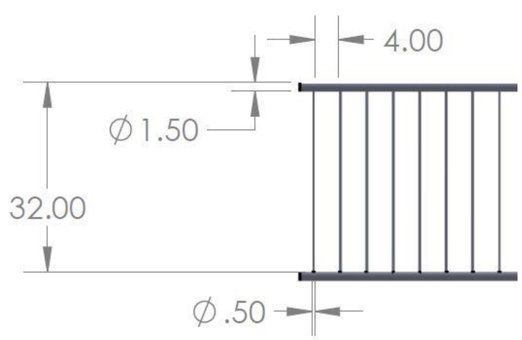
Railing Scheme

- Bottom horizontal 1.5 inch diameter pipe
- Mid-height handrail of 1.5 inch diameter at 14 inches off center from the bottom horizontal
    - The mid-height handrail shall not be in line with the top and bottom horizontal rails, it should instead be slightly offset such that when the vertical rails are in place, the handrail remains continuous, and is not segmented by these rails
- Top handrail of 1.5 inch diameter at 32 inches off center from the bottom horizontal

Vertical Rails

- Shall have a diameter of 0.5 inches
- Shall be set at 4.5 inches off center
    - The vertical rails shall begin at the bottom of the diagonals
    - The exception to this offset occurs where the diagonals meet the flat section, at which points the verticals will only be at 4 inches off center.
- There will be 146 vertical rails

A sample of the section is included below



Appendix U: Myra Hiatt Kraft Bridge City Proposal

Michael V. O'Brien
City Manager

CITY OF WORCESTER

THE MYRA HIATT KRAFT FOOTBRIDGE WITHIN HISTORIC ELM PARK

The Naming & Replication of One of the City's Most Iconic Park Features in Honor of Our
Myra Hiatt Kraft
An Incredible Woman Who Built Bridges of Understanding, Compassion & Kindness Throughout Her Lifetime

Prepared By: Michael V. O'Brien, City Manager

The Naming of the Historic Wooden Footbridge in honor of Myra Hiatt Kraft

For her dedication to her family, extended family and our community – a trait learned from her father, the great Jacob Hiatt, an amazing leader, benefactor and great friend to our City, our community will name the iconic Wooden Footbridge at our Elm Park in honor of Myra Hiatt Kraft, an incredible woman who built bridges of understanding, compassion & kindness throughout her lifetime.

OFFICE OF THE CITY MANAGER, CITY HALL, WORCESTER, MA 01608
TELEPHONE (508) 799-1175 | FAX (508) 799-1208
EMAIL: citymanager@worcesterma.gov

- Leadership from the Top – The Lieutenant Governor, the Mayor, the City Council and the City Manager will advocate and will support the naming of this bridge through all the necessary public processes to secure, for perpetuity, the official designation of the bridge in honor of Myra Hiatt Kraft.

Partners:

- The Greater Worcester Land Trust searched archives at the State, at the Historic Museum and in the City to try and locate the original construction plans for the Wooden Footbridge, but unfortunately they are not available.
- Worcester Polytechnic Institute with management and oversight will work with the talents of our Worcester Technical High School Staff, Faculty and Students to build new, to all historic standards, this Bridge, with state of the art materials and through historic woodworking techniques, sheer talent, skill, energy and enthusiasm. The amazing 21st century equipment and resources of our Technical School, in collaboration with WPI and their high talented students will be used throughout this process.
- The WPI staff and students, with full City-team tactical support, will steward the historic replication through all necessary approvals of the State Historic Commission, the local Historic Commission, the Conservation Commission, Code Reviews and other necessary steps as a lesson in civics and government as well.
- Every step will be facilitated and expedited with the full attention and guidance of the Principal of the Technical High School, Department Head and Professor of WPI Civil, Environmental, and Architectural Engineering Program, and the City Manager.
- Estimates of the City's costs to undertake this effort through conventional City public bidding exceed $400,000. We believe we can complete this for $150,000 by and through the application of the talents of our Technical High and WPI Students. Home Depot has expressed interest to assist us with some the required materials. The City will participate financially for the specialized equipment to work on, upon and under and to otherwise set this new Bridge within the waterway. My work now focuses on a private partner for up to $50,000 to address all other material and other specialized needs.

