Structural Improvement Design for Northern Strand Community Trail Bridge over Saugus River

A Major Qualifying Project Proposal Submitted to the Faculty of
Worcester Polytechnic Institute
In Partial Fulfillment of the Requirements for the Bachelor of Science Degree
By

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Abstract

The project's goal was to design an improvement solution to the 115-foot pedestrian bridge over the Saugus River along the Northern Strand Community Trail for Stantec Consulting Services, Inc. Three improvement alternatives were considered -- aesthetic improvements, rehabilitation, and replacement. The final improvement design consisted of a 130-foot prefabricated replacement truss structure and new concrete abutments that met AASHTO LRFD requirements. Final deliverables included steel member and abutment sizes, a demolition and construction plan, cost estimate, and a project schedule.

Executive Summary

Project Scope and Objectives

Stantec Engineering Consultants, Inc is currently redesigning the Northern Strand Community Trail (NSCT) corridor for optimal pedestrian usage. The NSCT passes through several towns in Massachusetts, including Everett, Revere, Malden, Saugus, and Lynn. This project includes the design of three pedestrian bridges. The WPI students worked in conjunction with the Stantec team on redesigning the existing bridge, which passes over the Saugus River.

This project sought to develop an improvement alternative for the trail's 115-foot pedestrian bridge that spans the Saugus River. In 2018, the Wood Advisory Services provided Stantec with an evaluation of the primary timber members that support the bridge. This report revealed the significant variability in the remaining cross section of the bridge piles. Resistograph measurements yielded a remaining cross-section range of 17% to 92%, with four critical piles under 25% remaining cross section. The report also highlighted deteriorated pile caps, insufficient creosote retention levels, and failed cross-bracing. Based on this timber investigation, the team determined the bridge to be structurally deficient and assisted Stantec in developing a safe and economical improvement design for the bridge.

The structural improvement project focused on designing a new structure that would involve minimal environmental disturbances to the surrounding land. The project site's location next to a protected wetland placed environmental restrictions on feasible design options and construction techniques. Additionally, construction within a dormant railway right-of-way and adjacent to a MassDEP hazardous waste site had the potential to expose the wetlands and underlying river to harmful pollution. As a result, the project pursued safe and economical design options that mitigated environmental impacts to the wetland and Saugus River.

Methods Overview

In order to complete the project goals and objectives, the team first identified bridge users and associated loading requirements to meet AASHTO LRFD requirements. After the existing site conditions were assessed, improvements alternatives were researched and determined. Improvement alternatives consisted of aesthetic improvements, rehabilitation, and replacement. The alternatives were then evaluated and analyzed through a series of decision matrices to determine the most suitable option and respective design option. Once each alternative and respective design option was researched the team delivered final designs and recommendations that best addressed capstone design criteria.

Recommendations

The team's final recommendations were to replace the existing bridge with a prefabricated single span truss bridge and new abutments. Using a prefabricated structure was the most efficient option when considering construction and cost. The ability to hoist the structure into place, reducing necessary construction activities on site which in turn reduces the project’s impact on the surroundings. In addition to the design, a demolition plan and construction plan were proposed to Stantec. These plans were based on a comprehensive site analysis and sought to ensure efficient demolition and construction phasing, while considering the environmental constraints posed by the site. Finally, cost estimates and schedules were created for the project, which captured the cost and time implications of two proposed demolition options -- 1) removal of bridge deck and piles, 2) removal of bridge deck and pile caps (no pile removal).

Acknowledgements

We would like to sincerely thank the following people for without their help completing this project would not have been possible. First, we would like to thank Worcester Polytechnic Institute for the opportunity to represent them in the professional field. We especially thank our advisors, Professor Albano and Professor LePage, for their valuable guidance. Additional thanks go towards Professor Salazar for his willingness to answer our project management questions and working in the labs during his lectures. We thank Stantec Consulting Services, Inc. for the resources and office space to complete this project. Lastly, we thank the following people from Stantec for making themselves invaluable throughout the duration of our project: Aleece D'Onofrio, Jen Ducey, Hillary King, Erica Lotz, David Melo, Dennis Reip, Joseph Rubino, Joseph Salvetti, Carly Soderholm, and Nelson Sosa.

Authorship

The project team worked together to complete the final MQP report. Everyone contributed equally to writing and editing the background, methods, findings, and results sections of the paper. Project member Sean Burke wrote the Design Capstone Statement and structural sections related to bridge rehabilitation. His work involved developing the structural hand-calculations for bridge members as well as the AutoCAD drawing sheets. Project member Marlies de Jong focused on the structural sections relating to bridge replacement and aesthetic improvements and the Professional Licensure Statement. Her work focused on creating the Excel spreadsheets and RISA-3D analysis for the final bridge design. Project member Ben Leveillee's focus was on the demolition plan, construction and staging plans, cost estimation and scheduling, as well as the design of abutment walls.

Capstone Design Statement

This Major Qualifying Project (MQP) required the project team to develop preliminary design alternatives and improvements for a decommissioned railroad bridge. The bridge is currently part of the Northern Strand Community Trail (NSCT) in Saugus, Massachusetts that serves pedestrians and bicyclists. In order to determine that the project met design requirements and stakeholders' needs, the team incorporated the following design constraints: health and safety, cost, environmental impacts, aesthetics, constructability, and ethics. By addressing these design constraints, the design project met the requirements for a Capstone Design Experience, as outlined by the Accreditation Board for Engineering and Technology (ABET).

Health and Safety

Safety was the primary design factor for this project. Any recommendations for improving the pedestrian bridge had to retain its structural integrity and serviceability requirements during all required load scenarios defined by the *AASHTO Guide Specification for the Design of Pedestrian Bridges*. The project schedule structure was developed in accordance with responsible construction phasing. Any additional recommendations maintained the safety of construction workers during the construction process as well as pedestrians and cyclists upon installation.

Cost

The final design for this improvement project considered material and labor costs based on the following sources: 1) the Massachusetts Department of Transportation statewide average bid prices; 2) Stantec past projects; 3) *RS Means Heavy Construction Data*. This project recommended a final design that was cost-effective without sacrificing structural functionality or material quality.

Environmental

The pedestrian bridge over the Saugus River is surrounded by wetlands. This project site posed significant environmental design criteria because not only are wetlands sensitive natural environments, but they are also protected by the Massachusetts Wetlands Protection Act. With the bridge in close proximity to protected land, environmental concerns such as hazardous material contamination and destruction of natural habitat are magnified. It was vital that preventative measures be taken to mitigate these risks. Because the scope of work was within wetlands or within a 100-foot buffer area around the wetlands, the final design and construction recommendations were made to minimize these impacts on the surrounding environment.

Aesthetics

In addition to being surrounded by wetlands to the West, the bridge is located in close proximity to residential properties to the North and East. Due to the high visibility of the bridge, this project considered the aesthetics of the bridge and any structural improvements to it. A redesign that is both aesthetically pleasing and quick to construct minimized the visual disruption to the natural landscape.

Constructability

It was imperative the final design have an ease of constructability where the bridge could be prefabricated off-site and easily installed on-site in order to meet time constraints. Typical members that are available to be prefabricated are easier to construct compared to custom member sections that would require additional labor for on-site fabrication. Additionally, there is limited site access which added an extra constraint when creating construction plans to install the bridge.

Ethics

The eight canons established by the American Society of Civil Engineer's (ASCE) Code of Ethics were followed while developing the final design and recommendations. Research considered the variety of perspectives surrounding this project to ensure that final design and recommendations provided a holistic solution to this problem. Design decisions held "safety paramount" and considered the surrounding communities' environmental and social diversity (ASCE, 2017).

Professional Licensure Statement

The National Council of Examiners for Engineering and Surveying (NCEES) is the organiz-ation that determines professional licensure for all engineers in the United States. By advancing licensure for engineers and holding all engineers accountable to the same standards, the NCEES can best ensure the wellbeing and safety of the public. Earning a professional license in the United States allows only qualified individuals to design, certify, and stamp engineering documents.

In order to obtain a professional engineering license, the following requirements in education, exams, and experience must be met. First, Professional Engineer (PE) candidates must have an Accreditation Board for Engineering and Technology (ABET) certified bachelor's degree. Next, candidates must pass the Fundamentals of Engineering (FE) exam to become an Engineer in Training (EIT). An EIT must then complete a minimum of four years of acceptable and verifiable experience in their related field under a PE. Then, an applicant must take and pass the Principles and Practice of Engineering Exam to prove their skills and knowledge. Each state may have additional requirements for applicants and each state individually approves PE certification.

Upon obtaining a PE license, engineers can further advance their career where they can own a firm, privately consult, and place bids for government contracts. However, PE's are not only held responsible for the safety of the designs they stamp, but also for the safety of the people who use their designs. Such accountability is not taken lightly and, as such, requires an extensive amount of time and dedication to obtain licensure.

It is important to note that a PE in civil engineering would be required to fully complete the following project. The bridge structure has the potential to negatively impact the user's safety if calculations were not carried out correctly. As a result, a PE would need to approve the design and calculations carried out by the project team before document submittal.

1.0 Introduction

The Northern Strand Community Trail (NSCT) is a 10.5-mile trail corridor located North of Boston, Massachusetts. As shown in Figure 1, this trail begins at the Mystic River in Everett and ends in Lynn, connecting the urban environments of Revere, Malden, and Everett to the beaches of the North Shore. The NSCT is largely separated from roadways and vehicles, and therefore is a safe walking and cycling environment for residents of the greater Boston area. Since the trail was established in 1993, a local advocacy group, Bike to the Sea, has been working to raise awareness and funding to improve the trail. The NSCT is part of a larger initiative, known as the East Coast Greenway, to build a 3,000-mile protected biking and walking route from Maine to Florida.

Figure 1: Map of the Northern Strand Community Trail (Bike to the Sea, Inc.)

The communities surrounding the trail are looking to protect the safety of the cyclists and pedestrians that use it. However, the nine-span, 115-foot, bridge over the Saugus River has been proven to be a hazard to trail goers. Preliminary timber pile sampling and testing results indicated that the bridge, as seen in Figure 2, is not structurally sound for continued pedestrian use (WAS, 2018).

Figure 2: Saugus River Pedestrian Bridge

The goal of the project was to assess the condition of the bridge and develop potential solutions that address the failing structure. The objectives seen in Figure 3 guided the team's process for the project.

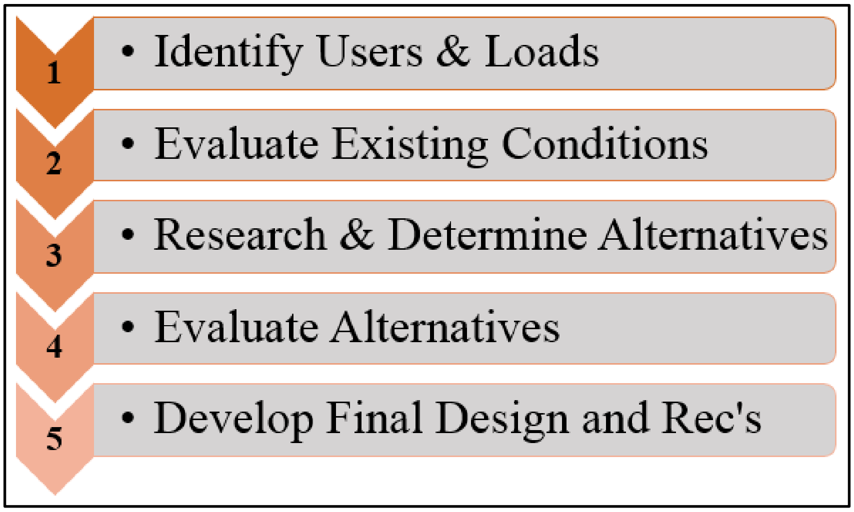

Figure 3: The Five Project Objectives

The first objective to achieve this goal was to identify bridge users and associated loading requirements. Observations from site visits and the findings from the Wood Advisory Services, Inc. (WAS) were combined to develop a complete understanding of the bridge's existing conditions for the second objective. As part of the third objective, the team developed a list of rehabilitation and replacement options with different bridge designs and fabrication methods that complied with governing codes and specifications. The fourth objective consisted of an evaluation of all the design and alternative options. The most effective design was the one that addressed the health and safety, cost, constructability, environmental, and ethical constraints defined for this project. Ultimately, the last objective focused on delivering a final recommendation for the bridge's design alternative. These recommendations included structural design and models, a project schedule, and a cost estimate.

2.0 Background

This section will give an overview the history of the Northern Strand Community Trail (NSCT) including the surrounding land and its classifications. Existing conditions of the pedestrian bridge over the Saugus River will be reviewed, followed by a discussion on options for design improvements.

2.1 Trail History

Before the NSCT became a well-known bike path, it was the Saugus Branch commuter railroad, serving residents of Saugus, Malden, Everett, Revere, and Lynn. The Eastern Railroad Company began construction on the line in 1850 and serviced its first customers three years later. Usage of the railroad steadily increased to its peak in the 1890s, when the Saugus Branch offered 36 trips into Boston per day. After the construction of the Boston Elevated Railway in the early 1900s, the demand for the Saugus Branch gradually declined until its last passenger trip in 1958. Once commuter service was discontinued, it was converted into a freight line until 1993, after which the railroad was abandoned (Revolvy, 2018).

Bike to the Sea's involvement began in 1993 when the freight line was discontinued. Bike to the Sea is a local social advocacy group that promotes the use and maintenance of the NSCT. This group organizes community activities, fundraisers, and general awareness of the trail. Nearly all of the original railroad material has been removed to make the path more suitable for pedestrian use. Sections in Everett and Malden have also been paved to improve conditions for bikers. Since the transformation from rail to trail, the local community has embraced the new recreational path as an alternative means of commuting (PB/Harris, 2002). The close residents are not the only ones to show interest in the NSCT. The trail has gained recognition by the state government, recently being awarded $1.5 million to fund redesigning the 10.5-mile path (Mass.gov, 2018). The 2.5-mile section through Saugus is one of the unpaved portions, with a beautiful ride along the Saugus River. This part of the trail has a bridge, which serviced freight trains during the later years of the railroad's life.

In 2002, the MBTA made an evaluation of the Saugus Branch, with consideration of recommissioning it for various forms of use. Options for alternative use include converting the railroad into a truck haul route for Boston Logan Airport or potentially an MBTA Urban Ring bus route. A third option would have been to run a rapid transit line for commuter service. These options have not been pursued due to restrictions created by the current surrounding environment. The right-of-way is now constricted by numerous crossings and abutting residential properties, making the prospect of recommissioning unlikely (PB/Harris, 2002).

2.2 Current Land, Land Use and Zoning

Saugus is currently divided into twelve designated zoning districts as mandated by the town's zoning by-laws which were updated in 1997 (Ortiz, 1997). The bridge is located on land zoned as open space and is surrounded by zoned single-family residential dwellings (R1) to the North and East direction, and heavy industrial zones (I2) in the West and South directions. After the group conducted a site visit, they found that the bridge is surrounded by residential units and undeveloped land with no signs to indicate any construction in the near future.

A third of Saugus' land is classified as tax exempt. Tax exempt land is typically publicly owned by a government entity. Parks and open spaces, such as the NSCT, are among the most common land uses to fall under this classification. Within tax exempt land use, there is a right-of-way easement which is granted over land being used for transportation services or public utilities (MAPC, 2018). About 14% of Saugus' total land area is classified as a right-of-way which is primarily comprised of roads, curbs, and sidewalks (MAPC, 2018). Another notable right-of-way is the NSCT. Since the bridge is a former railroad track turned pedestrian-bicycle path and also contains a large utility line, it meets the classifications for a right-of-way land use.

The Saugus River is approximately 13 miles long and eventually leads to the Atlantic Ocean, and drains a watershed with an area of 47 square miles (Saugus, n.d.). The team developed the map in Figure 4 based on Massachusetts Department of Environmental Protection and Geographical Information System (GIS) data layers, and it shows the salt marshes and tidal flats that surround the bridge. These soils are often composed of deep mud and peat, which is decomposing plant matter (US EPA, 2015). As a result, the water flowing under the bridge is brackish and the area is subjected to tidal flooding on a consistent basis. This combination of land classifications poses some unique environmental concerns for development on or around the bridge.

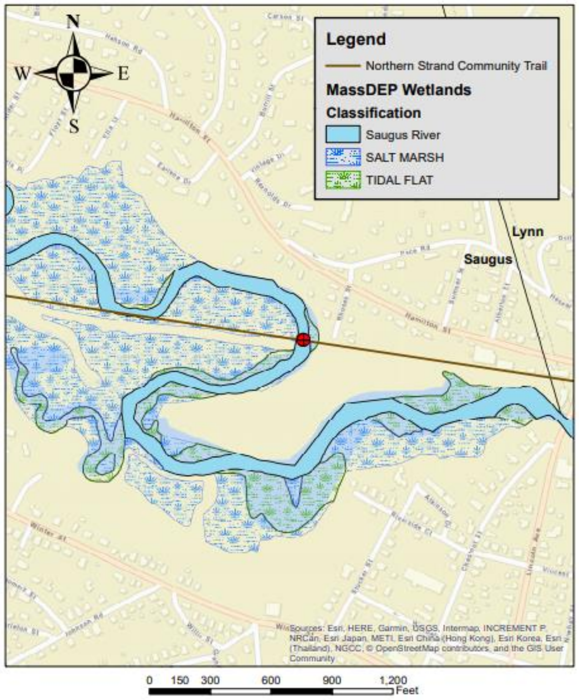

Figure 4: Map of Saugus River and Bridge

2.3 Existing Bridge Conditions

The NSCT pedestrian bridge that passes over the Saugus River is the focus of this project because it has been shown to be structurally compromised. An analysis of the bridge done by the Wood Advisory Services, Inc. (WAS) evaluated the integrity of the timber piles supporting the structure. The scope of this test included both visual and lab testing of 20 core samples taken from the timber. The visual inspection of the bridge revealed numerous locations where significant deterioration of the wood led the inspectors to believe the service life of the bridge is nearing its end. Compromised members included piles, pile caps, cross bracing, and the abutments. Additionally, resistographs were created for 70% of the existing timber piles to quantify the amount of deterioration and assess the remaining effective cross-sectional area. The results from this testing were extremely variable across the different readings, with an effective cross section ranging between 17% and 92%, as seen in Table 1.

Table 1: Timber Pile Integrity Data from Wood Advisory Services, Inc.

PILE NO. =>	1		2		3		4		5		6		7		8		9		10		11	
	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'
BENT NO.	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)
West Abutment																						
1	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*
2	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*
3	14	65	14	62	12	84	15	87	12	34	15	83	14	62	12	92	13	85	14	80	14	83
4	14	33	13	83	14	51	14	33	13	57	14	86	14	86	12	17	13	83	14	86	14	84
5	13	48	14	86	14	85	15	85	12	38	15	44	14	73	15	54	13	57	14	80	14	51
6	14	18	14	86	15	44	14	85	14	46	14	73	14	62	15	68	13	25	14	56	14	62
7	14	72	14	73	14	73	15	87	14	41	14	60	14	62	15	87	15	48	15	64	15	75
8	14	51	12	69	14	33	15	44	14	49	15	44	16	56	14	86	15	44	15	44	13	48
9	*	*	*	*	*	*	*	*	*	*	15	44	14	73	15	64	15	87	16	77	14	86
10	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	14	25	15	62	*	*
East Abutment																						

NOTE:		
	*	No data recorded
	ϕ	Diameter of pile @ breast height (in.)
	A'	Estimated effective remaining cross sectional area (% of calculated cross-section from diameter)
		00% to 25% Estimated residual cross section (%)
		26% to 50% Estimated residual cross section (%)
		51% to 75% Estimated residual cross section (%)
		76% to 100% Estimated residual cross section (%)

The final data collected was the creosote retention. This is a measurement of the creosote treatment that is applied to the timber to protect it from weathering and extend the service life. Recommended creosote retention levels range from 16 pcf to a minimum of 8 pcf (Webb, Webb, Zarembski, 2016). The measured value of the timber piles was 7.60 pcf. The combination of timber deterioration and lack of creosote limits the potential service life of the substructure. The presence of these conditions was a major consideration while examining improvements to the bridge.

2.4 Bridge Components

Prior to analyzing different design options, the fundamental functions of the existing bridge's members were researched. A strong understanding of a bridge's structural components is vital for an effective design. There are two main categories to a bridge: the substructure and superstructure. The substructure is comprised of the foundation, wing walls, abutments, and piers. Foundations evenly transmit the loads generated by everything above it into the strata beneath it, while wing walls are an extension of the abutments to retain any strata present. Abutments retain the earth behind the structure while supporting dead and live loads from the superstructure. Piers are primarily designed to transmit the loads to the foundations and to resist horizontal forces. The superstructure is comprised of all the other bridge components that are placed above the substructure. The decking and girders all help transmit generated live loads into the substructure (Bridge Masters, 2017).

2.5 Design Alternative: Aesthetic Improvements

A plenitude of options was considered to improve the NSCT bridge. The first option was to leave the bridge in its current condition without any structural modifications. However, if the bridge was left as is, rehabilitation efforts would be needed in the future to address any deficiencies. In order to maintain the bridge in its current state, the structure needed to be analyzed to determine that it was adequate to provide service for required loadings and deflections. The substructure was especially key during analysis since the inspection report suggested that the bridge may be at the end of its service life, due to deterioration of the timber piles. If the bridge could safely support its self-weight and expected live loads, then simple minor aesthetic improvements could be designed and the core of the structure could be left alone. Adding lighting, removing graffiti, and replacing the decking or railing were all viable design options to proceed with. Modifications of the trail leading up to the bridge could also be incorporated into the design such as repaving the path, landscape planning, graffiti removal, and adding benches or bike racks.

2.6 Design Alternative: Rehabilitation

If the bridge did not meet safety and serviceability requirements, then rehabilitation designs needed to be considered. Additionally, the NSCT bridge contains a 24-inch gas utility line that was of concern. Design and construction would need to be mindful of the gas line and ensure that proper measures were taken to preserve the pipe. There are also electrical power lines running along the bridge which are less cause of concern yet still needed to be addressed. The bridge may also have large historical significance to Saugus which would require a mindful approach during rehabilitation to ensure that the historical integrity of the structure was not compromised.

After reviewing case studies, there were several rehabilitation options that could have been pursued depending upon structural analysis results. New girders, joint replacement, deck strengthening, parapets replacement, abutment modifications, and pile upgrades were among the considerations that could be incorporated into rehabilitation design.

Case Studies

In 2007 at the request of the American Association of State Highway and Transportation Officials (AASHTO), the National Cooperative Highway Research Program (NCHRP) released a report called “Guidelines for Historic Bridge Rehabilitation and Replacement.” It provides a protocol for making decisions in the design process of historic bridge rehabilitation to bring them into conformance with current design and safety standards.

A series of 16 historic bridge projects made from stone, metal, or concrete were compiled by Statistical Research, Incorporated (SRI) in 2011. SRI is a cultural resource management firm that focuses on historic preservation with a variety of structures including bridges. The list of projects was compiled in conjunction with the NCHRP's Guidelines to help consider rehabilitation or replacement. Although arch and truss bridges comprised a majority (14) of the 16 studied, they provided valuable insight on how to proceed with historic bridge rehabilitation (Table 2). There was also a movable span and metal girder bridge studied in the analysis.

Table 2: Historic Bridges Studied

Type of Bridge	Name of Bridge	Location	State
Stone arch	Johns Burnt Mill Bridge	Mount Pleasant and Oxford Townships, PA	PA
Stone arch	Prairie River Bridge	Merrill, WI	WI
Concrete arch	Carrollton Bridge	Wabash River, IN	IN
Concrete arch	Robert A. Booth (Winchester) Bridge	Douglas County, OR	OR
Movable span	Bridge of Lions	St. Augustine, FL	FL
Metal truss	Tobias Bridge	Jefferson County, IN	IN
Metal truss	New Casselman River Bridge	Grantsville, MD	MD
Metal truss	Walnut Street Bridge	Mazeppa, MN	MN
Metal truss	Pine Creek Bridge	Borough of Jersey Shore, PA	PA
Metal truss	Washington Avenue Bridge	Waco, TX	TX
Metal truss	Lone Wolf Bridge	San Angelo, TX	TX
Metal truss	Goshen Historic Truss Bridge	Goshen, VA	VA
Metal truss	Hawthorne Street Bridge	Covington, VA	VA
Metal truss	Ross Booth Memorial Bridge aka Winfield Toll Bridge	Putnam County, WV	WV
Metal arch	Lion Bridges	Milwaukee, WI	WI
Metal girder	Hare's Hill Road Bridge	Chester County, PA	PA

The review of the bridges in Table 2 revealed that rehabilitation was the more expensive option compared to replacement due to the bridge's age and level of deterioration. Cost-effective techniques can be utilized to minimize high expenses such as using bolts instead of rivets and using welding to repair cracks in plates. However, rehabilitation was chosen due to each bridge's historical significance. In order to determine the bridge's significance to the community, having historical preservation experts involved in all stages of the rehabilitation process provides valuable guidance (SRI Foundation, 2011).

Additionally, early implementation and coordination with all stakeholders and resource agencies, such as the town of Saugus and the Saugus Rivershed Water Council, are essential to avoid disagreements regarding the project's decision-making process. Ultimately, actively engaging the community allows for the designer to obtain their support for the project.

If a bridge was built before standard specifications were issued, which is common with bridges built in the early-twentieth century, samples from beam and pile members need to be tested to determine data on the material's strength (SRI Foundation, 2011). Sample results will determine if structural components need to be replaced or simply upgraded. Furthermore, methods to reduce the bridge's self-weight, such as reinforced deck systems, allow for a raised live load capacity to increase the bridge's usability (SRI Foundation, 2011).

All the case studies highlighted that rehabilitation on a project will require constant inspection, analysis, and design to address unforeseeable conditions that may not be detectable until the construction process begins. Additionally, if there was little predicted increase in traffic flow, the bridge had undergone rehabilitation as opposed to replacement (SRI Foundation, 2011).

2.7 Design Alternative: Replacement

If neither aesthetic improvements nor rehabilitation are feasible options, the focus shifts to complete replacement. Common replacement structures include beam and girder, arch, truss, cantilever, suspension, and cable-stayed bridges. To narrow the scope of replacement options, the team conducted a review of 16 pedestrian bridge case studies to identify the most optimal replacement structure.

2.7.1 Replacement: Case Study Review

The study began with a review of eleven completed pedestrian bridges built by Contech Engineered Solutions. Contech Engineered Solutions is a nationwide engineering consulting firm that specializes in prefabricated structures. Of the eleven bridges that were reviewed, eight were truss bridges and the remaining three were arch bridges.

Additionally, the study analyzed three university reports, each detailing a pedestrian bridge design. A critical step in each report was exploring three to four possible bridge types, and all three reports considered beam and girder bridges as shown in Table 3. While none of the reports selected the beam and girder option for final design, it was a consideration for all the reports therefore illustrating its applicability to a range of pedestrian bridge projects.

Based on all 14 case studies, it was evident that the most common pedestrian bridges are beam and girder, truss, and arch. A variety of factors are considered prior to selecting one of these three designs, and these factors are discussed in the following section.

Table 3: Bridge Type Considerations from University Reports

Report #1 (Adams & Gould, 2014)	**Report #2 (DeCelle et al., 2013)**	**Report #3 (Raskett & Rebello, 2017)**
Simple Girder	Simple Beam	Simple Girder
Aluminum Truss	***Truss****	***Arch****
Whipple Truss*	Arch	Cable-Stayed
Flatcar	Cable-Stayed	N/A
Note: *** denotes that option was selected for final design		

2.7.2 Replacement: Design Considerations

There are five factors that must be considered in order to select the most suitable structure for a given bridge replacement project. A bridge's load path is the foremost important consideration, as it defines the manner in which load is transferred through connected members. It is obvious that a bridge must be capable of successfully transferring loads from its deck to the foundation, otherwise it is structurally deficient. The specific direction and members that the load passes through has implications on member sizes, connection details, and foundation design. For example, slender members subjected to compression must have sufficient compressive strength as well adequate bracing to prevent buckling whereas members subject to tension solely need sufficient tensile strength (McCormac & Csernak, 2018).

The second consideration is foundation design. A bridge is only as strong as the soil supporting it. As a result, foundations must distribute the bearing pressures in a manner that does not exceed the soil's bearing capacity. Additionally, foundations must secure the bridge to the supporting earth. For bridges that span bodies of water, foundations must extend far enough into the soil to safely guarantee prevention against under-scour by the movement of water (Fadum, n.d.).

The third and fourth considerations for replacement bridge design are span length and vertical clearance, respectively. The clear distance that a bridge can reasonably span without intermediate support structures is determined by design and material properties. The Federal Highway Administration (FHWA) estimates that continuous beam highway bridges constructed of standard steel W-sections have a maximum single span length of 120 feet, whereas arch bridges are capable of single spans greater than 150 feet (FHWA, 2015). When the distance that needs to be traversed exceeds the reasonable simple-span limit, intermediate piles must be installed. Similar to span length, the vertical clearance between a bridge's superstructure and water is critical for environmental and recreational factors. Bridges must accommodate for changes in water body height generated by high and low tides as well as flooding and tidal surges. In fact, NOAA National Storm Surge Hazard Maps estimate that the Saugus River at the location of the pedestrian bridge has the potential to experience a six-foot storm surge for a Category 1 hurricane. In terms of recreational reasons, the bridge needs to have adequate vertical clearance to allow for canoes, kayaks, and small motorboats to pass underneath the bridge (Code of Federal Regulation 33 CFR § 115.70).

The final consideration for replacement bridge design is constructability. Designs that utilize prefabricated segments of typical member sections will be more economical and easier to construct than design alternatives consisting of custom member sections that require on-site fabrication. Transportation and access to the site are two factors that have large implications on constructability. Members and prefabricated sections should be less than 120 feet in length to facilitate easier transportation, and only construction vehicles that can feasibly access the site should be used for construction (FHWA, 2015).

By utilizing these five considerations, design teams can select the most suitable replacement bridge type for their given project.

3.0 Methodology

The team's goal for the project was to assess the condition of the bridge and determine an alternative that best addressed the failing structure and its constraints. This goal was achieved through a series of three phases. The first phase focused on identifying bridge users and load requirements. The second phase surveyed the bridge's existing conditions and researched design alternatives. The third phase evaluated the design alternatives and accordingly developed a final design and recommendations. An outline of the three phases and their corresponding objectives can be found in Figure 5. A flowchart outlining the entire methodology can be seen in Appendix B.

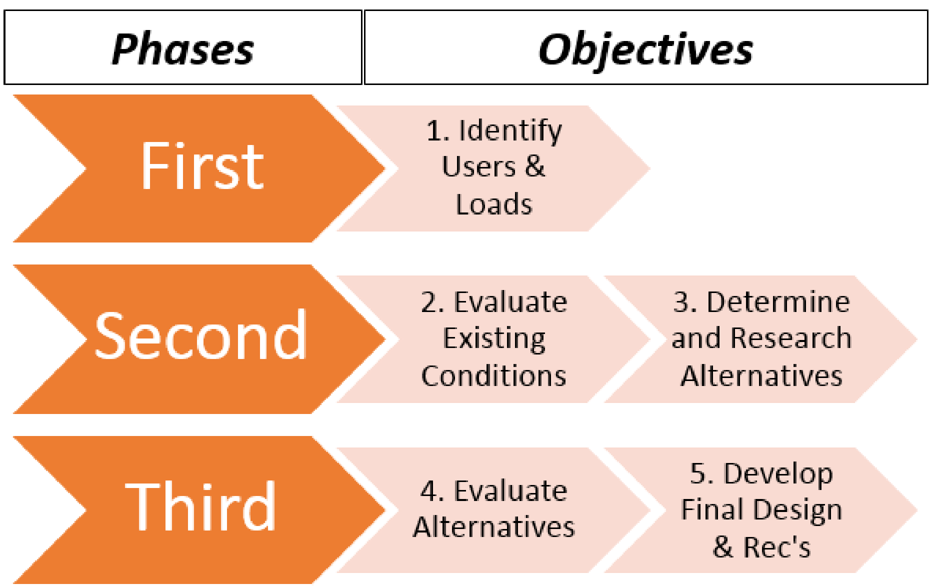

Figure 5: The Three-Phased Approach

3.1 Identification of Bridge Users and Load Requirements

First, the bridge's primary function was identified along with its main users. After identifying bridge users, associated loadings and loading requirements were determined in conjunction with the *AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges* (2009). Additional references included the *AASHTO LRFD Bridge Design Specifications, 8th Edition* (2017) and the *Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals, 6th Edition* (2015).

3.2 Evaluation of Existing Conditions

After identifying the bridge's users and loading requirements, the current conditions of the bridge and project site must be evaluated. The inspection report submitted by the Wood Advisory Services, Inc. was the primary resource used to better understand the extent of deterioration in the structural members of the bridge (WAS, 2018). Three site visits were then conducted by the team to provide further perspective on the bridge's structural conditions, surrounding environmental areas, and site access for future construction activities. These site visits also provided an opportunity to visualize the findings of the WAS report.

The Massachusetts Online Viewer (OLIVER), through MassGIS, was used to map zones of notable environmental features surrounding the project site. The GIS layers, specified in Table 4, were selected from an environmental screening summary provided by Stantec and used for analysis. These layers outlined potential environmental concerns, highlighted the applicable jurisdictional boundaries, and revealed permitting considerations associated with the project. Additionally, the Massachusetts Cultural Resource Information System (MACRIS) provided historic and cultural information regarding the bridge.

The team attended a meeting at the Stantec office, which was focused on hazardous material management. This meeting discussed topics directly related to the Northern Strand

Community Trail project and provided valuable information for the evaluation of the site conditions surrounding the bridge.

Table 4: MassGIS OLIVER Layers Used for Existing Conditions Evaluation

MassGIS OLIVER Layer Classes and Names	
Hydrography	DEP Wetlands (Detailed)
	FEMA National Flood Hazard
Conservation	NHESP Estimated Habitats of Rare Wildlife
	NHESP Priority Habitats of Rare Species
	NHESP Certified Vernal Pools
	Potential Vernal Pools
	Open Space by Level of Protection
	Areas of Critical Environmental Concern (ACECs)
Hazardous Materials	DEP Tier Classified 21E Sites
	Activity and Use Limitation (AUL) Sites

3.3 Research and Determination of Improvement Alternatives

The team researched alternatives for aesthetic improvements, rehabilitation, and replacement. The methods for research are described below.

3.3.1 Aesthetic Improvements

After evaluating the bridge's existing conditions, it must be determined if AASHTO requirements are satisfied. Due to the WAS report, the substructure was of particular concern. Research conducted on the life-span of timber bridges was a major consideration in evaluating the viability of only making aesthetic improvements to the bridge. Options as mentioned in "2.5 Design Alternative: Aesthetic Improvements" were discussed. Additionally, during site visits, options for aesthetic improvements were visualized.

3.3.2. Rehabilitation

Research for rehabilitation alternatives focused on addressing the bridge's most severely deteriorated members. These members are the greatest threat to the bridge's ability to support and transmit loading, and it was critical that any rehabilitation efforts addressed these specific members. Once the critical members were identified, the team researched proven timber rehabilitation techniques from the Federal Highway Administration (FHWA) as well as several state Departments of Transportation (DOTs). This research focused on the performance, constructability, environmental considerations, and costs associated with each rehabilitation method.

3.3.3 Replacement

The research of replacement alternatives focused on prefabricated and in-situ constructed beam and girder, truss, and arch bridges. These were identified to be the most common replacement structure types for pedestrian bridges as mentioned in "2.7.1 Replacement: Case Study Review." Research was conducted in line with the design criteria identified in "2.7.2 Replacement: Design Considerations." Research included academic publications as well as design resources developed by AASHTO, the FHWA, and state DOTs. Through this research, the team established additional design criteria that were later used when evaluating bridge replacement options.

3.4 Evaluation of Alternatives

The third phase began by compiling research into a Pro-Con List for each project alternative. The purpose was to identify the positive and negative attributes for each alternative based on the following criteria: health and safety, cost, environmental impacts, aesthetics, constructability, and ethics. These criteria were selected based off of the primary constraints identified in the capstone design statement and encompassed all major factors to consider when implementing a successful project. A template for the Pro-Con List can be found below in Table 5. The team's process for selecting the most favorable option is described in the following sections.

Table 5: Pro-Con List Template

	Project Alternative		
Attributes	Aesthetic Improvements	Rehabilitation	Replacement
Health and Safety	+/-	+/-	+/-
Demo. Cost	+/-	+/-	+/-
Const. Cost	+/-	+/-	+/-
Environmental Impacts	+/-	+/-	+/-
Aesthetics	+/-	+/-	+/-
Constructability	+/-	+/-	+/-
Ethics	+/-	+/-	+/-

After the project alternative was selected via the Pro-Con List, the team developed several design options. These design options were then placed through the decision matrix to evaluate which option best satisfied the six project criteria.

3.4.1 Decision Matrix: Criteria Weightings

The decision matrix evaluation began by assigning varying weights to each of the six criteria. Some criteria had greater significance to the overall project than others and the weighting scale ranged from one to three to reflect this. Table 6 highlights the resources used to determine weightings.

The weighting for the "Safety" criterion was determined based on its significance within the American Society of Civil Engineer's Code of Ethics. The weightings for "Economics" and "Environmental" were assigned based on Stantec's project budget and the sensitivity of the surrounding environment, respectively. The sensitivity of the surrounding wetlands and river was determined by site visits and the MassGIS OLIVER environmental analysis, specified in Section "3.2 Evaluation of Existing Conditions." The "Aesthetic" weighting was determined by the bridge's historical classification from MACRIS. Additionally, "Constructability" and "Availability" were weighted based on Stantec's proposed project timeline. Once determined, these weights were used as magnification factors for the ratings, which were based on the methods described in the next section.

Table 6: Resources Used to Weight the Decision Matrix Criteria

Criteria	Method of Determination			
	ASCE Code of Ethics	***Stantec Input and Resources***	***Research***	***Site Visits***
Safety	X			
Demolition Cost		X	X	
Construction Cost		X	X	
Environmental		X	X	X
Aesthetics			X	
Constructability		X	X	X
Availability		X	X	

3.4.2 Decision Matrix: Design Option Ratings

The ratings for each option were determined. Each identified design option had a separate category in the matrix and was assigned numerical ratings ranging from one to five, depending on the degree to which they satisfied the following criteria.

First, the safety of each design option was defined by its fracture critical classification. A fracture critical structure is one that experiences “complete structural failure upon failure of a single member” (FHWA, 2015). For this matrix, each design option was rated based on its resistance against fracture critical failure, and this rating was determined from the *FHWA Steel Bridge Design Handbook*.

The cost rating of each design option was determined from Section “3.3 Research and Determination of Improvement Alternatives” and previous Stantec project cost estimates from

Contech Engineered Solutions (Reardon, 2013 &2018). The cost was divided into two categories, demolition and construction costs.

The environmental rating of each design option was determined by the anticipated impact of construction activities on the surrounding wetlands, river, and wildlife. Resources from the *FHWA Steel Bridge Design Handbook* and Minnesota DOT provided initial insight into the construction methods for several design options (Phares B, 2015). Subsequent site visits were then used to identify means of construction that were feasible for the project site. Finally, the OLIVER environmental analysis provided aid to determine the environmental designations and regulations of the surrounding area, which limits the type of acceptable construction methods. The results of these three research methods were combined to rate each design option based on the anticipated environmental impact of its construction. The higher the design option's impact on the environment, the lower the rating was for that option.

The rating for aesthetics was determined by two factors: 1) visual appeal; and 2) resemblance to surrounding environment. The team began by observing images of past pedestrian bridges on the Contech Engineered Solutions website (Contech, n.d.). From there, site visits were used to assess how different design options would best fit in with the surrounding environment. Designs that would reduce vertical clearance or involve excessive disturbances to the wetlands were eliminated, whereas structures that would aesthetically complement its surroundings were favored.

The ratings for constructability were assigned based on the efficiency of the bridge construction process as well as the practicality of implementation on site. Each design option is associated with different levels of labor, and these ratings were based off of construction information provided by the *FHWA Steel Bridge Design Handbook*. Construction information included relative member weights, the number of members, the required amount of bolting or welding, and temporary supports during construction. Efficient designs that were projected to have a relative ease of construction were assigned higher values, whereas more labor intensive and complicated designs were rated lower.

Finally, the availability of each design option was determined based on the past projects of the following pedestrian bridge prefabricators: Contech Engineered Solutions, Big R Bridge, and Excel Bridge. These prefabricators were identified by Stantec as regional leaders for pedestrian bridge design and installation. The availability of each design option was based on their presence within the project portfolio of these three companies.

Once the decision matrix was completed, team members summed the weighted ratings for each design option across all criteria. The design option with the highest score was then selected as the preliminary design. A template of the decision matrix can be viewed in Table 7.

Table 7: Decision Matrix Template

Weights	Criteria	Design Option		
		Option A	*Option B*	*Option C*
To Be Determined	Safety	To Be Determined		
	Economical			
	Environmental			
	Aesthetics			
	Constructability			
	Availibility			

Rating Scale	
1	Poor
2	Below Average
3	Average
4	Above Average
5	Excellent

Weighting Scale	
1	Low Importance
2	Medium Importance
3	High Importance

3.5 Development of Final Design and Recommendations

After completing the activities in Section "3.4 Evaluation of Alternatives," the team proceeded with the best design option to develop the final design. The project concluded with the development of a prefabricated bowstring truss design for the Saugus River pedestrian bridge.

The final deliverables consisted of a structural design, structural models, abutment design, a project schedule structure that incorporated a staging and demolition plan, and also a cost estimate. An overview of the final superstructure design development can be found below in the following bulleted list. Complete spreadsheet and hand calculations for this process can respectively be found in Appendix F and Appendix G.

- Determined Span
- Determined Preliminary Geometry according to FHWA Guidelines
- Sized Preliminary Members
- Determined Loads
- Performed RISA-3D Analysis for LRFD Requirements
- Ensured Strength Requirements Sufficient
    - Updated member sizings and underwent RISA-3D Analysis until passing
- Ensured Stability Requirements are Sufficient
    - Updated member sizings and underwent RISA-3D Analysis until passing
- Ensured Vibration Requirements are Sufficient
    - Updated member sizings and underwent RISA-3D Analysis until passing

For the preliminary structural design, the team first determined superstructure geometry. The *FHWA Steel Bridge Design Handbook* specifies that the minimum truss height must be 10% of the total span length. Once the principle geometry was determined, preliminary members and member sizes were established based on previous shop drawings of a Contech prefabricated pedestrian bridge. Hand calculations were carried out to determine the live, dead, and wind loads based on governing codes identified in Section “3.1: Identification of Bridge Users and Load Requirements.”

After determining preliminary loads, the bridge was modeled in RISA-3D on a two-dimensional X-Y plane. RISA-3D was used for the primary structural design software since it provides a general two or three-dimensional analysis of applied loads to modeled structures. By specifying boundary conditions, the team was able to complete a 2D analysis on the 2D truss structure.

First, strength requirements were met through performing design iterations in a Microsoft Excel spreadsheet. The spreadsheet was then checked for accuracy with hand calculations. The spreadsheets and hand calculations were to ensure that the strength of the tension and compression members had sufficient capacity. Additionally, the slenderness ratio was checked (length/radius of gyration) for axial members to ensure that no buckling would occur. If any of these checks failed, members were resized and re-analyzed. This iterative design process was used until strength requirements were met.

After strength requirements were met, serviceability requirements were checked according to *AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges*. Stability of the top chord was ensured by verifying that the truss verticals were adequate to resist the lateral design force applied to them. Secondly, vertical deflections were checked to satisfy the unfactored pedestrian live load deflection. Horizontal deflections were analyzed to satisfy the unfactored wind live load deflection. With a total span length of 130 feet, the maximum deflection for both directions was set at 4.33 inches.

The last serviceability requirement that the structure had to meet was vibration limits in the vertical and lateral directions. Vibration limits were based on meeting the minimum frequency requirements. For the vertical direction, the fundamental frequency of the bridge without the presence of a live load must be greater than 3 Hertz to avoid the first harmonic frequency, while the lateral direction frequency must be greater than 1.3 Hertz.

An Eigensolution analysis through RISA-3D was used to determine if the structure passed vibration limits. Hand calculations approximated the truss as a simply supported beam with a 1-kip load placed in the center. These calculations assumed the load was placed at a single node with a single degree of freedom while software analysis analyzed the point load with 6 degrees of freedom at each node as the vibrations were generated throughout the structure. Software analysis yielded more accurate results compared to hand calculations of the structure's fundamental frequency. The team initially proceeded with hand calculations to determine vibration limits. However, hand calculations proved to be too conservative hence the software analysis was then incorporated into vibration limits. If any of these serviceability requirements failed, an iterative redesign process occurred until all criteria were satisfactory. Lastly, once all strength and serviceability requirements were met with updated member sizes and loads, the model was then rendered.

The abutments for the bridge were designed through the following process. First, a preliminary abutment type was chosen to best address the site requirements. Initial dimensions were then chosen for analysis. Live and dead loads from the bridge were determined after the structural design of the bridge was complete. In addition to these loads, self-weight and soil pressure forces acting on the abutments were calculated. Once all forces were determined, the governing limit state was chosen and applicable load modification factors were applied in accordance with the *AASHTO LRFD Bridge Design Specifications*. Necessary reinforcement was then found based on flexural resistance as specified by AASHTO LRFD. The team developed this design prior to knowing accurate soil properties; however, at this point soil tests were conducted by Stantec and the abutment design was adjusted to meet the found soil properties. Complete spreadsheet and hand calculations for this process can be found in Appendix I and Appendix J. The following criteria guided the design of the abutment walls:

- Determined Soil Condition
- Determined Abutment Type
- Determined Loads
- Sized Preliminary Abutment Dimensions
- Ensured Moment, Bearing, and Passive Resistance was Sufficient
- Determined Reinforcement

In addition to the structural design, a project schedule identifying the critical path was created for the chosen design option. This schedule identified critical tasks and milestones in the project to assist the pre-construction planning process. An expected duration for construction was determined as well as a cost estimate based on this schedule. The cost estimate was produced using material takeoffs and the material and labor prices set forth by Massachusetts Department of Transportation (MassDOT, 2019.)

4.0 Findings

The following sections delve into the team's findings throughout the project.

4.1 Identification of Bridge Users and Load Requirements

The identified bridge users were: pedestrians, bicyclists, and occasional emergency or maintenance vehicles. After consulting the *AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges* (2009), loading requirements were found for each user. Although equestrian loading is specified in the *Guide*, Stantec's trail planning team stated that this particular loading case was not applicable to this project. The Stantec team also specified the bridge's walkway must be at least 12 feet wide to accommodate trail expansion.

Pedestrian and Dead Loads

When designing structural members, the *Guide* required that pedestrian loads be set at 90 pounds per square foot. The final result was 0.54 kips per linear foot per truss. The dead load was approximated based on the linear weight of the central truss panel. Due to the bridge's nature as a bowstring truss, the amount of material and associated weight varies along the truss length. The team calculated the linear dead weight of the entire bridge based on the linear weight of the central truss panel. As a result, the total dead load of the bridge was approximated as 0.284 kips per linear foot per truss.

Vehicle Load

The vehicle loading was based on deck width. Since the deck was wider than 10 feet, an H10 design vehicle was needed. An H10 vehicle has a front axle loading of 4 kips and a rear axle loading of 16 kips. Additionally, the *Guide* requires the H10 vehicle loading to be applied over a

14-foot axle spacing and a 6-foot wheel spacing. It is important to note that the *AASHTO LRFD Bridge Design Specification* specifies that the vehicle load and pedestrian load shall not be applied to the bridge structure simultaneously. Therefore, the pedestrian load was the governing live load for structural analysis.

Wind Load

The horizontal wind loading was based on a design life of 50 years and 105 mile-per-hour design speed. The design wind pressure was calculated using the following equation: $P_z = 0.00256K_zGV^2I_rC_d$. All factors were obtained from *Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals, 6th Edition* (2015). Once the wind pressure was calculated, the projected vertical area that would be impacted by the wind was determined based on member surface areas. The product of wind pressure and projected vertical area was doubled to account for windward and leeward truss effects on both trusses. The total horizontal wind loading resulted in 0.735 kips per linear foot.

The vertical wind loading was identified as the wind load acting on the exposed underside of the superstructure at a windward quarter point. A vertical pressure of 0.02 kips per square foot was applied over the full deck width as required by the *AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges*. The total vertical wind loading resulted in 0.255 kips per linear foot. Next, the vertical wind loading was broken down into its windward and leeward components to determine their wind effects. Based on the results in Table 8, the leeward truss was used as the governing wind load combination.