Timing:

- The steps to name the Historic Wooden Footbridge within Elm Park in honor of Myra Hiatt Kraft begin immediately. The formal proposal will be submitted by the City Manager to the City's Parks and Recreation Commission for their May 2012 Meeting. A Resolution of the City Council in support of this and a letter of recommendation from the Lieutenant Governor and the City Manager will accompany this proposal. It is expected that this would be formally, unanimously approved in the May 2012 meeting of the Commission.
- The efforts of the Technical High and WPI Students to remove, replicate and replace the Bridge will begin upon the finalization of all funding sources. The vast volume of replication and replacement efforts will begin in the Fall of 2012 with the start of the new School Year.
- Formal Naming Ceremony with appropriate plaques will be set and scheduled for the Spring of 2013.

Appendix V: Interview Summaries

Interview Summary For Rob Antonelli of the Worcester Parks and Recreation Department

Erik Snodgrass and Matthew Valcourt interviewed Rob Antonelli, Assistant Commissioner of the Worcester Parks and Recreation Department, on September 19, 2013 at 12 P.M. Mr. Antonelli was interviewed in person at Elm Park. Bill Richards, a project manager for the construction currently taking place at Elm Park, assisted Mr. Antonelli in answering some of the questions. The following is a transcription of the conversation that took place.

We first asked Mr. Antonelli about future goals regarding the usage of the pond that the bridge overlies. It is expected that in the future people will be able to traverse the pond in kayaks during the summer, as well as ice skate during the winter. For this reason, there needs to be a certain amount of clearance between the height of the high point of the water and the height of the bridge. Mr. Antonelli stated there is probably some flexibility in dropping the bridge slightly as a long as the appropriate clearance level is respected. We also asked about the fluctuation in the water level of the pond. Mr. Antonelli said the water level has not been at its high point since this past spring. Once they began working on the park, the started draining the pond. The water in the pond comes from drainage from the street, and runoff from the park itself. They used to pump drinking water in the pond, which would keep the pond level up, however it promoted algae growth in the pond due to the phosphorous levels. They are currently working with a private company to figure out a sustainable source of water for the pond.

We then asked Rob about the specific sides of the pond in relation the bridge. One side is a big open circle without many obstacles while the other is swampier and would be much more difficult to traverse. He does expect that this swampier side will be open to the public as well. There is a boathouse on this side of the pond that they built a long time ago, and the ultimate goal is to restore it and sail from that location throughout the rest of the pond. Rob stated that ultimately they may want to look into getting different types of boats into the pond, so this needs to be taken into consideration during the design phase of the bridge. He also stated that while changing dimensions of the bridge such as the height is completely acceptable, changing the dynamics of the bridge such as the arch, most likely could not happen. The architectural design itself and the historical integrity need to remain intact.

Mr. Antonelli went on to explain that a big question facing the renovation of the bridge is whether or not we can make it handicapped accessible. He talked about options such as manipulating the dimensions of the bridge to achieve an acceptable slope, or introducing a switchback on each side of the bridge that could bring you up to the ultimate height of the bridge and bring you back down. These options will most likely come down to what amount of visual impact the city is willing to sustain as compared to the current conditions. Mr. Antonelli went further in depth explaining the “fill” option, which would be changing the topography of the land in order to meet ADA requirements. He explained that they could do things such as dress up the pathways leading up the bridge with rock sidewalls or some other type of aesthetically appealing component to help reduce any visual impact the land manipulation may have. Mr. Antonelli also stated that is possible that the bridge will not be able to meet ADA requirements due to the historical impact changing the design would have, and in that case other options would need to be explored.