Table 8: Windward and Leeward Vertical Wind Effects

Vertical Wind Force	Equation	Result
Windward	$F_W = WS_{V,T}*[(0.25*w_{deck})+(0.5*w_{chord})]/w_{bridge}$	67.29 plf
Leeward	$F_L = WS_{V,T}*[(0.75*w_{deck})+(0.5*w_{chord})]/w_{bridge}$	187.71 plf

Earthquake Loads

It is important to note that the team did not consider any earthquake provisions to ensure adequate seismic performance. According to Section 4.7.4.2 in the *AASHTO LRFD Bridge Design Specifications, 8th Edition* (2017), "seismic analysis is not required for single-span bridges, regardless of seismic zone." Since the project's final design consists of a single-span truss bridge, no seismic analysis was required.

Load Combinations

Once the governing vertical and lateral loads were determined, as seen in Table 9, limit states and associated load factors were identified according to *AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges*. Each vertical load was assigned a corresponding load factor for each limit state for every load combination per Table 10. Strength I generated a load of 1.30 kips per linear foot and served as the governing load combination for the remainder of the structural analysis.

Table 9: Governing Loads

Dead Loads	Pedestrian Live Loads	Vertical Wind Load	Horizontal Wind Load
0.393 klf	0.540 klf	0.187 klf	0.735 klf

Table 10: Load Combinations

	Load Factors		
Limit States	**Dead Loads**	**Pedestrian Live Loads**	**Vertical Wind Loads**
Strength 1	1.25	1.75	0
Strength 3	1.25	0	1.00
Service 1	1.00	1.00	1.00
Service 2	1.00	1.30	0
Service 3	1.00	1.00	0
Service 4	1.00	0	1.00
Fatigue 1	0	1.00	0
Extreme Event 1	1.00	0.50	0
Extreme Event 2	1.00	0.50	0

4.2 Evaluation of Existing Conditions

The following section addresses the findings for existing conditions of the bridge and the surrounding area.

4.2.1 Existing Bridge Conditions

Information from the Massachusetts Cultural Resource Information System (MACRIS), revealed that the bridge was built in 1943 and that there is no historical designation tied to the bridge due to its “fairly recent date of construction” (Boston & Maine Railroad, 1987).

Research shows that timber bridges have an expected maximum service life of 75 years when properly pressure-treated (Wacker & Brashaw, 2017). Since the bridge was built in 1943 and is creosote treated, applying a 75-year life span would render the structure past its recommended service life. Based on the timber borings from the WAS Report, the bridge's most heavily deteriorated members are the piles. Four piles are in critical condition, with less than 25% of remaining cross-section, and 18 piles are in sub-moderate condition, with more than 25% but less than 50% remaining cross-section. Additionally, the portion of the pile caps that extend beyond the width of the deck to support the utility line were in "poor" structural condition. Finally, all cross-bracing was described as "severely deteriorated" and "having already failed" (WAS, 2018).

Currently, the bridge is composed of 10 bents carrying both iron and wooden stringers and is a double-track wooden trestle that carries a 24-inch gas line. Dennis Reip, a structural engineer for Stantec, stated that no utility company has claimed ownership of the gas line, which limits the potential for future relocation activities (January, 2019). Due to the lack of ownership, alterations of the utility line by any party would require said company to assume liability and poses too high a risk. As such, it's recommended that a construction plan should leave the gas line undisturbed to eliminate liability.

4.2.2 Existing Environmental Conditions

River and Wetlands

The team's research of the environment surrounding the bridge began with the underlying river and the adjacent wetland. By nature of its location over the Saugus River, the bridge is located in FEMA Flood Zone "AE," as identified by the FEMA Flood Insurance Rate Map found in Appendix C and seen in Figure 6. The land within this zone is susceptible to flooding by a 100-year storm, and can reasonably expect a flood elevation of 10 feet above sea-level. As a result, a design that seeks to reduce flood damage to the bridge must place the superstructure above the 10-foot flood elevation.

The Saugus River is designated as an estuary and is affected by tidal shifts and therefore falls under jurisdiction of Chapter 91 of the Massachusetts Public Waterfront Act. As such any, construction activities on the river will have to comply with the provisions set forth in Chapter 91. This includes licensing for alterations/demolition of existing structures and the construction of new structures. The licensing process is extensive and requires approval by the Waterway Regulations Program, Massachusetts Executive Office of Energy and Environmental Affairs, Massachusetts Environmental Policy office, and the surrounding community. Additionally, the bridge's location falls within the 100-foot protective buffer of the adjacent wetland. As such, the project is also subject to the regulations of the Wetlands Protection Act (WPA), which requires the approval by the presiding local conservation commission. Approval by the Town of Saugus' Conservation Commission will hinge on the project's ability to perform the desired work without harming the wetland. As a result, it is vital that any final design and recommendations carefully consider and minimize environmental impact to ensure the success of the project.

Finally, the Saugus River is designated by the United States Coast Guard as navigable by logs, log rafts, rowboats, canoes, and small motorboats as specified in the Code of Federal Regulations 33 CFR § 115.70 (H. King, personal communication, February 11, 2019). With this designation, the placement of the bridge's lowest horizontal member should maintain the current clearance that the existing structure provides.

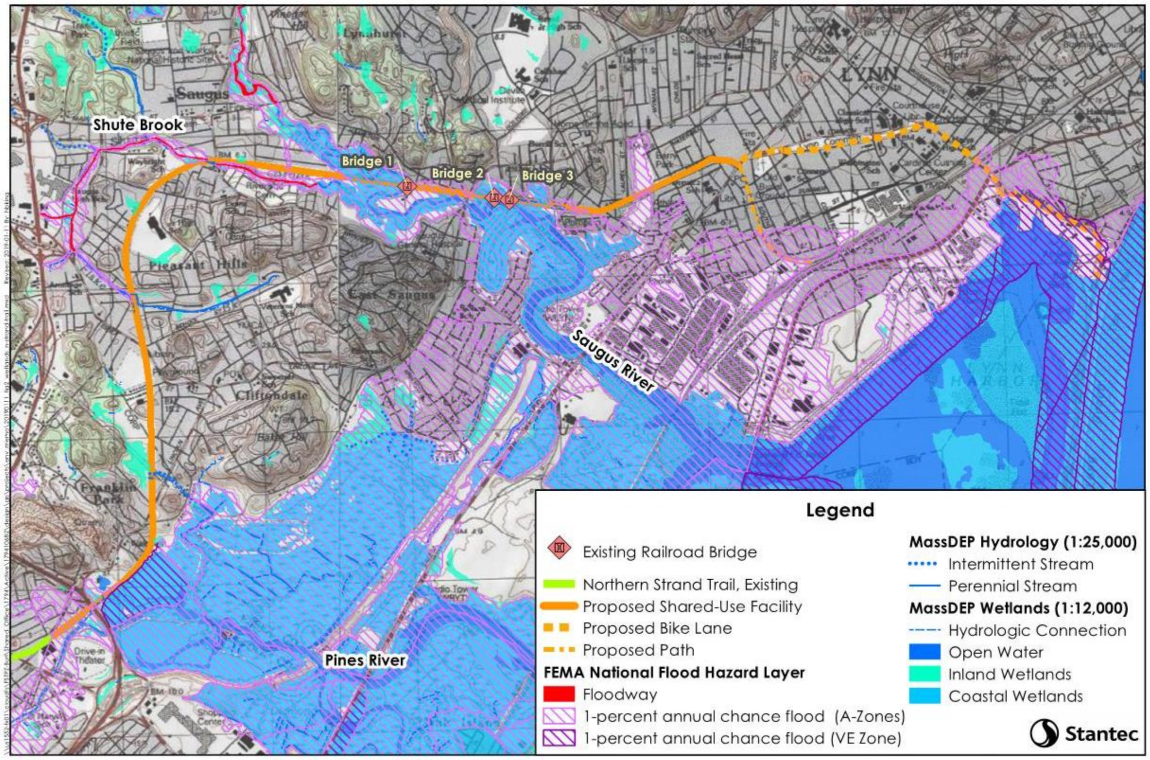

Figure 6: DEP Wetlands and FEMA Flood Zones (Stantec, 2019)

Open Space

Input from Stantec, MassGIS OLIVER research, and a team site visit revealed that site access is limited. The most optimal means of access is from the path's eastern approach, off of Lincoln Avenue or Rhodes Street. The open space adjacent to the path -- the Bacon Property, also known as the Saugus River Reservation -- is currently designated as a passive recreation site, meaning the only activities that the space accommodates is walking or hiking as seen in Figure 7 (Town of Saugus, 2018).

After investigating the project's surrounding area through OLIVER, there was initially no indication that the area had any environmental hazards. However, Joseph Salvetti, a Stantec Licensed Site Professional (LSP), revealed that the Bacon Property had once been used for industrial purposes. The property was identified by the Executive Office of Energy and

Environmental Affairs as having a host of contaminants in the soil, including petroleum, polychlorinated biphenyl (PCBs), and lead. The Bacon Property was previously used as a power generation facility, construction yard, welding shop, and garage. The site served as a power generation facility between 1915 and 1930, storing electrical transformers on site. From 1963 until 2004, there were five 265-gallon aboveground storage tanks (AST), one 1000-gallon underground storage tank (UST), and one 2000-gallon UST (Martzolf and Salvetti, 2018).

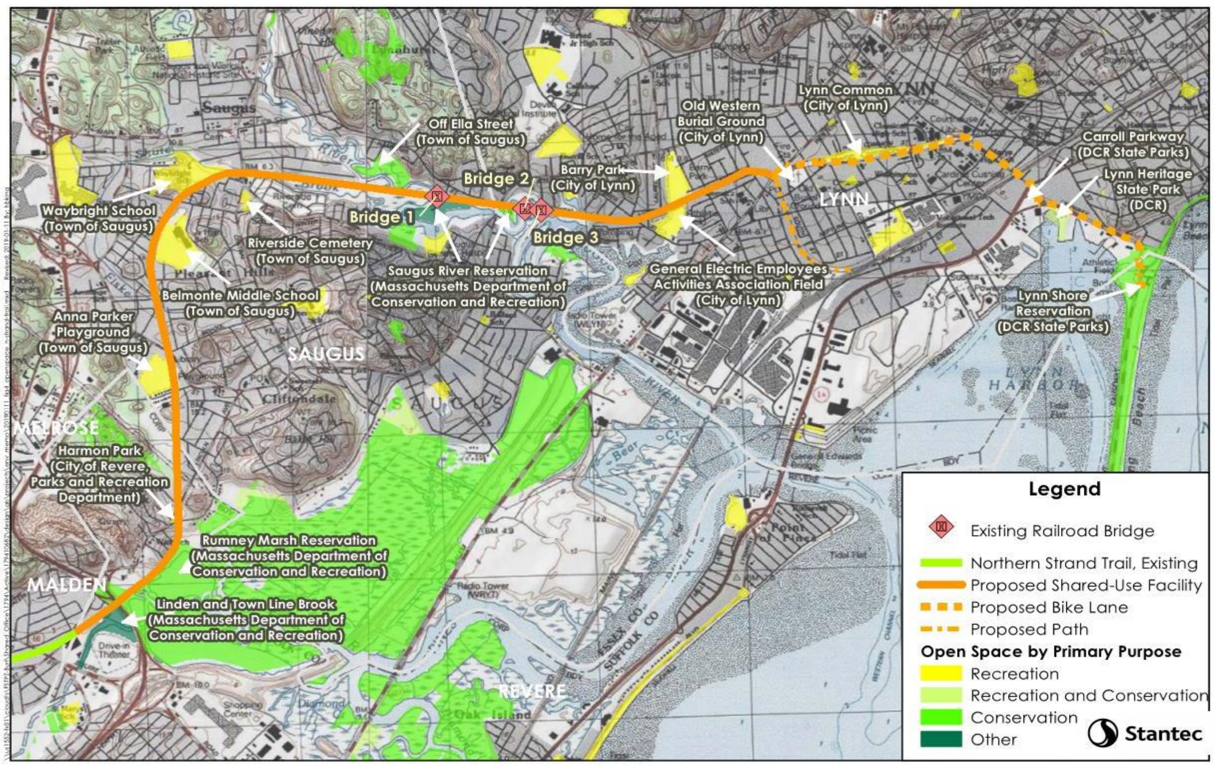

Figure 7: Protected and Recreational Open Space (Stantec, 2019)

This presence of contamination on the Bacon Property has the potential to deter contractors from using the site for staging purposes. However, Joseph Salvetti, LSP advised that soil tests be taken to identify locations of the property where contamination is not present. By identifying clean areas, it would then be possible for a contractor to stage equipment in a location that would not require them to risk becoming liable for remediation of preexisting contaminates.

Areas of Critical Environmental Concern

Areas of Critical Environmental Concern (ACEC) are key areas where additional management is needed in order to protect significant natural resources and cultural values (NEPA, 2018). The program was created when the Federal Lands Policy and Management Act was passed to establish conservation efforts for the Bureau of Land Management. Endangered species, uncommon geological features, and wildlife resources are protected through ACECs.

In Massachusetts, an ACEC is defined as a place that “receives special recognition because of the quality, uniqueness, and significance of its natural and cultural resources” (Mass.gov). The program was established in 1975 when the Massachusetts legislature appointed the Secretary of Environmental Affairs to identify and designate areas of environmental concern (Mass.gov). Presently, 30 ACECs have been designated to cover approximately 268,000 acres in 76 communities (Mass.gov). However, the area surrounding the bridge site is not designated as an ACEC and will not be an issue of concern for any future planning efforts as seen in Figure 8.

Waste Sites and Reportable Releases

The Massachusetts Department of Environmental Protection (DEP) is responsible for guaranteeing clean air, land, and water, and preserving wetlands and coastal resources. The DEP has three programs that provide remediation for contamination in the environment. The 30 CMR 40.000 of the Massachusetts Contingency Plan (MCP) outlines the process of how a contaminated site must be assessed and remediated. Reported releases of oil and or hazardous materials at the site would dictate any special management practices that would be implemented for any future construction activities (Mass.gov). The DEP has an online search to obtain additional information of release locations through the Waste Site Reportable Releases lookup. However, there are no reportable releases of any waste sites that coincide with the railway corridor (Mass.gov). However as previously mentioned, the Bacon Property is of noticeable concern and does appear as a waste site. Construction activities that utilize the Bacon Property will need to take cautionary measures as previously mentioned.

Hazardous Materials

The most commonly found contaminants along railway corridors (like the NSCT) are metals, herbicides, pesticides, PCBs, and petroleum products because they are widely used in railroad operations (Stantec Environmental Planning Team, 2019). Other chemicals that could be found are due to older railroad usage which include coal and ash from engines, creosote from coated timber ties, and waste oils from railroad dust control operations (Mass.gov). However, the aforementioned chemicals are exempt from reporting requirements due to the MCP and historic usage of the site (Mass.gov). While the bridge site will not need to report of any hazardous materials, measures should be taken to minimize dust and soil disturbances during construction.

Natural Heritage and Endangered Species

The Natural Heritage and Endangered Species Program (NHESP) is vital for the protection and conservation of species and their habitats. Massachusetts has 427 endemic plant and animal species that are protected under the Massachusetts Endangered Species Act (MESA) (M.G.L. c.131A). Areas that can be designated as a Priority Habitat of Rare Species include wetlands, uplands, and marine habitats (Mass.gov). They are based on the known geographical location of state-determined rare species including both plants and animals, as specified under MESA. Additionally, Estimated Habitats of Rare Species are a subset of priority habitats and are based on the geographical location of state determined rare wetlands wildlife as specified under the WPA (Mass.gov). As both estimated and priority habitats are part of the NHESP, designations of such lands on a potential project generates the process to ensure that the project is in compliance with all regulations (Mass.gov). However, there are no priority or estimated habitats in its vicinity as seen in Figure 8. Therefore, there is no concern of damaging the habitats of rare species during construction activities.

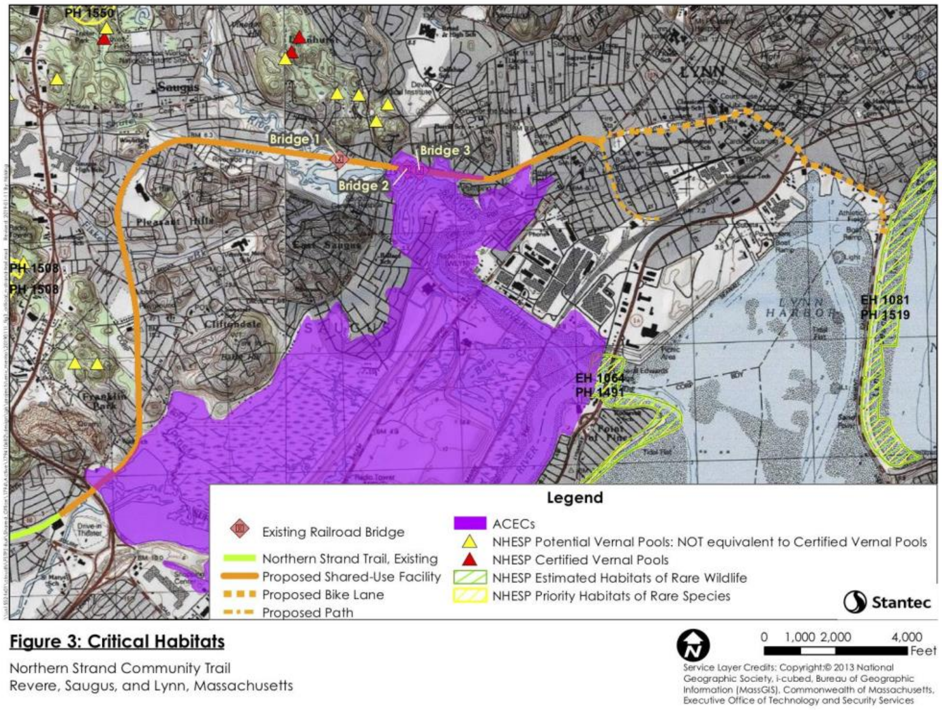

Figure 8: Saugus River Bridge Critical Habitats, ACECs, and NHESPs (Bridge 1) **(**Stantec, 2019)

Necessary Permits

Table 11 outlines all applicable permits for project scope. Prior to beginning construction, the following permits will have to be filed and approved.

Table 11: List of Applicable Permits (Source: Stantec Environmental Planning Team, 2019)

Permit	Agency	Activity
Notices of Intent (NOI) / Order of Conditions (OOC)/	Local Conservation Commission/ MassDEP Wetlands Division	Alteration of wetland or work in the regulated area
Environmental Notification Form (ENF)	Massachusetts Environmental Policy Act	This form is a first step in obtaining a Chapter 91 license
Construction Access Permit	Department of Conservation and Recreation	Staging of construction equipment on DCR owned property
Coastal Zone Management (CZM) Federal Consistency Review	MA CZM	MEPA requires CZM review for this project
Chapter 91 Section 14 Structure License	MassDEP	Demolition of existing structure and erection of new structure

4.3 Research and Determination of Improvement Alternatives

4.3.1 Aesthetic Improvements

The team considered two types of aesthetic improvements: direct improvements to the bridge's superstructure and improvements to the bridge approach. Direct improvements to the bridge included removal and upgrade of decking and railing, graffiti removal, and the addition of overhead lighting. Modifications of the bridge approach included the addition of benches and bike racks, repaving, and landscaping that would enhance the natural surroundings.

4.3.2 Rehabilitation

Based on the current state of the bridge described in Section "4.2.1 Existing Bridge Conditions," any effective rehabilitation efforts would have to address the piles, pile caps, and cross-bracing.

With regards to piles, there are a wide array of rehabilitation methods for mild-to-moderate cases of deterioration. The most common method for pile rehabilitation, is to wrap the deteriorated section of the pile in order to restore a portion of the lost cross-section. One such method consists of wrapping the pile in a fiber reinforced polymer (FRP) and filling the deteriorated cross section with polyurethane grout as shown in Figure 9 (Phares, 2015). A second method encases the pile with corrugated metal piping that is then filled with concrete. These two methods not only serve to improve axial and bending capacity, but also create an outer shell that serves as a barrier against future deterioration (Phares, 2015). A third method becomes particularly useful if piles within the same bent require rehabilitation. Rather than separately encasing each of the piles, the Minnesota DOT recommends encapsulating the piles in a single concrete grade beam (Phares B, 2015).

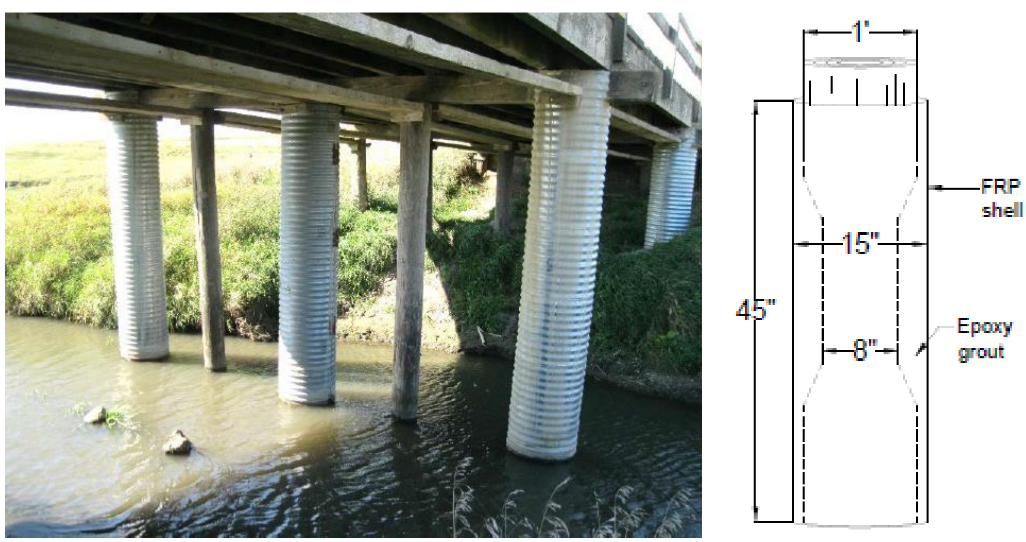

Figure 9: Pile Metal Jacketing, on left (Phares, 2015) and
Pile FRP Wrap with Epoxy Grout, on right (White et al., 2007)

For more severe cases of pile deterioration, splicing becomes the preferred method of rehabilitation. Splicing a pile begins with shoring the pile cap with a strut and jack. Once the pile is no longer a part of the load path, the deteriorated segment is removed, and a new pile of similar diameter is inserted into place as seen in Figure 10. The new pile section is fastened to create continuity with the original remaining pile either through timber fishplates, boltings with epoxy, or FRP wrap. Finally, the jacking is removed and the pile cap rests on the restored pile (Phares, 2015).

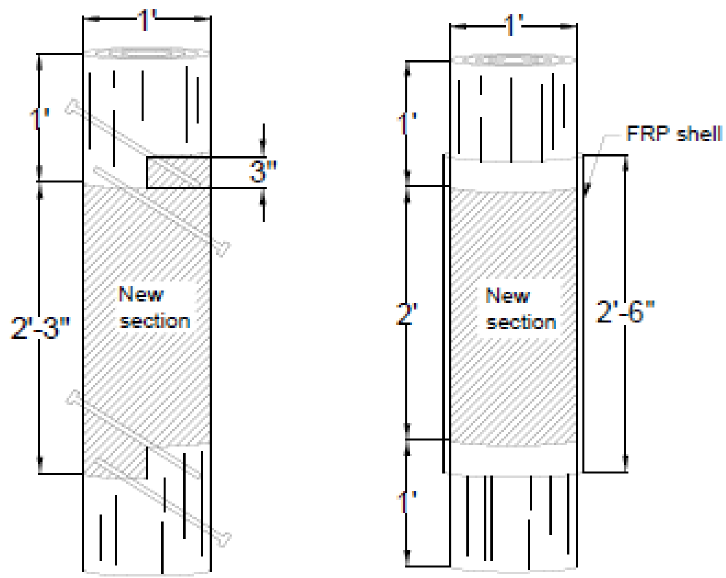

Figure 10: Splicing with Bolt Fasteners, on left and Splicing with FRP Wrap, on right (White et al., 2007)

Compared to pile rehabilitation, techniques for pile cap restoration are limited. To replace such a member, the bridge superstructure is jacked up and set on a temporary cap as seen in Figure 11. Once the superstructure is raised, the old cap is cut, removed, and replaced by a new cap, which is slid into place. Finally, the jacks are lowered and the superstructure is fastened to the new pile caps (Phares B, 2015). The WAS report identified that the pile cap sections underneath the bridge experienced minimal deterioration. However, the complete replacement of all 10 pile caps would be required in order to address the advanced deterioration located on the cap segments that extend beyond the deck.

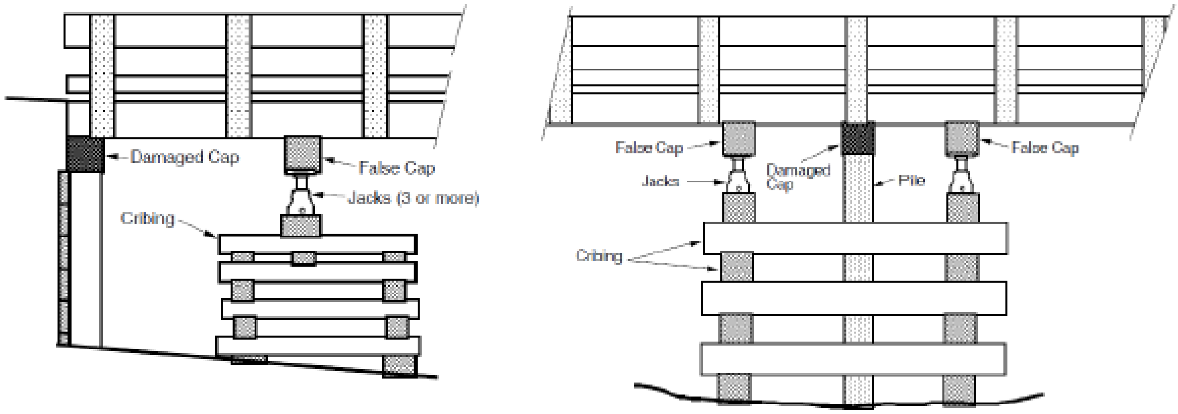

Figure 11: Pile Cap Jacking and Rehabilitation (Johnson, 2002)

Lastly, the team found that there is limited potential to rehabilitate the already failed cross-bracing. The combination of the severe deterioration and lack of cross-bracing remaining on the structure makes replacement rather than rehabilitation a more feasible approach to address the deteriorated cross-bracing.

In addition, all of the aforementioned rehabilitation methods would require the construction of a cofferdam and a dewatering operation due to the bridge's location over the Saugus River.

4.3.3 Replacement

The team researched three bridge replacement options: beam and girder, truss, and arch. These three bridge types were chosen as they are commonplace for replacement pedestrian bridge projects. Each replacement type considered safety, economics, environmental impacts, aesthetics, and constructability. Additionally, any replacement options would need to take the current existing structure and utility line into account.

Beam and girder bridges have a continuous span where loading is transferred from the decking to the floor beams into the abutments. Safety for bridge users is an issue of concern as lateral bracing becomes critical when spans exceed more than 200 feet (FHWA, 2015). However if the span is under 200 feet, the bridge can be easily constructed which, as a result, yields lower costs and lateral bracing is also not a cause for concern. Although beam and girder bridges have easy constructability characteristics, it is not practical to consider this design for the site because it would require the installation of intermediate supports and direct construction within the river. Another safety issue that was of significant concern is that beam and girder bridges are failure critical. This phenomenon occurs when failure of one member can directly lead to failure of the whole structure. Furthermore, environmental impacts are considerable since the span over the river is larger than the 120-foot continuous span guideline provided by the FHWA.

With this information, the team researched a through-girder design seen in Figure 12, a common design type for a beam and girder bridge replacement. Two girders are placed near the edge of the deck while shallow floor beams connect the bottom flanges of the girders. This bridge type is excellent for meeting vertical clearance requirements underneath the structure. Yet this bridge is classified as fracture critical and needs to be considered when determining the best replacement structure.

Figure 12: Through-Girder Bridge Example (Steel Construction Information)

The second bridge type that was researched was a truss bridge. Similar to beam and girder bridges, truss bridges have a significant safety issue as they are classified as fracture critical. However, they are advantageous because the member arrangement generates compression and tension rather than bending in members (FHWA, 2015). Additionally, this configuration allows for cost efficiency as lighter and more slender members can carry the same forces in beam and girder bridges (Hibbeler, 2015). A typical single span for a truss bridge ranges from 54 feet up to 183 feet based on the case study reviews of 13 Contech pedestrian bridges. Use of a single span for a truss bridge minimizes environmental impacts by reducing the amount of construction activity within the river and surrounding wetlands.

As a result, the two truss bridges that best accommodated the existing project site conditions were through and half-through trusses. A through truss places the deck flush with the bottom chord. This deck placement is optimal for project sites that have a restriction on vertical clearance. However, the bridge depth is completely controlled by the floor system depth and it is difficult to widen the deck once the trusses are put in place (FHWA, 2015). A half-through truss places its deck between the top and bottom chord but must be placed with enough height in order to install sway bracing below the deck. A half-through truss is a viable design option when the clearance level beneath the bridge is limited (FHWA, 2015).

The last bridge that was researched was an arch bridge. The natural curve in the arch creates a flattering aesthetic that enhances the bridge’s natural surrounding environment. However, an arch bridge requires heavier members and are taller than either truss or beam and girder bridges. Generally, heavier members cost more as they require more material. The costs are also higher for arch foundations as they require deep elements to support the compressive forces generated by the arch. Deeper foundations can also cause a large environmental impact in the surrounding area as they require more effort to construct them. On the other hand, tied arch bridges have vertical tension cables with the deck placed at the bottom of the arch. This configuration is able to eliminate the large thrust forces that are generated and as a result, can be built upon weaker soils (O’Brien, 2015). Additionally, a tied arch bridge can easily span 115 feet which reduces the environmental impact surrounding the construction site since no piers are needed in the water to help support the structure.

Additionally, all bridge types have the potential to be prefabricated, meaning that they are built off-site and then delivered in segments to be assembled on-site. This allows for improved constructability and minimizes the impact of construction activities on site.

4.4 Evaluation of Alternatives

4.4.1 Pro-Con List

The Pro-Con List shown in Table 12 evaluated five separate project alternatives, including aesthetic improvements, rehabilitation, and three replacement approaches with varying amounts of demolition. A complete evaluation can be found in Appendix D.

Table 12: Abridged Pro-Con List Evaluation of Design Alternatives

	Project Alternatives					
Attributes	**Aesthetic Improvements**	**Rehabilitation**	**Replacement (No Demo)**	**Replacement (Partial Demo A)**	**Replacement (Partial Demo B)**	**Replacement (Full Demo)**
Ethics	VIOLATE	+	+	+	+	VIOLATE
Health and Safety	n/a	+	VIOLATE	+	+	n/a
Demolition Costs	n/a	+	n/a	+	-	n/a
Construction Costs	n/a	-	n/a	+	+	n/a
Environmental	n/a	-	n/a	+	-	n/a
Aesthetics	n/a	-	n/a	-	+	n/a
Bridge Constructability	n/a	-	n/a	+	+	n/a

Aesthetic improvements consisted of non-structural alterations to improve the appearance of the bridge, including deck upgrades, graffiti removal, and lighting installation. The rehabilitation alternative was defined as structural repairs to all bridge members with greater than 50% deterioration. These repairs would consist of splicing the four critical piles, jacketing the 18 sub-moderate piles, replacing 10 pile caps, and replacing all cross-bracing. Finally, four replacement alternatives were considered during this evaluation, all of which included complete superstructure and abutment replacement. “Replacement with No Demolition” consisted of leaving the entire existing structure untouched and placing the new structure over it. “Replacement with Partial Demolition A” was comprised of cutting bents between the pipeline and walking surface along with the removal of the decking, railing, and pile caps. This alternative would leave the piles in the river, while allowing enough vertical clearance to span the new bridge over it. “Replacement with Partial Demolition B” includes removal of decking, railing, and pile caps while cutting back the piles below the river’s mudline. Finally, “Replacement with Full Demolition” consisted of complete removal of all existing structural members, including those under the gas line.

“Aesthetic Improvements” and “Replacement with Full Demolition” were eliminated from project consideration due to their failure to comply with the ASCE Code of Ethics. Only performing aesthetic improvements on a structurally deficient bridge jeopardizes the safety of bridge users, thus violating the Code’s first canon, “Hold Safety Paramount.” Furthermore, replacement with complete demolition of the existing bridge would involve removal of the supports under the 24-inch gas pipeline. This type of work involving a live gas line which borders a residential community violates the commitment to safety in the ASCE Code of Ethics. Additionally as stated in Section “4.2.1 Existing Bridge Conditions,” no company has taken ownership of the utility and altering the supports of the pipeline without the consent of the utility company violates the Code’s sixth canon, “Uphold Professional Honor” (ASCE, 2018).

The design potential for “Replacement with No Demolition” was eliminated from project consideration due to its violation of health and safety requirements. The new bridge is to be constructed with a 75-year life span, consequently locking the existing structure in place beneath it. As the existing structure continues to deteriorate, it is at risk of collapse and injury of recreational boaters passing underneath. Additionally, the bridge site is located within the 100-year flood zone of the Saugus River. By the same token, as the existing bridge continues to deteriorate and weaken, it runs the risk of collapse during an extreme flood event. This would mobilize debris and become a hazard downstream.

The design potential for "Rehabilitation" was limited by its environmental implications on the Saugus River and surrounding wetlands. As per Section "4.3.2 Rehabilitation," pile jacketing and pile cap replacement would require the construction of a cofferdam and a dewatering operation within the Saugus River. This physical disturbance to areas within the 100-foot wetland buffer would lead to extensive environmental permitting and expensive environmental control measures. As a result, this alternative did not satisfy the environmental, constructability, or cost criteria.

Finally, the team identified the two remaining alternatives "Replacement with Partial Demolition A" and "Replacement with Partial Demolition B" as the options with the largest positive design potential. Both alternatives involve installing a new structure and abutments. Therefore, pedestrians, cyclists, and recreational boaters will be given a safe means of traversing the Saugus River, satisfying the "Health and Safety" and "Ethics" attributes. The two alternatives were too close in evaluation to make an objective decision and proceed with one final design. As such, cost estimates and schedules for both alternatives will be presented in Section "5.0 Final Design and Recommendations" for consideration.

4.4.2 Decision Matrix

Once "Replacement A and B" were selected as the best design alternatives, the team had to determine the best bridge type option for replacement. The three replacement prefabricated bridge options included beam and girder, truss, and tied arch. The replacement alternatives were evaluated using the decision matrix template and criteria described in Section “3.4 Evaluation of Alternatives.”

4.4.3 Criteria Weightings

The “Safety” and “Environmental” criteria received the highest weightings because they served as the primary evaluation criteria for the decision matrix. It was the team’s primary goal to uphold the safety of the public and the health of the surrounding natural environment, as put forth by the ASCE Code of Ethics (ASCE, 2017). “Cost,” “Constructability,” and “Availability” were considered secondary criteria resulting in a moderate weighting assignment. These criteria together impact the feasibility of a project, and the team was committed to selecting a design that was a realistic solution to the currently deteriorating bridge. Finally, “Aesthetics” was assigned the lowest weighting due to the bridge’s classification as a non-historic structure.

4.4.4 Design Option Ratings

The results of the decision matrix can be found in Table 13. The “Prefabricated Truss” received a scoring of 52, which was the greatest out of the three options. This score was due to the high safety, environmental, and constructability ratings in comparison to the other options. Although truss bridges are classified as fracture-critical by the FHWA, load path, structural, or internal member redundancy can provide resistance to this type of failure (L. Albano, personal communication, January 25, 2019). If the truss is not internally indeterminate or no parallel load paths exist, internal member redundancy can be ensured through built-up member detailing which provides a mechanical separation of elements to limit fracture proliferation across the entire member section (FHWA, 2012).

Additionally, truss bridges are capable of spanning up to 180 feet without intermediate supports, which eliminated the need for direct construction work within the Saugus River (FHWA, 2015). Finally, truss bridges make up the overwhelming majority of pedestrian bridge projects completed by Contech Engineered Solution, Big R Bridge, and Excel Bridge. This finding indicated that truss bridges are widely available and preferred for this type of short-span pedestrian bridge project.

Table 13: Results for Design Option Decision Matrix

Weights	Criteria	Design Options		
		Prefabricated Simple Girder	*Prefabricated Truss*	*Prefabricated Tied Arch*
3	Safety	6	9	12
2	Cost	6	8	4
3	Environmental	6	12	12
1	Aesthetics	2	3	5
2	Constructability	6	10	6
2	Availibility	10	10	6
	Total	***36***	***52***	***45***

The "Prefabricated Tied Arch" received a scoring of 45, which fell short of the score earned by the "Prefabricated Truss." Although the tied arch scored the highest out of the three options in terms of safety and aesthetics, it did not excel in terms of economics, constructability, or availability. The curved nature of tied arch bridges complicates the fabrication and erection processes, resulting in an overall more expensive design (FHWA, 2015).

The "Prefabricated Beam and Girder" received a scoring of 36, which was the lowest out the three options. This option received a particularly low "Safety" rating, due to the fracture-critical nature of the main girders that support the entire superstructure. Furthermore, this option received a poor "Environmental" rating due to its inability to traverse the current 115-ft span without requiring an intermediate support (FHWA, 2015).

Based on the results of the decision matrix, the “Prefabricated Truss” is the most optimal replacement design option. The final design and recommendations for implementation are presented in the following section.

5.0 Final Design and Recommendations

Through careful analysis and research, the team created a final design of the bridge and abutments. A demolition plan, construction plan, cost estimates, and project schedules were also included in the final recommendations to implement the new structure.

5.1 Final Structural Design

A total of 10 design iterations were analyzed as summarized in Table 14. Designs 1 through 3 analyzed the trusses as two-dimensional structures in RISA-3D until strength and stability requirements were met. Designs 4 through 10 analyzed the trusses as a single three-dimensional structure in RISA-3D until vibration requirements were met.

Table 14: Design Iterations

Design Number	Primary Member Sizes	Reason for Failure
Design 1 (Two-Dimensional)	**Top/Bot. Chords:** HSS 10x10x3/8 **Verticals:** HSS 10x6x3/8 **Diagonals:** HSS 6x4x1/4	**Unreasonable Overdesign**
Design 2 (Two-Dimensional)	**Top/Bot. Chords:** HSS 6x6x5/16 **Verticals:** HSS 5x5x3/16 **Diagonals:** HSS 4x4x1/8 **Floor Beams:** W8x10	**Failing Top Chord Stability (Exceeded Slenderness Limit)**
Design 3 (Two-Dimensional)	**Top/Bot. Chords:** HSS 6x6x5/16 **Verticals:** HSS 6x6x1/8 **Diagonals:** HSS 4x4x1/8 **Floor Beams:** W10x19	**Satisfied Top Chord Stability, But Unreasonable Vertical Clearance**
Design 4 (Three-Dimensional)	**Top/Bot. Chords:** HSS 8x8x3/16 **Verticals:** HSS 7x7x3/16 **Diagonals:** HSS 4x4x1/8 **Floor Beams:** W10x30	**Failing Vibrations (via Hand Calculations)**
Design 5 (Three-Dimensional)	**Top/Bot. Chords:** HSS 9x9x1/4 **Verticals:** HSS 7x7x3/16 **Diagonals:** HSS 4x4x1/8 **Floor Beams:** W10x30	**Failing Vibrations (via Hand Calculations)**
Design 6 (Three-Dimensional)	**Top/Bot. Chords:** HSS10x10x1/4 **Verticals:** HSS 7x7x3/16 **Diagonals:** HSS 4x4x1/2 **Floor Beams:** W10x30	**Failing Vibrations (via Hand Calculations)**
Design 7 (Three-Dimensional)	**Top/Bot. Chords:** HSS 12x12x5/8 **Verticals:** HSS 7x7x3/16 **Diagonals:** HSS 4x4x1/8 **Floor Beams:** W10x30	**Failing Vibrations (via Hand Calculations)**
Design 8 (Three-Dimensional)	**Top/Bot. Chords:** HSS 12x12x5/8 **Verticals:** HSS 7x7x3/16 **Diagonals:** HSS 4x4x1/8 **Floor Beams:** W10x30 **Floor Diagonals:** W6x12	**Passed BUT Lacked Railing Beam**
Design 9 .(Three-Dimensional)	**Top/Bot. Chords:** HSS12x12x5/8 **Verticals:** HSS 7x7x3/16 **Diagonals:** HSS 4x4x1/8 **Floor Beams:** W10x30 **Floor Diagonals:** W6x12 **Railing Cross Beam:** W8x10	**Failed Vibrations (via Hand Calculations)**
Design 10 - Final (Three-Dimensional)	**Top/Bot. Chords:** HSS 9x9x3/8 **Verticals:** HSS 7x7x3/16 **Diagonals:** HSS 4x4x1/8 **Floor Beams:** W10x30 **Floor Diagonals:** W6x12 **Railing Cross Beam:** W8x10	**PASSED (via RISA-3D Analysis for Vibrations)**

The bridge's final geometry and member sizes were based on the generated loads as seen in Section "4.1 Identification of Bridge Users and Load Requirements." The bridge span was set as 130 feet, allowing for a 7.5-foot setback from the existing abutment to avoid any underlying structural members. In accordance with the *FHWA Steel Bridge Handbook* guidelines, the truss height was set at 13 feet. The bridge's final geometry can be found in Table 15, while the final member sizes are summarized in Table 16.

Table 15: Final Geometry

	Final Geometry			
	Span	Deck Width	Truss Centerline to Centerline	Truss Height (at center)
Dimensions	130 ft	12.75 ft	13.75 ft	13 ft

Table 16: Final Member Sizes and Strength Requirements

	Member Sizes					
	Top Chords	Bottom Chords	Truss Diagonals	Truss Verticals	Floor Beams	Diagonal Cross Bracing
Members	HSS 12x12x10	HSS 12x12x10	HSS 4x4x2	W 7x7x3	W 10x30	W 6x12
Maximum Axial Force (kips)	212.73	214.38	11.10	19.50	n/a	n/a

The final bridge design meets all strength and serviceability requirements. Strength results can be found in Table 16, which indicates that the axial and flexural strengths of the members exceed the applied axial and flexural loads. Serviceability results are listed in Table 17. The first serviceability requirement was the slenderness ratio which is based on the effective member length, total span length, and radius of gyration. For main members, the calculated slenderness was 87.73 which was less than the maximum value of 120. Top chord resistance served as the second requirement and was defined as the nominal compressive resistance, P_n, with an applied factor of safety of 0.90 per AASHTO specifications. The factored P_n value was checked against the largest generated compressive load in the structure which was 214.4 kips. Deflections were checked to ensure that they were below the L/360 margin of 4.33 inches. Per AASHTO specifications, vertical deflections were analyzed under an unfactored pedestrian live load and horizontal deflections were analyzed under an unfactored wind load. The maximum generated vertical and horizontal deflections were 1.79 and 2.32 inches respectively, which were under the limit. The last serviceability requirement was vibrations. Based on the Eigensolution analysis within RISA-3D, the vertical direction generated a frequency of 9.97 Hertz which exceeded the minimum requirement of 3 Hertz. The horizontal direction generated a frequency 2.21 Hertz which was greater than 1.3 Hertz.

Table 17: Serviceability Requirements

	Serviceability				
	Slenderness Ratio	Vertical Vibrations	Horizontal Vibrations	Vertical Deflections	Horizontal Deflections
Results	87.73	9.97 Hz	2.21 Hz	1.79 in	2.32 in

The bridge's supporting abutments were designed in accordance with applicable sections from *AASHTO LRFD Bridge Design Specifications*. AASHTO derives its concrete specifications from publications by the American Concrete Institute and reflects the most updated design codes for concrete structures. Figure 13 shows the final abutment design. Table 18 contains the results from the design calculations.

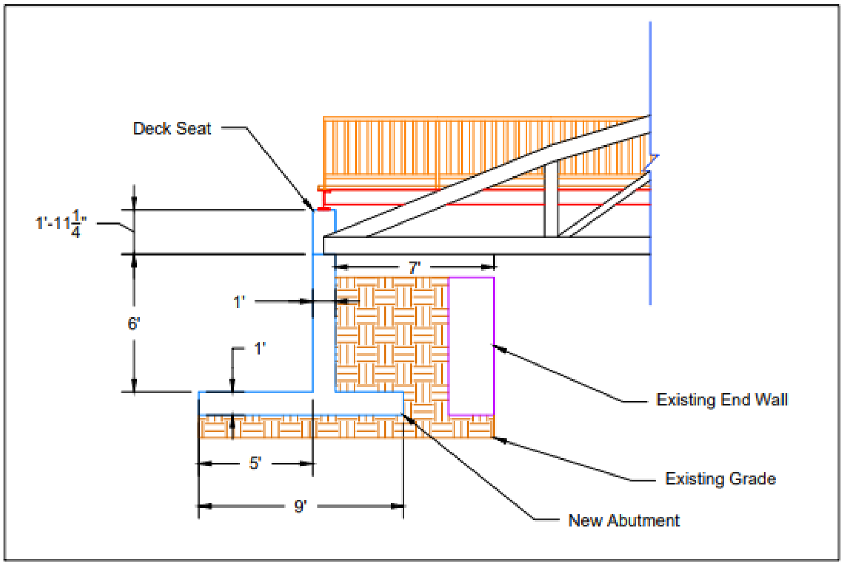

Figure 13: Abutment Details

The design had to meet three major criteria: overturning, bearing, and sliding. The soil on which the abutment rests is composed of silty sand. As such, the allowable bearing capacity of the soil is extremely low and required a relatively large footing to transfer vertical loads. First, Microsoft Excel spreadsheets were created to determine the footing dimensions and hand calculations were used to cross check the results for accuracy. A variety of dimensions and loading combinations were checked until acceptable dimensions were found. The final footing design does not require the addition of supporting piles. The walls were designed to have sufficient resistance against overturning only when the dead load is present and also when the combined dead and live loads are present on the bridge. Given the large surface area of the footing, the resistance to sliding was easily satisfied. *AASHTO LRFD Bridge Design*

Specifications provides specific requirements for the design of concrete reinforcement against internal shear and torsional resistance.

Table 18: Abutment Calculation Results

Design Criteria	Primary Equation	Factor of Safety
Moment resistance	Restoring moment (M_r) > Overturning moment (M_o)	$M_r/M_o = 9.26$
Bearing capacity	Allowable bearing pressure (Q_{allow}) > Maximum bearing pressure (Q_{max})	$Q_{allow}/Q_{max} = 1.09$
Passive resistance	Resistance force (P)>Sliding Force(P2)	P/P2 = 2.90

Lastly, renderings of the final bridge and abutment designs were generated through RISA-3D and AutoCAD. The wood decking was also included in the renderings. Figure 14 shows the final design in an isometric view followed by AutoCAD renderings in Figures 15 and 16.