We then asked Rob about the cement footings that are currently in place supporting the bridge. He believes that if the footings are structurally sound and meet all structural requirements, then the most cost-effective option would be to keep these footings in place and use them for the new bridge design. He said once the bridge is removed they could go in and power wash it, possibly paint it, to make them more aesthetically pleasing. They also have structural engineers on call and once the bridge is removed, they will take a look at these footings to ensure that they are structurally sound.

We asked Mr. Antonelli about any variances the bridge currently has regarding ADA requirements or complaints the parks department has received regarding the bridge. While the city has received many complaints regarding the bridge, the bridge itself does not have any variances and was not operating under any up until the point that it was shut down. The city has not asked for any waivers for the bridge and they have not received anything stating that they need to make the bridge accessible. The bridge was shut down in May of 2013 due to its uncertain structural integrity. Mr. Antonelli stated that in order to obtain a historical variance you either need to demonstrate that it costs too much to make it accessible for all persons or that the visual and historical impacts would be too drastic to achieve ADA compliance.

We then spoke more about ADA requirements regarding slope. Mr. Richards stated the maximum slope without the necessity of a handrail is 5%. You can go up to 8% as long as there is a handrail and a level landing every 30 feet. We asked Mr. Antonelli about the current slope of the bridge and he believes it is currently at 23%, although this number was not confirmed in person.

We asked Rob about the pathways leading up the bridge and what the plans were for those. They are planning on using asphalt with an aggregate called chip seal pushed into the asphalt, which also meets ADA requirements.

We were interested in whether or not Rob had any existing plans or specifications regarding the bridge. He does not have any plans for the bridge at this point but does have a site plan with the bridge and a full survey that he said he could provide to us. Bill Richards offered to set a benchmark at some point near the bridge. He also stated there are benchmarks around the park we could use as well to help us obtain elevations on the bridge. We also told him about our plans to fabricate plans for the bridge's "as is" condition and Rob offered to write a document giving our team access to the bridge to obtain any measurements we might need to finish the plans.

We then tried to gauge Rob's knowledge regarding future plans for the bridge itself and at this point he knows just about everything that our group has been told. For them, it's about us getting our design plans complete so they can rebuild it. The goal right now is to have the bridge finished by next year, but if we cannot make it handicapped accessible and Rob has to go to the ADA, then obviously the timetable will change drastically. We also need to work with the technical high school as well and respect their academic schedule. There are many different pieces that need to fit together the correct way to make this happen by next summer. Right now, the most important aspect is to get our design done. Once that happens, everything else will begin to fall into place according to Rob. We told Rob we would like to have existing plans complete for the bridge within the next two weeks and present to the historical commission by the beginning of November. Rob then adamantly stated that we should not waste our time exploring a design option that changes the visual components too much. He believes we are better off trying to find ways to make it more accessible then we would be to change the design of the bridge completely. The historical integrity is probably the most important aspect of the bridge.

We asked Rob about other things we could be doing to expedite the renovation process. Rob then stated that the goal is to move the bridge as soon as possible. For us, we need to stay on task in getting together paperwork for AAB, where Rob will most likely be defending our design option. In that paperwork, we need to also explore cost and why we are choosing one design alternative instead of another. Once the city approves a design option given by us, the fight will mainly lie between Rob and the City Manager with AAB about how they make that work in parks setting. At this point, we should be continually moving forward on the design portion of the bridge.

We also asked Rob about the option of exploring different, more durable and cost effective materials for the bridge. The city is open to using different materials, especially if they prove to be more durable and cost effective. Currently, the bridge is made of different types of wood, one type being spruce. The bracing will probably be similar to what's there now, however it is the wear points that need to be investigated regarding alternative materials. Ultimately, the city does not want to have to come to the park every year in order restore the bridge or repaint it. He talked about the possibility of putting caps around the bases near the footings so ponding doesn't occur on the footings themselves, as just one example of making the bridge more durable.