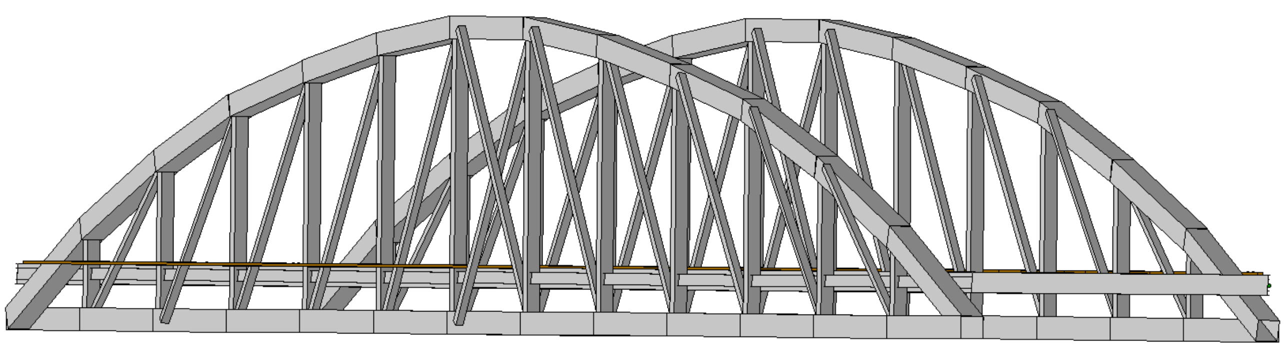

Figure 14: Isometric View of the Final Bridge Design

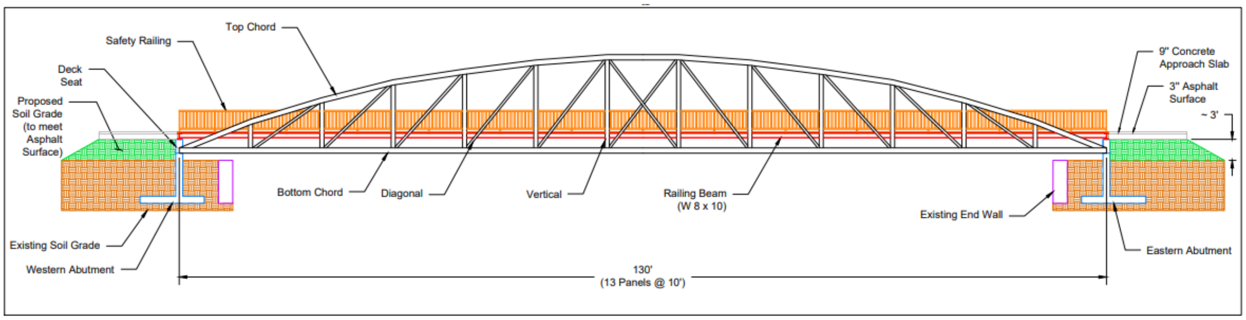

Figure 15: Elevation View of Bridge and Abutments

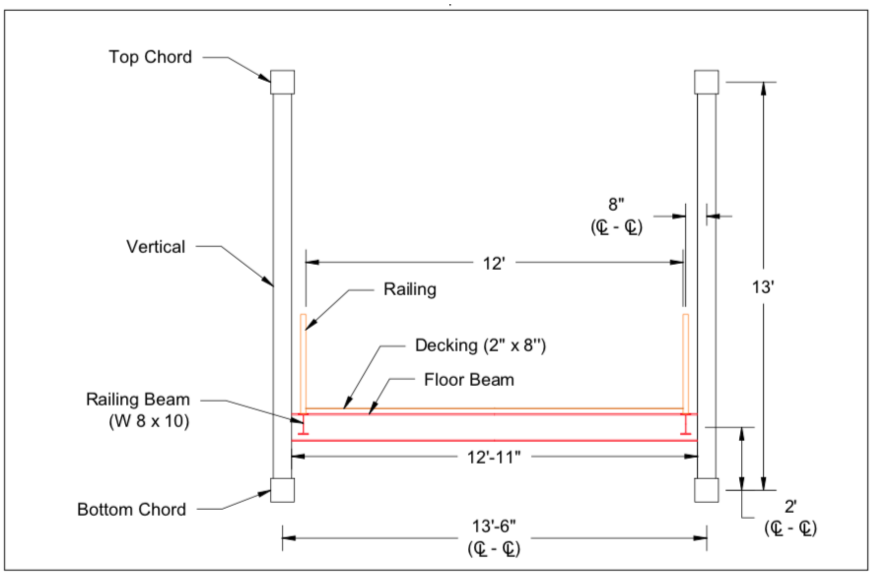

Figure 16: Cross Section View of Bridge

5.2 Partial Demolition Plan

The demolition plan focused on reducing the spread of hazardous material during demolition and reducing the impact on the surrounding environment. Before demolition, necessary site fencing will be established as shown in Figure 17. Fencing and signage will also be placed to the west, where the Northern Strand Community Trail (NSCT) intersects Central Street. There are two potential options for the contractor to access the site. First, ingress can be achieved from the east side off of Lincoln Avenue. The existing corridor access gate could be removed to allow equipment into the right-of-way. The second option is to use Rhodes Street as the access point. There are a pair of Jersey barriers between the street and the NSCT that would have to be relocated. More importantly, Rhodes Street is unaccepted by the Town of Saugus meaning that it is privately owned by the residents on the street. In order to use this road for site access, the residents would have to either give consent or be financially compensated for the use of the street. Logistically, this makes Rhodes Street more complicated to acquire for site access. However, Lincoln Avenue experiences much higher traffic volumes, which is a consideration during equipment mobilization and material deliveries, especially for the prefabricated bridge sections.

Once site access is established, equipment will need to be staged for future construction activities. As mentioned in Section “4.2.2 Existing Environmental Conditions,” the most suitable location for construction staging -- the Bacon property -- is contaminated with hazardous materials and requires soil testing to determine the exact location where equipment and material can be staged. As such, the staging area in Figure 17 may not represent most ideal location.

Figure 17: Site Logistics Plan

The first task of demolition is to install debris netting underneath the bridge deck to keep material from falling into the river. Once this is in place, crews can sawcut the bents along the bridge length between the pipeline and walking portion of the bridge. Perpendicular cuts can then be made to divide the bridge into sections for hoisting as seen in Figure 18. Before the bridge can be lifted, it will have to be separated from the piles. Cutting the bents from the piles can be done using a properly sized Snorkel ™ crane or Genie ™ lift, that are equipped with an articulating boom and extendable aerial work platform (AWP). By using an AWP, there will be no need for scaffolding. Hoisting can be accomplished by a truck mounted hydraulic crane, equipped with straps which can be secured to the pile caps. The bridge section can then be lifted and placed in the staging area for disassembly. This process can be repeated for each section of the bridge. By leaving the bridge as intact as possible before hoisting, it will reduce the amount of material falling into the river. This is a critical consideration due to the hazardous nature of the creosote treated wood.

In addition to the deck, there are fifty timber piles supporting the superstructure that will have to be addressed. The complete removal of these members from the riverbed is unlikely; therefore, the team has considered two alternative options. The first alternative involves cutting the piles at the pile caps, reducing them to the necessary height to allow for installation of the new bridge. This option would be relatively inexpensive; however, leaving the existing piles protruding from the river is not aesthetically pleasing. The second alternative is to cut the piles below the mudline and leave the remaining section underneath the riverbed. However, the cost and schedule implications of this process are significant. For this reason, two schedules and cost estimates have been developed for each demolition plan.

Figure 18: Sawcut Plan

5.3 Construction Plan

After the demolition phase is complete, the construction phase can begin. New abutments will be excavated and poured. Backfill will be a gravel base to improve drainage behind the concrete. The excavated soil will then be placed at the approaches of the bridge to transition the existing grade to the new deck height. This will eliminate the need to relocate the contaminated, excavated soil which would need to be disposed of at an appropriate location in compliance with Section 310 CMR 40.0032 of the Massachusetts Contingency Plan. After the concrete has sufficiently cured, the bridge can be prepared for installation. The prefabricated sections will be delivered to the site where they can be assembled and placed on rollers at the designated location shown in Figure 17. Once on rollers, the fully assembled bridge will be pushed via front loader to the edge of the east abutment. Once in place, a tandem pick will occur between the truck mounted crane and crawler crane. An example of this procedure was provided by Stantec and can be seen in Appendix E. However, an amendment will be made to this procedure: due to the uncertain stability of the existing east abutment, the bridge will be fully hoisted once it reaches the new abutment, which will be set back approximately seven feet from the existing abutment. Since the bridge will be picked prior to reaching the end of the embankment, no section of the bridge can cantilever during erection that may result in tension in the top chords and compression in the bottom chords. This will also avoid a potential collapse of the existing abutment due to the large surcharge from the bridge being rolled into position. The cranes will place the bridge on the new abutments where crews can install anchor bolts and transitions to the deck.

5.4 Cost Estimates and Schedules

The following cost estimates reflect the structural elements of the project and do not include specific items such as mobilization and environmental mitigation. The estimate as seen in Table 19 and full project schedule in Appendix L reflects the costs and time associated with partial demolition alternative A, where the existing timber piles will be left in place. The only difference between this information and that found in Table 20 and Appendix M, is that the demolition includes the removal of the piles.

Table 19: Cost Estimate for Replacement A

Item No.	Discription	Units	Unit Price	QTY	Cost
115.1	DEMOLITION OF BRIDGE NO. S-05-025				
995.02	BRIDGE STRUCTURE NO. S-05-025				
	BREAK DOWN FOR ITEM 995.02				
	BRIDGE STRUCTURE NO. S-05-025				
	PREFABRICATED TRUSS BRIDGE	LS	$ 418,145.20	1	$ 420,000
910.1	STEEL REINFORCEMENT FOR STRUCTURES - EPOXY COATED	LB	$ 4.00	2600	$ 11,000
903	3000 PSI, 1.5 INCH, 565 CEMENT CONCRETE	CY	$ 440.00	25	$ 11,000
121.1	UNCLASSIFIED EXCAVATION	CY	$ 38.00	130	$ 5,000
151.1	GRAVEL BORROW FOR BACKFILLING STRUCTURES	CY	$ 45.00	120	$ 5,400
	BREAK DOWN FOR ITEM 115.1				
	DEMOLITION OF BRIDGE NO. S-05-025				
181.14	DISPOSAL OF HAZARDOUS WASTE	TON	$ 400.00	70	$ 28,000
	REMOVAL OF EXISTING BRIDGE	LS	$ 31,100.00	1	$ 32,000
				TOTAL	$ 512,400

Table 20: Cost Estimate for Replacement B

Item No.	Discription	Units	Unit Price	QTY	Cost
115.1	DEMOLITION OF BRIDGE NO. S-05-025				
995.02	BRIDGE STRUCTURE NO. S-05-025				
	BREAK DOWN FOR ITEM 995.02				
	BRIDGE STRUCTURE NO. S-05-025				
	PREFABRICATED TRUSS BRIDGE	LS	$ 418,145.20	1	$ 420,000
910.1	STEEL REINFORCEMENT FOR STRUCTURES - EPOXY COATED	LB	$ 4.00	2600	$ 11,000
903	3000 PSI, 1.5 INCH, 565 CEMENT CONCRETE	CY	$ 440.00	25	$ 11,000
121.1	UNCLASSIFIED EXCAVATION	CY	$ 38.00	130	$ 5,000
151.1	GRAVEL BORROW FOR BACKFILLING STRUCTURES	CY	$ 45.00	120	$ 5,400
	BREAK DOWN FOR ITEM 115.1				
	DEMOLITION OF BRIDGE NO. S-05-025				
181.14	DISPOSAL OF HAZARDOUS WASTE	TON	$ 400.00	70	$ 28,000
	REMOVAL OF EXISTING BRIDGE	LS	$ 150,000.00	1	$ 150,000
				TOTAL	$ 630,400

Several sources were used for the cost data used in these estimates. First, input was received from Stantec professionals in variety of departments including structural, environmental, and waterfront. Another source was the database for MassDOT weighted bid items (MassDOT, 2019). This provided insight into standard prices for basic construction items such as concrete and excavation. Finally, the *RSMeans Heavy Construction Data Book* was used to check daily outputs for line items and generate a schedule for specific activities (Hale, 2015). The estimates in Table 19 and Table 20, were derived from a quantitative analysis of the required material, which can be seen in Appendix N. Each item was estimated for exact quantities, and additional material was included in case additional material was required on site. For lump sum items, “Prefabricated Bridge,” and “Removal of Existing Structure” a 20% contingency was assigned to the entire activity due to the high risk of delays and unforeseen conditions.

5.5 Final Recommendations

If Stantec continues to pursue with the bridge design along with the demolition and construction plan presented in this report, the team offers the following three recommendations:

1) Design HSS connections in accordance to "Appendix K: Additional Requirements for HSS and Box-Section Connections" Tables K3.2, K4.2, and K5.1 of the *AISC Steel Construction Manual, 15th Edition*;
2) Design reinforcement for the deck seat;
3) Update abutment design with Stantec's full geotechnical report upon availability; and
4) Reuse soil from abutment excavation as fill for the proposed grade on bridge approaches.

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Appendices

Appendix A: Project Proposal

Structural Improvement Design for Northern Strand Community Trail Bridge over Saugus River

A Major Qualifying Project Proposal Submitted to the Faculty of
Worcester Polytechnic Institute
In Partial Fulfillment of the Requirements for the Bachelor of Science Degree
By

Sean Burke
Marlies de Jong
Ben Leveillee

December 14, 2018

Advisors:
Professor Suzanne LePage
Professor Leonard Albano

Sponsor's Representative:
Jennifer Ducey
Senior Associate, Stantec

Capstone Design Statement

This Major Qualifying Project (MQP) will require the project team to develop preliminary design alternatives and improvements for a decommissioned railroad bridge. The bridge is currently part of the Northern Strand Community Trail (NSCT) in Saugus, Massachusetts that serves pedestrians and bicyclists. In order to determine that the project meets design requirements to best accommodate the surrounding environment and also meet stakeholders' needs, the team will incorporate the following design constraints: health and safety, economical, environmental, aesthetics and constructability, and ethics. By addressing these design constraints, the design project will meet the requirements for a Capstone Design Experience, as outlined by the Accreditation Board of Engineering and Technology (ABET).

Health and Safety

Safety is the primary design factor for this project. Any recommendations for improving the pedestrian bridge must retain its structural integrity and serviceability requirements during all required load scenarios defined by the AASHTO Guide Specification for the Design of Pedestrian Bridges. The work breakdown structure will be developed in accordance with responsible construction phasing. Any additional recommendations will maintain the safety of construction workers during the construction process as well as pedestrians and cyclists once the bridge is functional.

Economical

The final design for this improvement project will consider material and labor costs based on statewide weighted bid average prices provided by the Massachusetts Department of

Transportation. This project will recommend a final design that is cost-effective without sacrificing structural functionality or material quality.

Environmental

The pedestrian bridge over the Saugus River is surrounded by wetlands. This project site poses significant environmental design criteria because not only are wetlands sensitive natural environments, but they are also protected by the Massachusetts Wetlands Protection Act. With the bridge in close proximity to protected land, environmental concerns, such as stormwater runoff and destruction of natural habitat, are magnified. It is vital that preventative measures be taken to mitigate these risks. Because the scope of work is within wetlands or within a 100-foot buffer area around the wetlands, the Town of Saugus Conservation Commission will be closely monitoring any development. In order for this improvement to have a realistic chance of approval by the commission, the final design and construction recommendations must minimize these impacts on the surrounding environment.

Aesthetics & Constructability

In addition to being surrounded by wetlands to the West, the bridge is located in close proximity to residential properties to the North and East. This project will have to consider the aesthetics of the bridge and any structural improvements to it, due to the high visibility of the bridge. A redesign that is both aesthetically pleasing and fast to construct will minimize the visual disruption to the natural landscape.

Ethics

While developing the final design and recommendations, the team will follow the eight canons established by the American Society of Civil Engineer’s (ASCE) Code of Ethics. Research will consider the variety of perspectives surrounding this project to ensure that final design and recommendations provide a holistic solution to this problem. Design decisions will hold “safety paramount” and consider the surrounding communities’ environmental and social diversity (ASCE, 2017).

1.0 Introduction

The Northern Strand Community Trail (NSCT) is a 10.5-mile trail corridor located North of Boston, Massachusetts. As shown in Figure 1, this trail begins at the Mystic River in Everett and ends in Lynn, connecting the urban environments of Malden and Everett to the beaches of the North Shore. The NSCT is completely separate from roadways and vehicles, and therefore is a safe walking and cycling environment for residents of the greater Boston area. The trail is part of a larger initiative, known as the East Coast Greenway, to build a 3,000-mile protected biking and walking route from Maine to Florida.

Figure 1: Map of the Northern Strand Community Trail (Bike to the Sea, Inc.)

The communities surrounding the trail are looking to protect the safety of the cyclists and pedestrians that use it. However, the nine-span, 115-foot, bridge over the Saugus River has been proven to be a hazard to trail goers. Preliminary timber pile sampling and testing results indicate that the bridge is not structurally sound for continued pedestrian use (WAS, 2018).

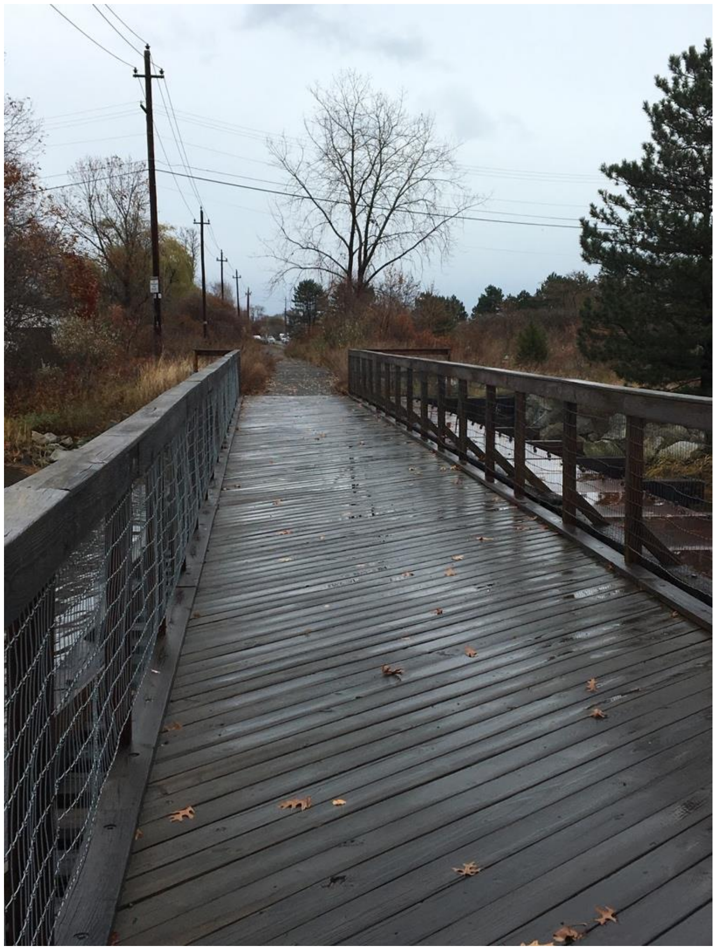

Figure 2: Saugus River Pedestrian Bridge

The goal of the project is to assess the condition of the bridge and develop potential solutions that address the failing structure. The following objectives will guide the team's process for the project:

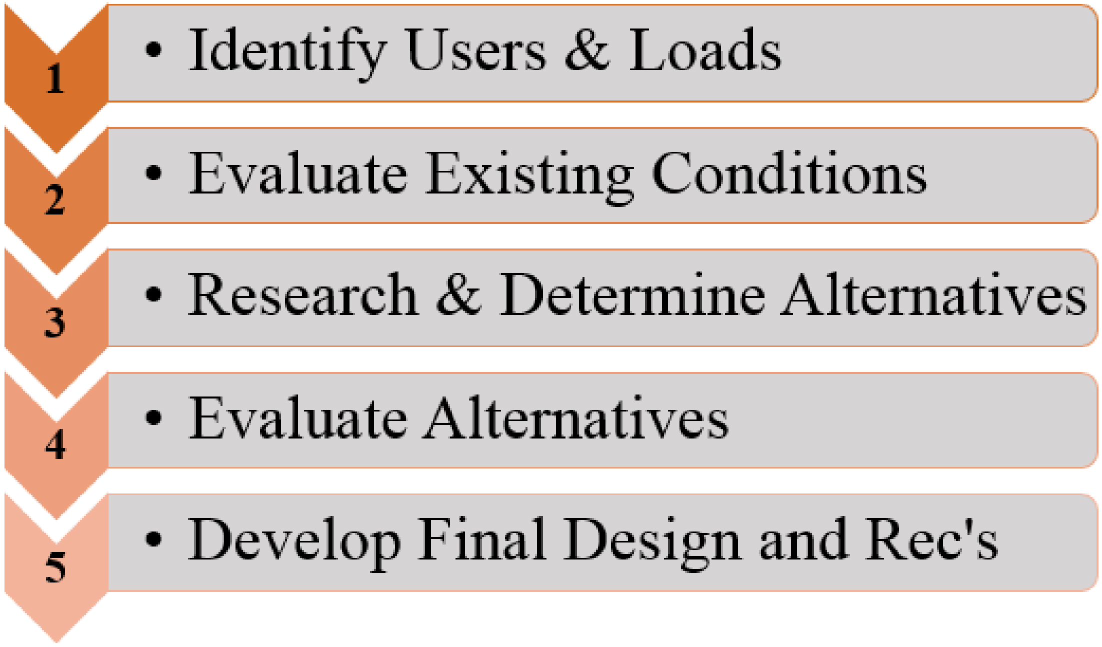

Figure 3: The Five Project Objectives

The first objective to achieving this goal is to identify bridge users and associated loading requirements. Then the team will combine observations from site visits and the findings from the Wood Advisory Service (WAS) to develop a complete understanding of the bridge's existing conditions. As part of the third objective, the team will develop a list of rehabilitation and replacement options with different bridge designs, fabrication methods, and materials, that comply with Massachusetts building codes. The fourth objective will involve an evaluation of the rehabilitation and replacement options. The most effective design will be one that addresses the health and safety, economic, constructability, environmental, and ethical constraints defined for this project. Ultimately, the last objective will focus on delivering a final recommendation for the bridge's design alternative. These recommendations will include structural design and models, a work breakdown structure, and an approximate cost estimate.

2.0 Background

This section will give an overview the history of the NSCT including the surrounding land and its classifications. Existing conditions of the pedestrian bridge over the Saugus River will be reviewed, followed by a discussion on options for design improvements.

2.1 Trail History

Before the NSCT became a well-known bike path, it was the Saugus Branch commuter railroad, servicing residents of Saugus, Malden, Everett, Revere, and Lynn. The Eastern Railroad Company began construction on the line in 1850 and serviced its first customers three years later. Usage of the railroad steadily increased to its peak in the 1890s, when the Saugus Branch offered 36 trips into Boston per day. After the construction of the Boston Elevated Railway in the early 1900s, the demand for the Saugus Branch gradually declined until its last passenger trip in 1958. Once commuter service was discontinued, it was converted into a freight line until 1993, after which the railroad was abandoned (Revolvy, 2018).

Bike to the Sea's involvement began in 1993 when the freight line was discontinued. Bike to the Sea is a local social advocacy group that promotes the use and maintenance of the NSCT. This group organizes community activities, fundraisers, and general awareness of the trail. Nearly all of the original railroad material has been removed to make the path more suitable for pedestrian use. Sections in Everett and Malden have been also paved to improve conditions for bikers. Since the transformation from rail to trail, the local community has embraced the new recreational path as an alternative means of commuting. The close residents are not the only ones to show interest in the NSCT. The trail has gained recognition by the state government, recently being awarded $1.5 million to fund redesigning the 10.5-mile path (Mass.gov, 2018). The 2.5-mile section through Saugus is one of the unpaved portions, with a beautiful ride along the Saugus River. This part of the trail has a bridge that passes over the river which serviced freight trains during the later years of the railroad's life.

In 2002, the MBTA made an evaluation of the Saugus Branch, with consideration of recommissioning it for various forms of use. Options for alternative use include converting the railroad into a truck haul route for Logan Airport or potentially an MBTA Urban Ring bus route. A third option would have been to run a rapid transit line for commuter service. These options have not been pursued due to restrictions created by the current surrounding environment. The right-of-way is now constricted by numerous crossings and abutting residential properties, making the prospect of recommissioning unlikely (PB/Harris, 2002).

2.2 Current Land, Land Use and Zoning

Saugus is currently divided into twelve designated zoning districts as mandated by the town's zoning by-laws updated in 1997 (Ortiz, 1997). The bridge is located on open space zoned land and is surrounded by zoned single-family residential dwellings (R1) to the North and East direction and industrial zones (I2 – heavy) in the West and South directions. After the group conducted a site visit, they found that the bridge is surrounded by residential units and undeveloped land with no signs to indicate any construction in the near future.

A third of Saugus' land is classified as tax exempt. Tax exempt land is typically publicly owned by a government entity. Parks and open spaces, such as the NSCT are among the most common land uses to fall under this classification. Within tax exempt land use, there is a right-of-way easement which is granted over land being used for transportation services or public utilities (MAPC, 2018). About 14% of Saugus' total land area is classified as a right-of-way which is primarily comprised of roads, curbs, and sidewalks (MAPC, 2018). Another notable right-of-way is the NSCT. Since the bridge is a former railroad track turned pedestrian-bicycle path and also contains a large utility line, it meets the classifications for a right-of-way land use.

The Saugus River is approximately 13 miles long and eventually leads to the Atlantic Ocean, and drains a watershed with an area of 47 square miles (Saugus). The team developed the map in Figure 4 based on Massachusetts Department of Environmental Protection and Geographical Information System data layers, and it shows the salt marshes and tidal flats that surround the bridge. These soils are often composed of deep mud and peat, which is decomposing plant matter (US EPA, 2015). As a result, the water flowing under the bridge is brackish and the area is subjected to tidal flooding on a consistent basis. This combination of land classifications poses some unique environmental concerns for development on or around the bridge.

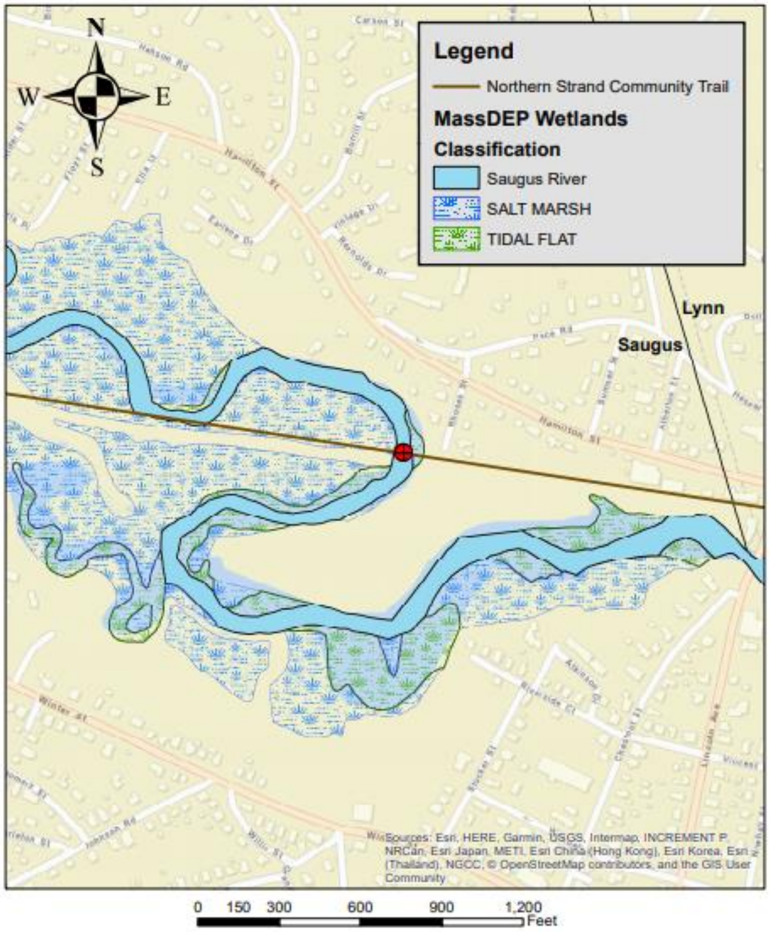

Figure 4: Map of Saugus River and Bridge

2.3 Existing Bridge Conditions

The NSCT pedestrian bridge that passes over the Saugus River is the focus of this project because it has been shown to be structurally compromised. An analysis of the bridge done by the Wood Advisory Service (WAS) evaluated the integrity of the timber piles supporting the structure. The scope of this test included both visual and lab testing of 20 core samples taken from the timber. The visual inspection of the bridge revealed numerous locations where significant deterioration of the wood led the inspectors to believe the service life of the bridge is nearing its end. Compromised members include cross bracing, pile caps, and the end wall.

Additionally, resistographs were created for 70% of the existing timber piles to quantify the amount of deterioration and assess the remaining effective cross sectional area. The results from this testing were extremely variable across the different readings, with an effective cross section ranging between 17% and 92% (Table 1).

Figure 5: NSCT Bridge over the Saugus River

Table 1: Timber Pile Integrity Data from Wood Advisory Services

PILE NO. =>	1		2		3		4		5		6		7		8		9		10		11	
	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'	ϕ	A'
BENT NO.	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)	(in)	(%)
West Abutment																						
1	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*
2	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*
3	14	65	14	62	12	84	15	87	12	34	15	83	14	62	12	92	13	85	14	80	14	83
4	14	33	13	83	14	51	14	33	13	57	14	86	14	86	12	17	13	83	14	86	14	84
5	13	48	14	86	14	85	15	85	12	38	15	44	14	73	15	54	13	57	14	80	14	51
6	14	18	14	86	15	44	14	85	14	46	14	73	14	62	15	68	13	25	14	56	14	62
7	14	72	14	73	14	73	15	87	14	41	14	60	14	62	15	87	15	48	15	64	15	75
8	14	51	12	69	14	33	15	44	14	49	15	44	16	56	14	86	15	44	15	44	13	48
9	*	*	*	*	*	*	*	*	*	*	15	44	14	73	15	64	15	87	16	77	14	86
10	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	14	25	15	62	*	*
East Abutment																						

NOTE:

*	No data recorded
ϕ	Diameter of pile @ breast height (in.)
A'	Estimated effective remaining cross sectional area (% of calculated cross-section from diameter)
	00% to 25% Estimated residual cross section (%)
	26% to 50% Estimated residual cross section (%)
	51% to 75% Estimated residual cross section (%)
	76% to 100% Estimated residual cross section (%)

The final data collected was the creosote retention. This is a measurement of the creosote treatment that is applied to the timber to protect it from weathering and extend the service life. Recommended creosote retention levels range from 16 pcf to a minimum of 8 pcf. The measured value of the timber piles was 7.60 pcf. The combination of timber deterioration and lack of creosote limits the potential service life of the substructure. The presence of these conditions will be a major consideration as the team examines improvements to the bridge.

2.4 Bridge Components

Prior to analyzing different design options, the fundamentals of the existing bridge's members were researched. A strong understanding of a bridge's structural components is vital for an effective design. There are two main categories to a bridge: the substructure and superstructure. The substructure is comprised of the foundation, wing walls, abutments, and piers. Foundations evenly transmit the loads generated by everything above it into the strata beneath it, while wing walls are an extension of the abutments to retain any strata present. Abutments retain the earth behind the structure while supporting dead and live loads from the superstructure. Piers are primarily designed to transmit the loads to the foundations and to resist horizontal forces. The superstructure is comprised of all the other bridge components that are placed above the substructure. The decking and girders all help transmit generated live loads into the substructure (Bridge Masters, 2017).

2.5 Design Alternative: Aesthetic Improvements

There are a plenitude of options that can be taken to improve the NSCT bridge. The first option is to leave the bridge in its current condition without any structural modifications. However, if the bridge is left as is, rehabilitation efforts may be needed in the future to address any deficiencies. In order to maintain the bridge in its current state, the design needs to be analyzed to determine that it is adequate to provide service for required loadings and deflections. The substructure would especially be a key area of focus in analysis since the inspection report suggested that the bridge may be at the end of its service life, due to deterioration of the timber piles. If the bridge can safely support its self-weight and expected live loads, then simple minor aesthetic improvements can be designed and the core of the structure can be left alone. Adding lighting, removing graffiti, and replacing the decking or railing would all be viable design options to proceed with. Modifications of the trail leading up to the bridge could also be incorporated into the design such as repaving the path, landscape planning, and adding benches or bike racks.

2.6 Design Alternative: Rehabilitation

If the bridge does not meet safety and serviceability requirements, then rehabilitation designs need to be considered. Additionally, the NSCT bridge contains a 24-inch utility gas line that is of concern. Design and construction would need to be mindful of the gas line and ensure that proper measures were taken to preserve the utilities. There are also electrical power lines running along the bridge which are less cause of concern yet still need to be addressed. Traffic maintenance and control costs will also need to be incorporated into rehabilitation design. The bridge may also have large historical significance to Saugus which would require a mindful approach during rehabilitation to ensure that the historical integrity of the structure is not compromised.

After reviewing case studies, there are several different rehabilitation options that could be pursued depending upon structural analysis results. New girders, joint replacement, deck strengthening, parapets replacement, abutment modifications, and pile upgrades are just among the few considerations that could be incorporated into rehabilitation design.

2.6.1 Historic Bridge Rehabilitation

The National Historic Preservation Act of 1966 and Section 4(f) of the U.S. Department of Transportation Act of 1966 outlined a national standard for determining if a historic bridge is to be rehabilitated or replaced. However, no national corresponding protocol was established for the decision process. In 2007 at the request of the American Association of State Highway and Transportation Officials (AASHTO), the National Cooperative Highway Research Program (NCHRP) released a report called "Guidelines for Historic Bridge Rehabilitation and Replacement". It provides a protocol for making decisions in the design process of historic bridge rehabilitation to bring them into conformance with current design and safety standards. The guidelines were developed based on case studies of effective practices with consideration to environmental issues (NCHRP, 2007).

The general approach is divided into 4 steps (NCHRP, 2007):

1. Understanding what constitutes as a historic bridge. This provides an understanding of the historical importance of the bridge and its related elements.
2. Applying structural analysis and determining bridge’s function. This iteration allows for a balance when addressing any structural and functional deficiencies along with historical and environmental issues.
3. Determining historical and environmental considerations. This addresses any issues that were not outlined in the previous step.
4. Applying decision making thresholds from information gathered from all previous steps to support whether rehabilitation is necessary and the best option.

2.6.2 Case Studies

A series of 16 historic bridge projects made from stone, metal, or concrete was compiled by the SRI Foundation in 2011. The projects were completed in conjunction with the NCHRP’s Guidelines to help consider rehabilitation or replacement. Although arch and truss bridges comprised a majority (14) of the 16 studied, they provided valuable insight on how to proceed with historic bridge rehabilitation (Table 2). There was also a movable span and metal girder bridge studied in the analysis.

Table 2: Historic Bridges Studied

Type of Bridge	Name of Bridge	Location	State
Stone arch	Johns Burnt Mill Bridge	Mount Pleasant and Oxford Townships, PA	PA
Stone arch	Prairie River Bridge	Merrill, WI	WI
Concrete arch	Carrollton Bridge	Wabash River, IN	IN
Concrete arch	Robert A. Booth (Winchester) Bridge	Douglas County, OR	OR
Movable span	Bridge of Lions	St. Augustine, FL	FL
Metal truss	Tobias Bridge	Jefferson County, IN	IN
Metal truss	New Casselman River Bridge	Grantsville, MD	MD
Metal truss	Walnut Street Bridge	Mazeppa, MN	MN
Metal truss	Pine Creek Bridge	Borough of Jersey Shore, PA	PA
Metal truss	Washington Avenue Bridge	Waco, TX	TX
Metal truss	Lone Wolf Bridge	San Angelo, TX	TX
Metal truss	Goshen Historic Truss Bridge	Goshen, VA	VA
Metal truss	Hawthorne Street Bridge	Covington, VA	VA
Metal truss	Ross Booth Memorial Bridge aka Winfield Toll Bridge	Putnam County, WV	WV
Metal arch	Lion Bridges	Milwaukee, WI	WI
Metal girder	Hare's Hill Road Bridge	Chester County, PA	PA

The review of the bridges in Table 2 revealed that rehabilitation was the more expensive option compared to replacement due to the bridge's age and level of deterioration. Cost effective techniques can be utilized to minimize high expenses such as using bolts instead of rivets and using welding to repair cracks in plates. However, rehabilitation was chosen due to each bridge's historical significance. In order to determine the bridge's significance to the community, having historical preservation experts involved in all stages of the rehabilitation process provides valuable guidance (SRI Foundation, 2011).

Additionally, early implementation and coordination with all stakeholders and resource agencies, such as the town of Saugus and the Saugus Rivershed Water Council, are essential to avoid disagreements regarding the project's decision-making process. Ultimately, actively engaging the community allows for the designer to obtain their support for the project.

The rehabilitation needs to ensure that the historical integrity of the bridge is maintained. For example, retaining the design of the piers and girders may be an extremely important aspect in order to maintain the historic character. This requires careful modification of current standard bridge elements so that they appear to match original bridge components.

If a bridge was built before standard specifications were issued, which is common with bridges built in the early-twentieth century, samples from beam and pile members need to be tested to determine data on the material’s strength (SRI Foundation, 2011). Sample results will determine if structural components need to be replaced or simply upgraded. Furthermore, methods to reduce the bridge's self-weight, such as reinforced deck systems, allow for a raised live load capacity to increase the bridge’s usability (SRI Foundation, 2011).

All the case studies highlighted that rehabilitation on a project will require constant inspection, analysis, and design to address unforeseeable conditions that may not be detectable until the construction process begins. Additionally, if there was little predicted increase in traffic flow, the bridge had undergone rehabilitation as opposed to replacement (SRI Foundation, 2011).

2.7 Design Alternative: Replacement

If neither aesthetic improvements nor rehabilitation are feasible options, the focus shifts to complete replacement. Common replacement structures include simple beam or girder, arch, truss, cantilever, suspension, and cable-stayed bridges. To narrow the scope of replacement options, the team conducted a review of 16 pedestrian bridge case studies to identify the most optimal replacement structure.

2.7.1 Replacement: Case Study Review

The study began with a review of eleven completed pedestrian bridges built by Contech Engineered Solutions. Contech Engineered Solutions is a nationwide engineering consulting firm that specializes in prefabricated, custom pedestrian bridges. Of the eleven bridges that were reviewed, eight were truss bridges and the remaining three were arch bridges.

Additionally, the study analyzed three university reports, each detailing a pedestrian bridge design. A critical step in each report was exploring three to four possible bridge types, and all three reports considered beam-and-girder bridges as shown in Table 3. While none of the reports selected the simple beam or girder option for final design, it was a consideration for all the reports therefore illustrating its applicability to a range of pedestrian bridge projects.

Based on all 11 case studies, it was evident that the most common pedestrian bridges are beam-and-girder, truss, and arch. A variety of factors are considered prior to selecting one of these three designs, and these factors are discussed in the following section.

Table 3: Bridge Type Considerations from University Reports

Report #1 (Adams & Gould, 2014)	**Report #2 (DeCelle et al., 2013)**	**Report #3 (Raskett & Rebello, 2017)**
Simple Girder	Simply-Supported Beam	Simple Girder
Aluminum Truss	***Truss****	***Arch****
Whipple Truss*	Arch	Cable-Stayed
Flatcar	Cable-Stayed	N/A
Note: *** denotes that option was selected for final design		

2.7.2 Replacement: Design Considerations

There are five factors that must first be considered in order to select the most suitable structure for a given bridge replacement project. A bridge's load path is the foremost important consideration, as it defines the manner in which load is transferred through connected members. It is obvious that a bridge must be capable of successfully transferring loads from its deck to the foundation, otherwise it is structurally deficient. The specific direction and members that the load passes through has implications on member sizes, connection details, and foundation design. For example, slender members subjected to compression must have sufficient compressive strength as well adequate bracing to prevent buckling whereas members subject to tension solely need sufficient tensile strength (McCormac & Csernak, 2018).

A logical second consideration is foundation design. A bridge is only as strong as the soil supporting it. As a result, foundations must distribute the bearing pressures in a manner that does not exceed the soil's bearing capacity. Additionally, foundations must secure the bridge to the supporting earth. For bridges that span bodies of water, foundations must extend far enough into the soil to safely guarantee prevention against underscour by the movement of water (Fadum).

The third and fourth considerations for replacement bridge design are span length and vertical clearance. The clear distance that a bridge can reasonably span without intermediate support structures is the determined by design and material properties. The FHWA estimates that for highway bridges constructed of standard steel W-sections, continuous beam bridges have a maximum single span length of 120 feet whereas arch bridges are capable of single spans greater than 150 feet (FHWA, 2015). When the distance that needs to be traversed exceeds the reasonable simple-span limit, intermediate piles must be installed and these supports limit the width of objects that can traverse underneath the bridge. Similar to span length, the vertical clearance between a bridge's superstructure and flowing water below is critical for environmental and recreational factors. Bridges must accommodate for changes in water body height generated by high and low tides as well as flooding and tidal surges. In fact, NOAA's National Storm Surge Hazard Maps estimate that the Saugus River at the location of the pedestrian bridge has the potential to experience a six-foot storm surge for a Category 1 hurricane (NOAA). In terms of recreational reasons, the bridge needs to have adequate vertical clearance to allow for canoers and kayakers to pass underneath the bridge.

The final consideration for replacement bridge design is constructability. Designs that utilize prefabricated segments of typical member sections will be more economical and easier to construct than ones consisting of custom member sections that require on-site fabrication. Transportation and access to the site are two factors that have large implications on constructability. Members and prefabricated sections should be less than 120 feet in length to facilitate easier transportation, and only construction vehicles that can feasibly access the site should be used for construction (FHWA, 2015).

With these five considerations, any design team will be capable of selecting the most suitable replacement bridge type for their given project. But the design considerations are not completed once a bridge type is selected; a new set of considerations appear once the team begins to determine the materials to be used in the bridge design.

2.8 Material Considerations

When considering approaches for bridge rehabilitation and replacement, it is essential to consider the materials as well as the structure type. The structural, economic, and environmental properties of the construction materials all have effects on the structure's performance and cost.

A material's modulus of elasticity, unit weight, and failure mechanisms are examples of notable structural properties that influence a structure's performance under loads. Additionally, a material's cost for manufacturing, transportation, and fabrication factor into the overall economic price of a project. Finally, the resources and technologies that are required to produce such materials place a stress on the environment, thereby creating an environmental cost.

Wood, steel, and concrete are the three most popular materials used in modern bridge construction, and each has unique properties that make it ideal for use as a construction material. Various structural properties of these three materials are summarized in Table 4. Designers must weigh the advantages and disadvantages of these three materials, and select either one or a combination of materials during the course of design.

Table 4: Structural Properties Comparison of Wood, Concrete, and Steel

	Sawn Wood	**Concrete**	**Steel**
Composition	Natural Timber	Composite consisting of Aggregate, Cement, and Water	Engineered Alloy of Iron and Carbon
Modulus of Elasticity	0.8 – 1.9 ksi	N/A; Compression Strength of 4 ksi	29,000 ksi
Unit Weight	36 pcf	150 pcf	490 pcf
Homogeneous?	No; Properties Vary Based on Knots and Grain Orientation	No	Yes
Pros	• Light • Can Be Hand-Cut to Proper Size • Ease of Construction	• Great Compressive Strengths • Opportunity for Reinforced or Prestressed • Limitless Shapes	• High Degree of Confidence • Ductile • Retains Strength After Significant Deformation
Cons	• Deterioration and Swelling • Suffers from Moisture Damage • Relatively Weak	• Limited Tensile Strength • Requires Formwork • Requires Curing Period of 28 Days	• Buckling • Corrosion when Exposed to Moisture and Salt

3.0 Methodology

The team's goal for the project is to assess the condition of the bridge and develop potential solutions that addresses the failing structure. This goal will be achieved through the following phases. The first phase will focus on identifying bridge users and load requirements. The second phase will survey the bridge's existing conditions and research design alternatives. The third phase will evaluate the design alternatives and accordingly develop a final design and recommendation. Ultimately, the team's goal is to deliver an effective solution that meets all governing specifications. A complete project schedule can be found in Section 3.5.

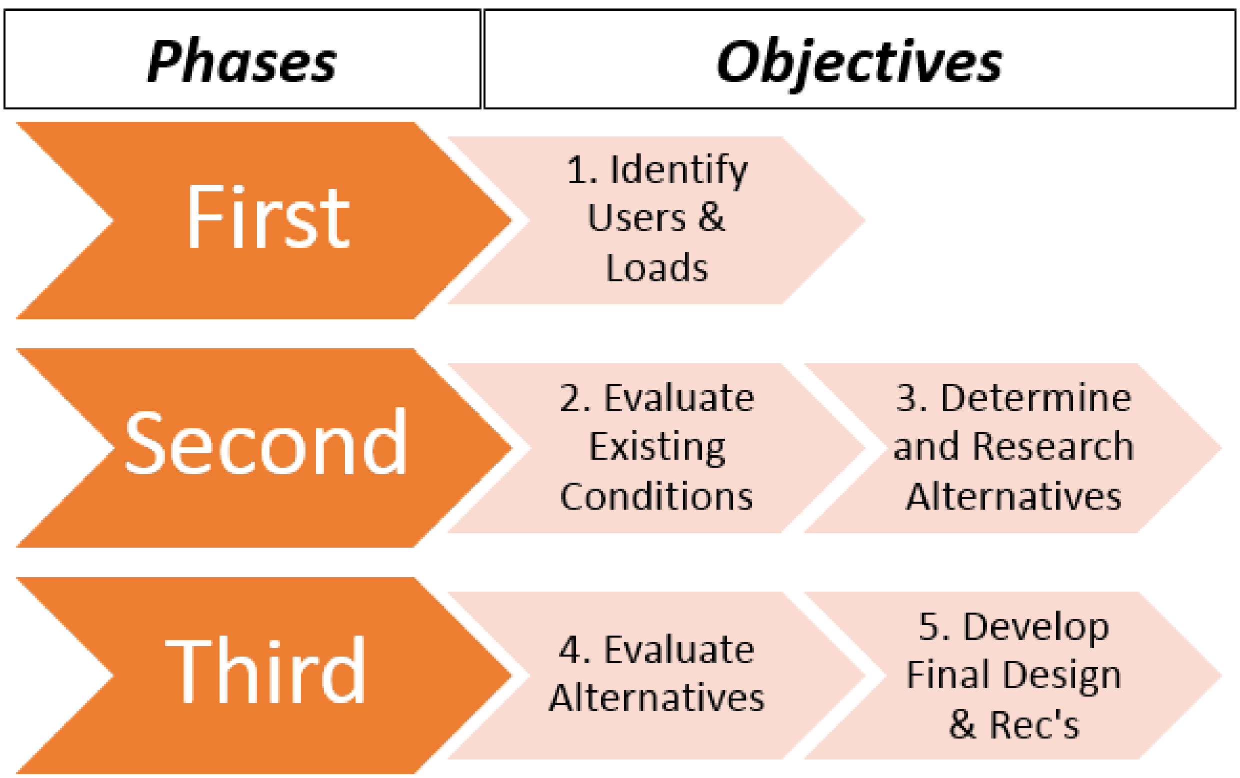

Figure 6: The Three-Phased Approach

3.1 Identify Bridge Users and Load Requirements

The team will first identify the bridge's primary function along with who its main users will be. Other factors such as snow or emergency vehicles need to be considered. After identifying bridge users, associated loadings and loading requirements will be determined in conjunction with Load and Resistance Factor Design (LRFD) and AASHTO bridge codes. Additional external loads such as wind and snow will also be incorporated into the loading requirements. This first step will provide a holistic frame for the team to have a fundamental understanding of the project.

3.2 Evaluate Existing Conditions

The inspection report submitted by Wood Advisory Services (WAS) will be a guiding resource in the team's assessment of the current structure. In order to be deemed suitable for continued use, the bridge must first meet AASHTO service requirements. Using these specifications, the team will determine the required performance of the structure and compare against its current conditions. The effective remaining cross section of the timber piles will be used to adjust current member sizes in calculations to reflect actual performance.

If the structure meets AASHTO requirements, then an evaluation of the remaining service life of the bridge will be made using the WAS report. As determined by WAS, the creosote retention of the timber is significantly below accepted standards. This degraded protection will be used to project the remaining service life of the bridge, before deterioration becomes too significant. If degradation is too rapid, then it may outway the current condition of the bridge. In other words, if the bridge will not be serviceable in several years, then it may be necessary to take corrective action, regardless of current performance.

3.3 Research and Determine Improvement Alternatives

The team will research alternatives within the three following bridge improvement categories and the methods for research are described below.

3.3.1 Aesthetic Improvements

After evaluating the bridge's existing conditions, the team will determine if it is structurally sound and currently meets AASHTO requirements. The substructure will be of particular focus due to the WAS inspection report. If the current design is indeed adequate, the team will move forward to determine aesthetic improvements to the bridge without any major structural modifications. Options as mentioned in "2.5 Design Alternatives: Aesthetic Improvements" will be considered along with additional research that the team will perform. Case studies, professional interviews, and scholarly articles will all be utilized to determine a cost effective improvement design.

3.3.2. Rehabilitation

If aesthetic improvements are not sufficient to meet required performance, then rehabilitation will be evaluated. This alternative will look to strategically enhance the bridge's substructure. Such improvements can be achieved by the removal and replacement of members with excessive deterioration. The structural weaknesses highlighted in the WAS report will be used to identify the members that are most critical. Also, after a full evaluation of the existing conditions is complete, the team will be more informed as to what must be done to achieve AASHTO compliance. Rehabilitation, however, is not always the most cost effective alternative.