We asked Rob about lighting for the bridge. He recommended down lighting, up lighting, even spot lighting as all options to consider. We would need to consider how we hide those electrical lines, and how we are going to make them tamper proof. They do not have cameras in the park yet but they will eventually. The locations of the cameras have already been pre-determined. The light pole currently next to the bridge is eventually going to be removed as well.

Mr. Antonelli stated the best way to get a hold of him was through his email and he would be sending us the clearance letters and site plans as soon as possible.

Meeting Summary with David LaPointe and Regan Harrold of Beal's and Thomas, Inc.

Erik Snodgrass and Matthew Valcourt had a meeting with David LaPointe and Regan Harrold of Beal's and Thomas, on February 18, 2014. They met at the Southborough office of Beal's and Thomas Inc. located at 144 Turnpike Road Southborough, MA. The purpose of this meeting was to discuss the Landscape CAD file that Erik and Matt had submitted to David so that proper engineering plans could then be created from it.

Upon their arrival Erik and Matt were met by David who then introduced them to Regan who is the lead Landscape Architect working on the engineering plans for Elm Park. After introductions we moved into a conference room where Erik and Matt's CAD file was displayed on a screen. The first part of the discussion was a quick recap of the last meeting that David, Erik and Matt were at with the Worcester Parks Department. They discussed the pond edge wall location in south mere, the lighting for the mere, and railings that might be needed because of vertical drops.

Next they discussed the general layout of the CAD that was provided to make sure both parties knew what was being modeled. Questions arose between the two groups as to the amount of space provided between contour lines in order to achieve the proper slope. It was determined that the CAD provided would be acceptable but minor alterations should be made.

Following that, they discussed the proper codes that would have to be followed in order to make the site design fall into compliance. They also discussed what needed to be done to the CAD file in order to come up with a final site plan. Topics such as the removal of trees, relocation and addition of lighting, and the moving of walkways were discussed. Erik and Matt expressed that they need to receive final blueprints for the site based on any changes that may be made, along with any cut and fill changes so that a proper cost estimate for materials introduced to the site could be performed.

Once discussion on Elm Park ended they were able to talk about the roles Beal's and Thomas plays in projects and how the firm runs overall. It was a great experience from a professional development aspect to learn about the inner workings of a landscape engineering company such as them. They were shown where all records for projects were cataloged, which included all correspondence and plans for closed projects. Next they were shown the 'war room' which is where there is a large space to store documents that pertain to current projects, printers for making blue prints, and other office equipment to make plans. After that David explained how the cubical work areas were divided. He said that each work area had specific projects they were working on and within the area were employees who had knowledge in the areas related to the project, such as a landscape architect and different engineers. Upon finishing the tour, David and Regan provided contact information for future correspondence purposes.

Appendix W: Historical Commission Presentation outcomes

Upon the completion of background and historical research as well as the development of an introductory design, the removal and reconstruction of the bridge was presented to the Worcester Historical Commission to ensure awareness of the project, as well as gain insight into any historical design criteria. This presentation occurred on October 24, 2013 with four group members in attendance. From this presentation and the following comments it became clear that the historical commission was very interested in the historical preservation of the old bridge as well as the accessibility to all park patrons.

An appropriate presentation was prepared for the commission December 5, 2013 to inform them of the progress of the project. This presentation included designs that could be implemented in place of the current structure. This design was compliant with the American Disabilities Act as well as the Architectural Access Board. The historical commission responses were positive; there only concern was in the design of a stable structure. However, the professional engineer will be ensuring that the final design is stable.

A final presentation we presented to the Worcester Historical Commission on Thursday April 10, 2014. In this presentation the final architectural design was presented to the commission for comments. After fielding questions from committee members and audience members, the group received positive feedback from the committee.