Therefore, cost of rehabilitation will be compared with the following design alternative to identify the most suitable option.

3.3.3 Replacement

The research of replacement alternatives will be limited to simply-supported, truss, and arch bridges as these were identified to be the most common replacement structure types for pedestrian bridges as mentioned in “2.7.1 Replacement: Case Study Review”. The team will conduct independent research regarding all five design considerations discussed in “2.7.2 Replacement: Design Considerations” for each of the three replacement types. This research will include academic publications as well as design resources developed by AASHTO and the Federal Highway Administration (FHWA). Following the completion of this independent research, the team will interview design professionals from Stantec regarding the same design considerations from Section 2.7.2. The team’s goal in conducting these interviews is to go beyond the benefits and drawbacks of each replacement type and begin to understand design implications as pointed out by professionals.

3.4 Evaluate Alternatives

The team will begin the third phase by evaluating the feasibility of each project alternative based on the six criteria defined in the Capstone Design Statement. The purpose of this objective is to identify the alternative that best satisfies the following criteria: health and safety, economical, environmental, aesthetics, constructability, and ethicalness.

The team will utilize a decision matrix to facilitate the evaluation process. Each of the six criteria will be a separate category in the matrix, and numerical ratings (ranging from one to five) for each of the alternatives will be placed in these categories depending on the degree to which they satisfy the criterion. Furthermore, the team will assign weightings (ranging from one to three) to each of the criteria due to the fact that some criteria are more critical to the project than others. The team will determine the weightings as well as the ratings based on “Section 3.3 Research and Determine Improvement Alternatives.” Once the decision matrix is completed, team members will sum the weighted ratings for each alternative across all criteria. The team will select the alternative with the highest overall score for design, as this alternative best satisfies the project criteria.

3.5 Milestones

The phases and objectives discussed above have been compiled into a tentative work schedule that the team will follow during the project. Each phase has been designated as a milestone to set concrete deadlines which ensure the completion of work within the specified project duration. Additionally, writing the report will be an ongoing process from January 9, 2019 until the project’s completion date, March 1, 2019. Figure 7 depicts which milestones will be completed and Figure 8 is a Gantt chart that shows the time duration that each phase and objective is estimated to take for completion.

		Project Start	1/9/2019
Milestone Description	Category	Start	No. Days
1.0 First	Milestone		
1.1 Identify Bridge Users	On Track	1/9/2019	3
1.2 Identify Regulations	On Track	1/9/2019	3
2.0 Second	Milestone		
2.1 Evaluate Existing Conditions	On Track	1/14/2019	5
2.2 Improvement Alternatives	On Track	1/14/2019	12
3.0 Third	Milestone		
3.1 Evaluate Alternatives	On Track	1/28/2019	5
3.2 Develop Final Recommendation and Design	On Track	2/4/2019	26
3.3 Present Final Recommendations ans Deisng to Stantec	Milestone	2/28/2019	
3.4 Submit Final Report and eCDR	Milestone	3/1/2019	

Figure 7: Milestone Start Dates

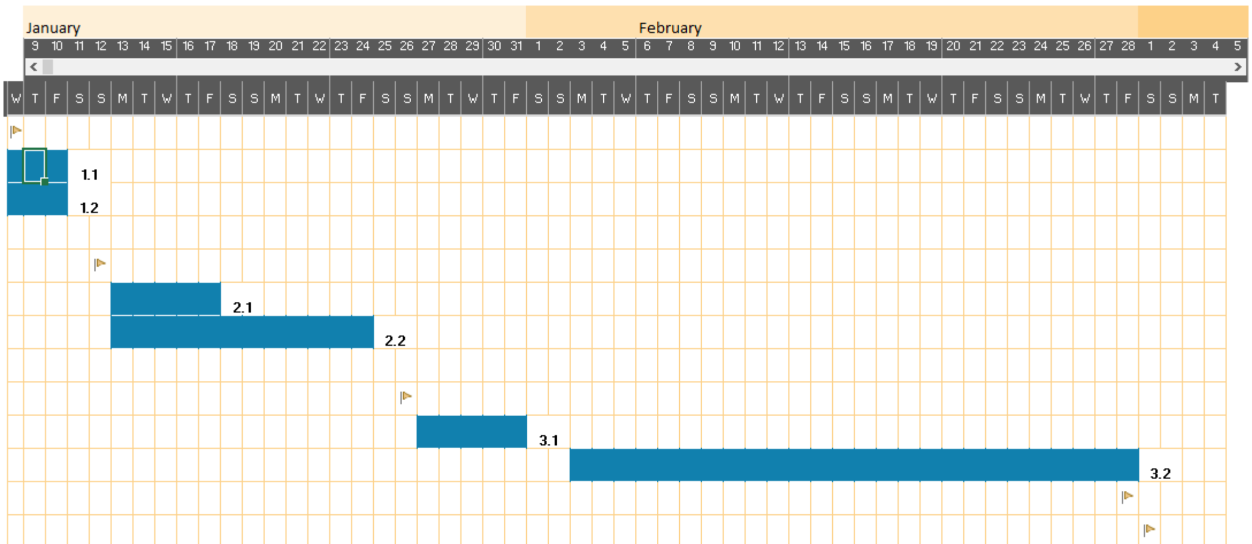

Figure 8: Project Schedule Gantt Chart

3.6 Develop Final Recommendations and Design

The project will conclude with the development of an improvement design for the Saugus River pedestrian bridge. The team's final deliverable will consist of a structural design and model, as well as a work breakdown structure and cost estimate.

In terms of the structural design, the team will define primary member sizes as well as common connection details that are compliant with the governing codes identified in "Section 3.1: Identify Bridge Users and Load Requirements". These member sizes and connection details will be accompanied by a finite element structural analysis model that will illustrate the bridge's functionality under load.

In addition to the structural design, a work breakdown structure with network diagram identifying the critical path will be created for the chosen design option. This schedule will identify critical tasks and milestones in the project to assist the pre-construction planning process. The team will be able to produce an expected duration for construction as well as a cost estimate based on this schedule. To produce the cost estimate, the team will use a material takeoff approach in conjunction with the 2017 material and labor prices set forth by Massachusetts Department of Transportation.

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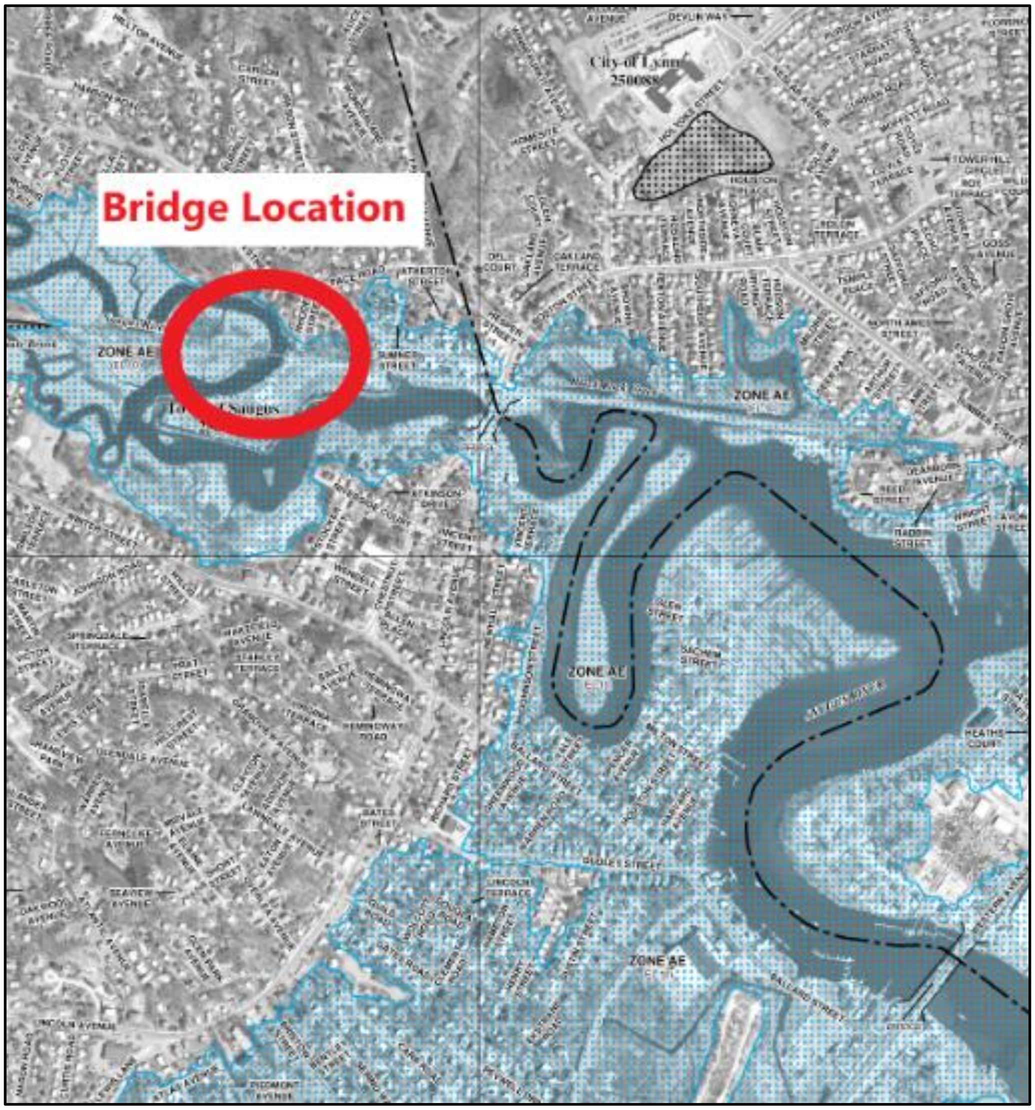
Appendix B: Methods Flowchart

Appendix C: FEMA Flood Insurance Rating Map of Project Area

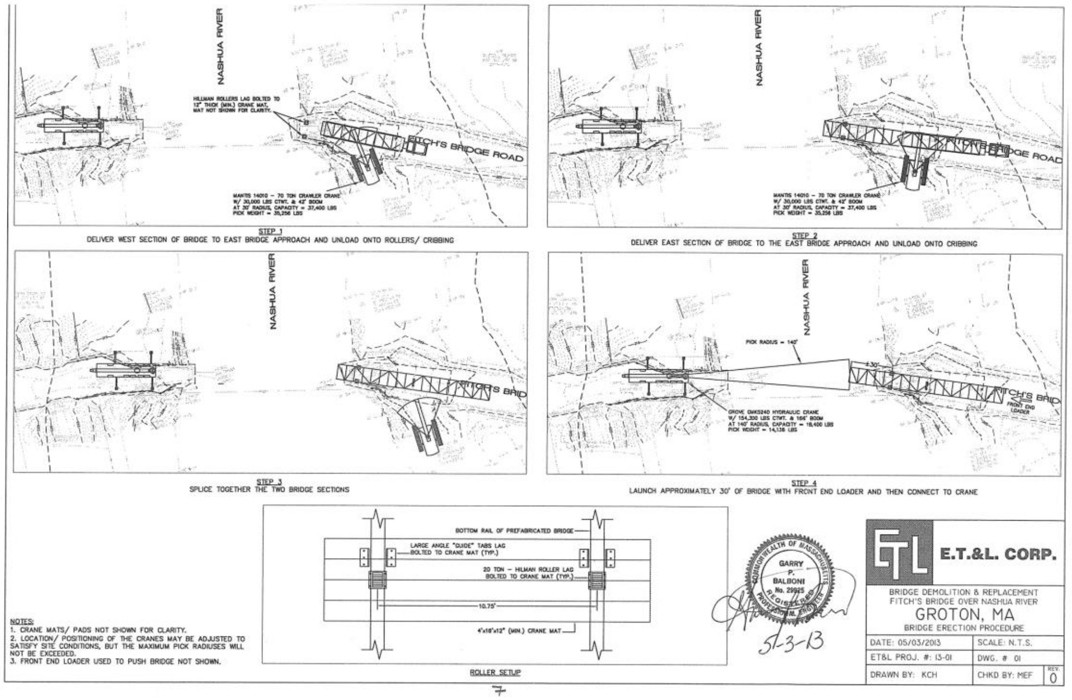


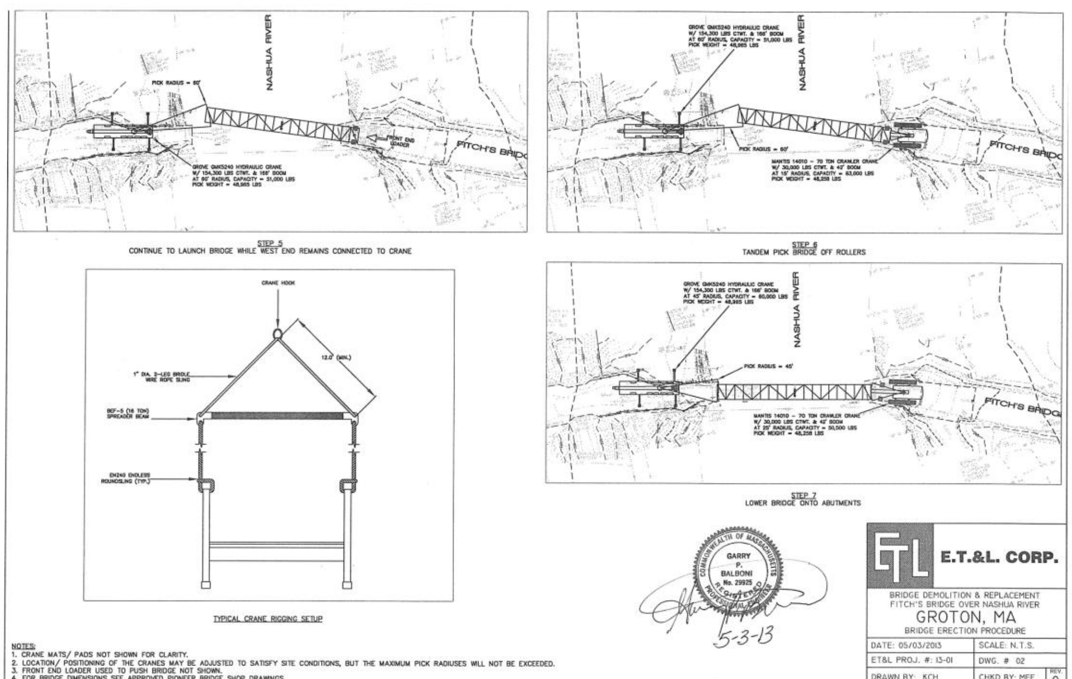
Source: FEMA, 2014

Appendix D: Pro-Con List Evaluation of Design Alternatives

	Project Alternative		
Attributes	**Rehabilitation (All Critical Members)**	**Replacement (Partial Demo A)**	**Replacement (Partial Demo B)**
Ethics	+ (satisfies all eight canons put forth in ASCE Code of Ethics)		
Health and Safety	+ (replacement of structurally deficient members, thus restoring bridge to non-deficient condition)		
Demolition Costs	+ (no demolition costs required for rehabilitation since the original structure will remain intact)	+ (moderate total cost with need for environmental control measures due to slight wetland disturbance)	- (high demolition cost due to dredging operation required to cut piles off below mudline)
Construction Costs	- (high costs of necessary techniques required for rehabilitating critical members and high costs resulting from environmental control measures from wetland disturbances)	+ (low construction costs due to the simple installation of a prefabricated bridge)	+ (low construction costs due to the simple installation of a prefabricated bridge)
Environmental	- (jacking required to replace piles & pile caps would require cofferdams/dewatering)	+ (no direct construction work within the river, and partial removal of hazardous creosote treated timber)	- (dredging would be required to remove the remainder of the piles requiring work within the river)
Aesthetics	- (metal jacketing and FRP wrapping of piles would detract from appeal)	- (leaving the failing piles underneath the new bridge would decrease visual appeal)	+ (removal of failing members and installation of new bridge would increase visual appeal)
Bridge Constructability	- (the extent of required rehabilitation measures limits the effectiveness of this alternative)	+ (bridge can be prefabricated off-site and easily installed on-site)	+ (bridge can be prefabricated off-site and easily installed on-site)
Notes:	"Aesthetic Improvements," "Replacement with No Demolition," and "Replacement with Full Demolition" were evaluated with this Pro-Con List. However, these options were eliminated from consideration due to their violation of the ASCE Code of Ethics as seen in "4.4.1 Pro-Con List."		

Appendix E: Example Roller and Tandem Pick Erection Procedure





<u>Source:</u> Stantec, 2019

Appendix F: Bridge Spreadsheet Calculations

BASIC GEOMETRY

Bridge Geometry	
Span	130 feet
Deck Width w_{deck}	12.75 feet
CL-CL trusses	13.5 feet
Truss Height at Center	13 feet

Member Sizes

Top Chords				Bottom Chords			
Section: HSS				Section: HSS			
Size	9	9	6	Size	9	9	6
Area	11.8 in^2			Area	11.8 in^2		
Weight	42.79 plf			Weight	42.79 plf		

Diagonals				Verticals			
Section: HSS				Section: HSS			
Size	4	4	2	Size	7	7	3
Area	1.77 in^2			Area	4.67 in^2		
Weight	6.46 plf			Weight	17.08 plf		

Floor Beams		Crossbracing	
Section: W		Section: W	
Size	10x30	Size	6x12
Area	8.84 in^2	Area	3.55 in^2
Weight	30 plf	Weight	12 plf

Addditional Member Geometry

Top Chord		Bottom Chord	
r	3.51 in	r	3.51 in

Diagonals		Verticals	
Length	16.4 ft	I_c	36 in^4

Floorbeams		Floor Diagonals	
I_x	170 in^4	Height	6.03 in
S_x	32.4 in^3	Length	16.2 ft
Spacing	10 ft	Spacing	10 ft

LOADS

Dead Loads		
Weight Per Truss Panel:		
Top Chord	(W_{TC})	427.9 lb
Bottom Chord (W_{BC})	(WBC)	427.9 lb
Verticals	(W_{V})	444.08 lb
Diagonals	(W_{D})	211.89 lb
Weight Per Truss:		
Floor Beams (W_{FB})		2713.2 lb
Floor Diagonals (W_{FD})		2721.6 lb
Decking (W_{deck})		600 lb
Total Sum (plf)		
Railing (W_{Rail})		30.74 plf
Total Dead Load		283.7 plf

Pedestrain Load (Live)		
Uniform Load	90	psf
(Per Ped. Spec 3.1)		
Tributary width/truss	6 ft	
Distributed Load	540	

Vehicle Load (Live)	
Deck width >10 ft	
12.75 > 10	
For a deck width over 10 ft, use the H10 Design Vehicle	
(Per Ped. Spec 3.2)	
Front Axle	4 kips
Rear Axle	16 kips
Axle Spacing	14 ft
Wheel Spacing	6 ft

Wind Load	
Horizontal Wind Loading (Ped Spec 3.4)	
P_z	$0.00256K_zGV^2I_rC_d$

Variables	Definition	Value	AASHTO Signs Spec. Reference
Kz	Height and Exposure Factor	1	3.8.4
G	Gust Effect Factor	1.14	3.8.5
V	Basic Wind Velocity	105	3.8.2
I_r	Wind Importance Factor	1.15	3.8.3
C_d	Wind Drag Coefficient	2	3.8.6-1
P_z	Design Wind Pressure	74 psf	3.8.3-1

Projected Vertical Area		
Top Chord	7.5	plf
Bottom Chord	7.5	plf
Verticals	15.17	plf
Diagonals	10.93	plf
Deck	1.67	plf
Floor Diagonals	5.03	plf
Railing	1.79	plf
Total	49.59	plf

Total Horizontal Wind of Superstructure (WS_H)	
WS_H	734.08 plf

Vertical Wind Loading

$WS_{V,T} = P_v * w_{deck}$

Variables	Definition	Value	Spec Reference
P_v	Vertical Uplift Force	0.02 ksf	AASHTO LRFD 3.8.2
w_{deck}	Deck width	12.75 ft	n/a
$WS_{V,T}$	Vertical Wind Loading	255 plf	

Vertical Load on Leeward Truss	187.71 plf
Vertical Load on Windward Truss	67.29 plf

LOAD COMINBATIONS

Limit State	DC1+DC2	PL	WS
Strength 1	1.25	1.75	0
Strength 3	1.25	0	1.00
Service 1	1.00	1.00	1.00
Serivce 2	1.00	1.3	0
Serivce 3	1.00	1.00	0
Service 4	1.00	0	1.00
Fatigue 1	0.00	1.00	0
Extreme Event 1	1.00	0.5	0
Extreme Event 2	1.00	0.5	0

References:
Spec. AASHTO LRFD 3.4
Ped Spec 3.7

Limit State	Generated Load
Strength 1	1.3 klf
Strength 3	0.54 klf
Service 1	1.01 klf
Serivce 2	0.99 klf
Serivce 3	0.82 klf
Service 4	0.47 klf
Fatigue 1	0.54 klf
Extreme Event 1	0.55 klf
Extreme Event 2	0.55 klf
Governing:	Strength 1

Tensile Strength Analysis (using Strength I Load Combination)			
Details	Member	$\Phi_t P_n$ (k)	P_{red} (k)
Determined by Top Chord Compression	BC1	411.525	195
	BC2	411.525	204.286
	BC3	411.525	205.263
	BC4	411.525	212.727
	BC5	411.525	212.245
	BC6	411.525	210
	BC7	411.525	208.456
	BC8	411.525	210
	BC9	411.525	212.245
	BC10	411.525	212.727
	BC11	411.525	205.263
	BC12	411.525	204.286
	BC13	411.525	195
	V1	162.86625	19.5
	V2	162.86625	13.929
	V3	162.86625	21.211
	V4	162.86625	12.409
	V5	162.86625	10.082
	V6	162.86625	10.993
	V7	162.86625	10.93
	V8	162.86625	10.082
	V9	162.86625	12.409
	V10	162.86625	13.929
	V11	162.86625	21.211
	V12	162.86625	19.15
Determined by Diagonals in Compression	D4	61.72875	0.763
	D5	61.72875	3.682
	D6	61.72875	2.532
	D7	61.72875	2.532
	D8	61.72875	3.682
	D9	61.72875	0.763

Compression Strength Analysis (Using Strength 1 Load Combination)

Member	Cross-Sectional Area (in^2)	i-Coordinate		j-Coordinate		Member Length (ft)	r_x (in)
		X (ft)	Y (ft)	X (ft)	Y (ft)		
TC1	11.8	0	0	10	4	10.7703296	3.51
TC2	11.8	10	4	20	7	10.4403065	3.51
TC3	11.8	20	7	30	9.5	10.3077641	3.51
TC4	11.8	30	9.5	40	11	10.1118742	3.51
TC5	11.8	40	11	50	12.25	10.0778222	3.51
TC6	11.8	50	12.25	60	13	10.0280856	3.51
TC7	11.8	60	13	70	13	10	3.51
TC8	11.8	70	13	80	12.25	10.0280856	3.51
TC9	11.8	80	12.25	90	11	10.0778222	3.51
TC10	11.8	90	11	100	9.5	10.1118742	3.51
TC11	11.8	100	9.5	110	7	10.3077641	3.51
TC12	11.8	110	7	120	4	10.4403065	3.51
TC13	11.8	120	4	130	0	10.7703296	3.51
D1	1.77	10	0	20	7	12.2065556	1.78
D2	1.77	20	0	30	9.5	13.7931142	1.78
D3	1.77	30	0	40	11	14.8660687	1.78
D10	1.77	90	11	100	0	14.8660687	1.78
D11	1.77	100	9.5	110	0	13.7931142	1.78
D12	1.77	110	7	120	0	12.2065556	1.78

Slenderness Ratio	F_e (ksi)	F_{cr} (ksi)	ΦP_n (k)	P_{red} (k)
36.82163971	211.1015854	45.28103026	480.884541	210.021
35.69335559	224.6585679	45.55272564	483.769946	203.586
35.24021902	230.4732603	45.65990859	484.908229	210.573
34.57051011	239.4893291	45.81626143	486.568696	207.56
34.45409294	241.1104877	45.84318845	486.854661	214.383
34.2840532	243.508106	45.88238326	487.27091	212.841
34.18803419	244.877839	45.90444507	487.505207	211.544
34.2840532	243.508106	45.88238326	487.27091	212.841
34.45409294	241.1104877	45.84318845	486.854661	214.383
34.57051011	239.4893291	45.81626143	486.568696	207.56
35.24021902	230.4732603	45.65990859	484.908229	210.573
35.69335559	224.6585679	45.55272564	483.769946	203.586
36.82163971	211.1015854	45.28103026	480.884541	210.021
82.29138617	42.26578966	30.47425394	48.5454865	11.335
92.9872869	33.10172226	26.57058085	42.3269353	1.348
100.2206882	28.49593964	23.98957882	38.2153991	11.096
100.2206882	28.49593964	23.98957882	38.2153991	11.096
92.9872869	33.10172226	26.57058085	42.3269353	1.348
82.29138617	42.26578966	30.47425394	*48.545487*	11.335

STABILITY

DEFLECTIONS

Direction	Horizontal	Vertical
Deflection (in)	2.317	1.792
Limits (in)	4.33	4.33
Pass	Y	Y

Lateral Frame Design Force

Verifying assumption that truss verticals are adequate to resist lateral forces

Variables	Defintion	Value
K	Design Effective Length Factor	2.566

Limit of 0.01/K	>0.003	
0.01/K		0.0039

Referencces:
Ped Spec 7.1.1

Top Chord Lateral Support

To Determine K:

1) Compute CL/P_c

$$C = \frac{E}{h^2\left[(h/3I_c) + (b/2I_b)\right]}$$

Variables	Definition	Value
E	Modulus of Elasticity	29,000 ksi
b	Floor Beam Span	162 in
h	Effective Vertical Height	132 in
I_b	Floor Beam MoI	170 in^4
I_c	Vertical MoI	36 in^4

C	0.9798	k/in

L	unbraced chord length in compression	120 in
P_c	critical buckling load *1.33	285.13 k

CL/P_c	0.4124

References:
Ped Spec 7.1.2

2) Determine K Value by Interpolation

Interpolation Table Values		
1/k	n=12	n=14
0.45	0.624	0.537
0.4	0.454	0.428

n = number of truss panels

X1	0.5805	Y1	0.45
X2	0.412358117	Y2	1/k
X3	0.441	Y3	0.4

1/k	0.3897
K	2.566

References:
Ped Spec 7.1.2
Ped Spec Table 7.1.2-1

Top Chord Compressive Resistance

Check KL/r

For main members:	$KL/r \leq 120$	
Top Chord Section:	HSS 9x9x6	
KL/r		$87.73 \leq 120$

Determine P_n (nominal compressive resistance)

$$\lambda = \left(\frac{KL}{r_s \pi}\right)^2 \frac{F_y}{E}$$

$\lambda =$ 1.34

If $\lambda \leq 2.25$, then:	
$P_n = 0.66^{\lambda} F_y A_s$	
P_n (k)	338.1

References
AASHTO LRFD 6.9.4.1-1
AASHTO LRFD 6.9.4.1-3

ϕ_c (Compressive Resistance Factor) = 0.9	
ϕ_c P_n (k) =	303.78

ϕ_c P_n > Max Factored Compressive Top Chord Force
303.78 k > 214.383 k

References:
AASHTO LRFD 6.5.4.2

VIBRATIONS

Direction	Vertical	Horizontal
Fundamental Frequency	3.0 Hz	1.3 Hz
Calculated	9.973 Hz	2.209 Hz
Passing	Y	Y

References:
Ped Spec 6

Note:
Calculations are based on RISA-3D Eigensolution analysis

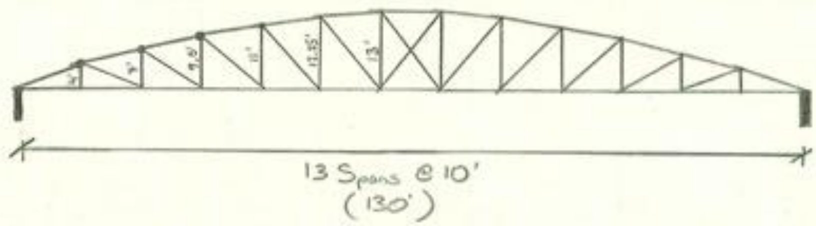
Appendix G: Bridge Hand-Calculations

Loads

- Dead Load (DL)
- Pedestrian Load (LL)
- Vehicle Load (LL)
- Wind Load (WS)

<u>Non-Factors</u>: Earthquake, Equestrian

Truss Geometry



Material Properties

Member	Size	Weight (lb/ft)	Area (in^2)	I (in^4)	r (in)	Workable Flat (in)		
Top Chord	HSS 9x9x3/8	42.79	11.8	145	3.51	7 5/16		
Bottom Chord	HSS 9x9x3/8	42.79	11.8	145	3.51	7 5/16		
Vertical	HSS 7x7x5/16	17.08	4.67	36	2.77	6 9/16		
Diagonal	HSS 4x4x1/8	6.46	1.77	4.40	1.58	3 7/16		

Member	Size	Weight (lb/ft)	Area (in^2)	I_x (in^4)	r_x (in)	d (in)	T (in)	b_f (in)
Floor Beam	W 10x30	30	8.84	170	4.38	10.5	8 1/4	5.81
Floor Diagonal	W 6x12	12	3.55	22.1	2.49	6.03	4 1/2	4.0

Dead Load (DL)

Top Chord (HSS 9x9x3/8)

Nominal Weight = 42.79 lbs/ft
Length = 10 ft

Weight (Based On Center Panel) = 1 member $\times$ (42.79 lb/ft)(10 ft)
↳ = 427.9 lbs

Bottom Chord (HSS 9x9x3/8)

« Same As Top Chord »

Weight (Based On Center Panel) = 427.9 lbs

Decking

Nominal Weight = 60 lb/ft³ (Hardwood)
Member Size = 2" x 8"
Truss Trib. Width = 12'/2 = 6'

Weight (Based On Center Panel) = $(\frac{2}{12})'(6')(10')(60\ lb/ft^3)$
↳ = 600 lb

Verticals (HSS 7x7x3/16)

Nominal Weight = 17.08 lbs/ft
Length = 13 ft

Weight (Based On Center Panel) = 2 members $\times$ (17.08 lb/ft)(13 ft)
↳ = 444.08 lbs

Diagonals (HSS 4x4x1/8)

Nominal Weight = 6.46 lb/ft
Length = 16.40 ft

Weight (Based On Center Panel) = 2 members $\times$ (6.46 lb/ft)(16.40 ft)
↳ = 211.89 lbs

Floor Beams (W10x30)

Nominal Weight = 30 lb/ft
Length = 12.75 ft

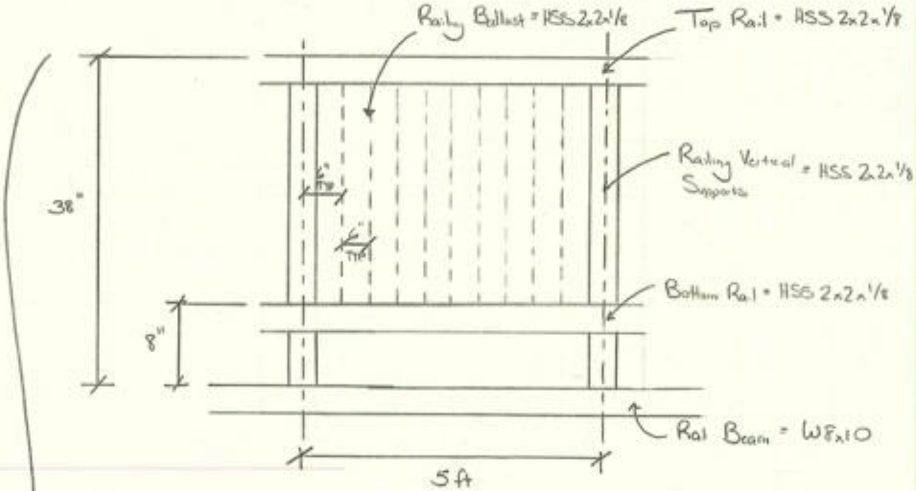
Weight (Based On Entire Bridge) = (14 members)(12.92 ft)(30 lb/ft) / 2 Trusses
↳ = 2,713.2 lb/truss

Floor Diagonals (W6x12)

Nominal Weight = 12 lb/ft
Length = 16.2 ft

Weight (Based On Entire Bridge) = (28 members)(16.2')(12 lb/ft) / 2 trusses
↳ = 2,721.6 lb/truss

Dead Load (DL) (continued)



Railing

Note: The Handrail detail served to approximate the dead weight of a handrail that could be installed on the bridge. The design is not fully compliant with ADA Standards, however the following standards were used to determine the basic geometry of the railing:

Height → ADA Std. 505.4
Clearance → ADA Std. 505.5
Non-Circular Cross Sections → ADA Std. 505.7.2

For Every 5 Feet of Railing...

Top Rail Weight = (5')(3.05 lb/ft) = 15.25 lb
Bot. Rail Weight = (5')(3.05 lb/ft) = 15.25 lb
Vert. Support Weight = (1 member)(36"/12)(3.05 lb/ft) = 9.15 lb
Ballast Weight = (9 members)(28"/12)(3.05 lb/ft) = 64.05 lb
Rail Beam Weight = (5 ft)(10 lb/ft) = 50 lb

$$\sum(\text{Weights}) = 153.7 \text{ lb}$$

Uniform Distributed Dead Load of Railing $= \sum(\text{Weights Over 5 ft})/5\text{ ft} = \frac{153.7\text{ lb}}{5\text{ ft}} = \underline{\underline{30.74 \text{ lb/ft}}}$

Dead Load (DL) (continued)

Uniform Dead Load of Half of Bridge (Based On Center Panel)

$$= \frac{W_{TC}}{10\text{ ft}} + \frac{W_{BC}}{10\text{ ft}} + \frac{W_{V}}{10\text{ ft}} + \frac{W_{D}}{10\text{ ft}} + \frac{W_{FB}}{130'} + \frac{W_{FD}}{130'} + \frac{W_{Deck}}{10'} + W_{rail}$$

$$= \frac{1}{10'}\left(W_{TC} + W_{BC} + W_{V} + W_{D}\right) + \frac{1}{10'}\left(W_{FB} + W_{FD}\right) + \frac{W_{Deck}}{10'} + W_{rail}$$

$$= \frac{1}{10'}\left(427.9\text{ lb} + 427.9\text{ lb} + 444.08\text{ lb} + 211.89\text{ lb}\right)$$
$$+ \left[\left(2,713.2\ \text{lb/truss}\right)\left(\frac{1\text{ Truss}}{130'}\right)\right] + \left[\left(2,721.6\ \text{lb/truss}\right)\left(\frac{1\text{ Truss}}{130'}\right)\right]$$
$$+ \frac{1}{10'}\left(600\text{ lb}\right) + 30.74\ \text{lb/ft}$$

$$= \underline{\underline{283.7\ \text{lb/ft}}}$$

Pedestrian Load (LL)

Uniform Pedestrian Load = 90 psf (Ped. Spec. - 3.1)

Pedestrian Tributary Width Per Truss = $\frac{\text{Pedestrian Walkway Width}}{2} = \frac{12'}{2} = 6'$

Uniform Distributed Pedestrian Load = $\omega_p = (90\ psf)(6\ ft)$

$\underline{\underline{\omega_p = 540\ ^{lb}/_{ft}}}$

Vehicle Load (LL)

Clear Span Width of Deck = $\underline{\underline{12\ ft > 10\ ft}}$ (Ped. Spec. - 3.2)

Thus, This Calls For H10 Design Vehicle

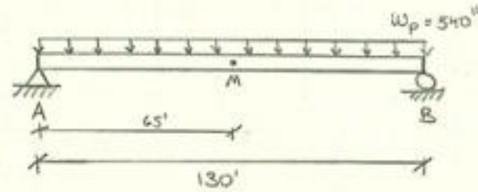
(This Loading Was Not Used In Load Combinations Because Live Load Is Governed By The Pedestrian Load, & These 2 Loadings Cannot Be Placed Simultaneously, As Per Ped. Spec - 3.2.)

H10 { Front Axle : 4 k
Rear Axle : 16 k
Axle Spacing : 14 ft
Wheel Spacing : 6 ft

Live Load : Determining The Governing Load

For the purposes of determining the governing live load, the truss structure will be simplified as a 130' simple beam.

Pedestrian Live Loading:

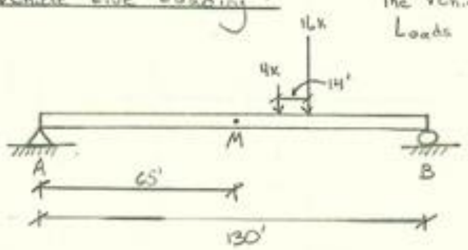


$$M_{max} = \frac{\omega \ell^2}{8} = \frac{(0.54^k/ft)(130')^2}{8} = 1{,}140.75 \text{ K-ft}$$

The Pedestrian Loading Generates A Maximum Moment of 1,140.75 ft @ Point M on the beam.

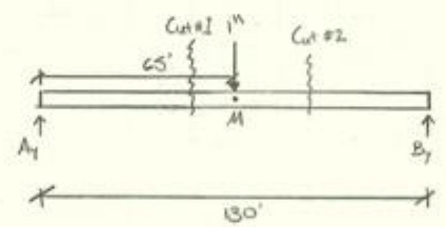
Vehicle Live Loading:

The Vehicle Load Is Composed of 2 Point Loads (4K & 16K) @ A Spacing of 14'.



As Per Ped. Spec. - 3.2, The Vehicle Loading "Shall Be Placed To Produce The Maximum Load Effects".

• Step 1: Determine Moment For Unit Load @ Midspan



Greatest Moment Will Occur @ Midspan (M). Thus, The Unit Load Was Placed @ Midspan (M).

$+\uparrow \sum F_y = 0$

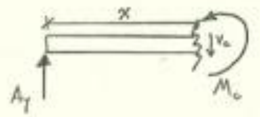
$0 = A_y + B_y - 1^k$

(Via Symmetry...)

$A_y = 0.5^k \quad B_y = 0.5^k$

Live Load: Determining The Governing Load (cont.)

◦ Step 1: (continued)

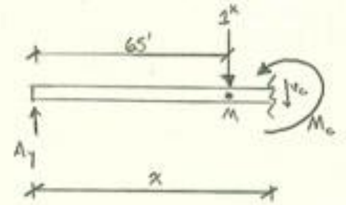


$\circlearrowleft^{+} \sum M_{cut} = 0$

$$0 = A_y(x) - M_c$$

$$M_c = (0.5^k)x$$

$$(0' \le x \le 65')$$



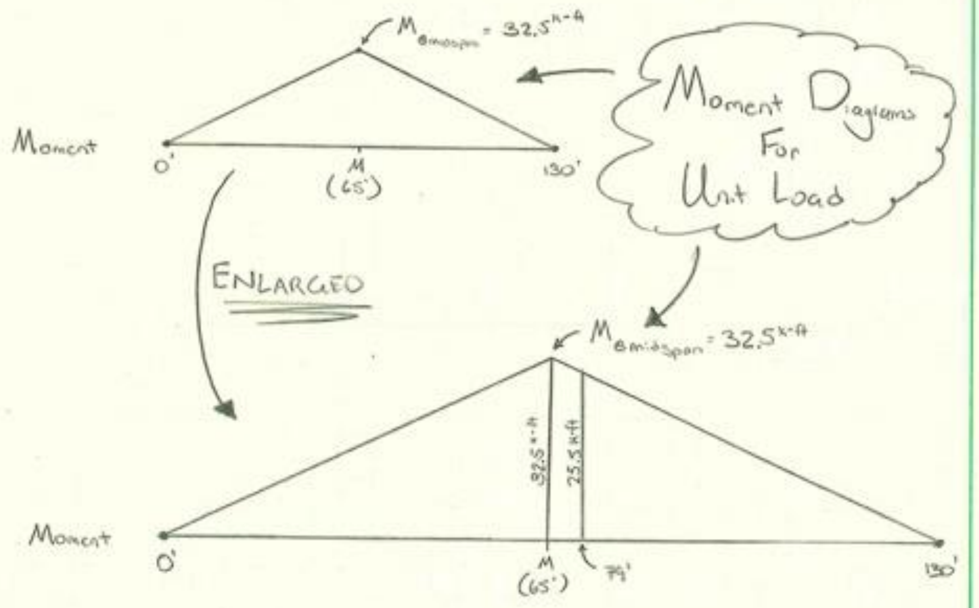
$\circlearrowleft^{+} \sum M_{cut} = 0$

$$0 = A_y(x) - (1^k)(x-65') - M_c$$

$$M_c = (0.5^k)x - (1^k)x + 65^{k\text{-}ft}$$

$$M_c = 65^{k\text{-}ft} - (0.5^k)x$$

$$(65' \le x \le 130')$$

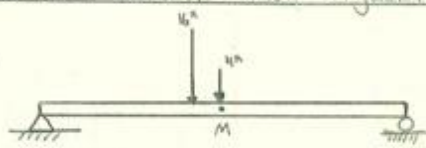


Live Load : Determining The Governing Load (cont.)

• Step 2 : Determine Vehicle Configuration w/ Largest Moment Effect

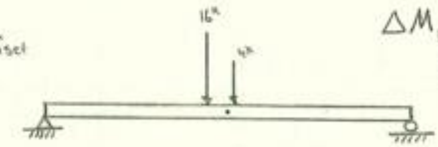
I

4ᴷ Load w/ 0' Offset From M



II

4ᴷ Load Is Offset 2' To Right of M

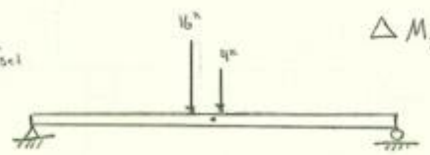


$$\Delta M_{1-2} = -4\left(\frac{32.5\text{ k-ft}}{65'}\right)(1^{ft}) + 16\left(\frac{32.5}{65}\right)(1)$$

$\hookrightarrow = 6$ k-ft

III

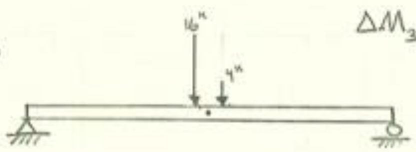
4ᴷ Load Is Offset 4' To Right of M



$\Delta M_{2-3} = 6$ - kft

IV

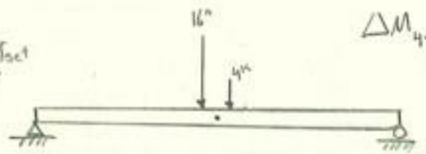
4ᴷ Load Is Offset 6' To Right of M



$\Delta M_{3-4} = 6$ k-ft

V

4ᴷ Load Is Offset 8' To Right of M

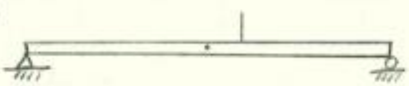


$\Delta M_{4-5} = 6$ k-ft

$\Delta M_{5-6} = 6$ k-ft

VI

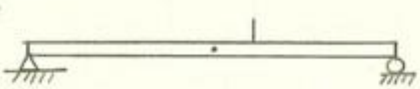
4ᴷ Load Is Offset 10' To Right of M



$\Delta M_{6-7} = 6$ k-ft

VII

4ᴷ Load Is Offset 12' To Right of M



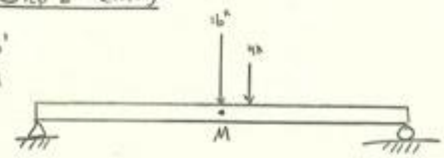
$\Delta M_{7-8} = 6$ k-ft

Live Load: Determining The Governing Load (cont.)

VIII

- Step 2: (cont.)

16^K Load w/ 0' Offset From M



There Was Never A ΔM That Was Negative; Therefore The Final Case (#8) Is The Vehicular Load Configuration That Produces The Greatest Moment Effect.

$$M_{case\ 8} = (16^K)(32.5^{K\text{-}ft/K}) + (4^K)(25.5^{K\text{-}ft/K})$$

$$\hookrightarrow = 622\ K\text{-}ft\ (@\ \text{Midspan})$$

- Step 3: Compare The Generated Moments

$$M_{max,\ Pedestrian} > M_{max,\ Vehicular}$$

$$(1{,}140.75\ K\text{-}ft > 622\ K\text{-}ft)$$

$\therefore$ The Pedestrian Load Serves As The Governing Load For Strength Design

Wind Load (WS)

Horizontal Wind

Basic Wind Speed (V) ⟶ 105 mph (AASHTO Signs 3.8.2)
Wind Importance Factor (I_r) ⟶ 1.15 (Ped. Spec. - 3.4)
Height Exposure Factor (K_z) ⟶ 1 (AASHTO Signs 3.8.4)
Gust Effect Factor (G) ⟶ 1.14 (AASHTO Signs 3.8.5)
Drag Coefficient (C_d) ⟶ 2 (AASHTO Signs 3.8.6)

Horiz. Wind Pressure: $P_z = 0.00256 * K_z * G * V^2 * I_r * C_d$ (AASHTO Signs 3.8.3)
$= 0.00256\,(1.00)(1.14)(105\text{mph})^2(1.15)(2.00)$
$= 74.0$ psf

Projected Vertical Area Per Linear Foot of Truss (Assuming Center Panel)

Top Chord: $(9'')(1/12')(10') = 7.5\ ft^2/panel$

Bot. Chord: $(9'')(1/12')(10') = 7.5\ ft^2/panel$

Verticals: $2\times(7'')(1/12')(13') = 15.17\ ft^2/panel$

Diagonals: $2\times(4'')(1/12')(16.4') = 10.93\ ft^2/panel$

Decking: $(2'')(1/12')(10') = 1.67\ ft^2/panel$

Floor Diagonals: $(6.03'')(1/12')(10') = 5.03\ ft^2/panel$

+ Railing: See Next Page $= 1.79\ ft^2/panel$

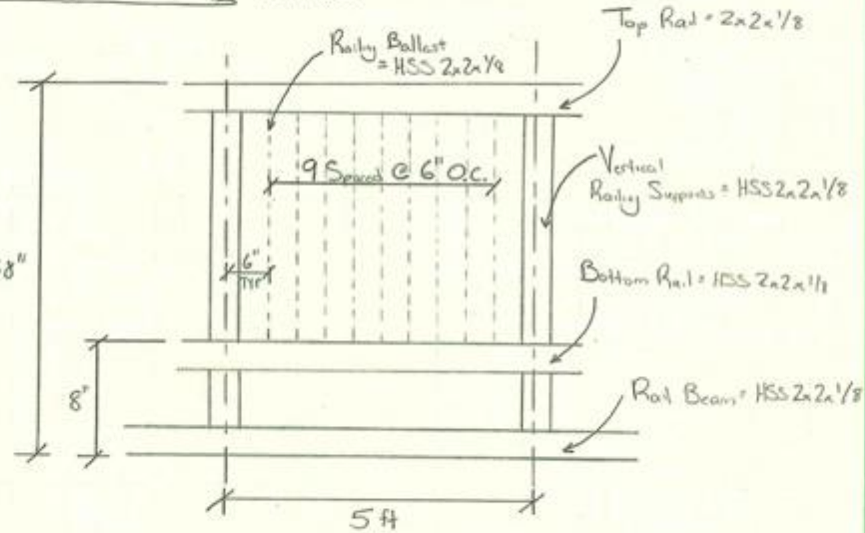
$\sum$ (Vertical Areas of Center Truss Members) $= 49.59\ ft^2/panel$

⇓

Projected Vertical Area Per Linear Foot $= \left(\frac{49.59\ ft^2}{Panel}\right)\left(\frac{Panel}{10'}\right)$
$= 4.96\ ft^2/ft$

Uniform Horizontal Wind Load On Entire Bridge (WS_H) $= (P_z)$(Projected Vert. Area Per Linear Foot)
$= (2\ \text{trusses})(74.0\ lb/ft^2)(4.96\ ft^2/ft)$
$= 734.08\ lb/ft$

Wind Load (WS) (continued)



Horizontal Wind (cont.)

Railing Area Exposed To Lateral Wind:

Top Rail: $SA_{exposed} = (2")(\frac{1}{12})(5') = \frac{5}{6}\ ft^2$

Bot. Rail: $SA_{exposed} = (2")(\frac{1}{12}')(5') = \frac{5}{6}\ ft^2$

Vertical Support: $SA_{exposed} = 1 \text{ member} \cdot (36")(2")/144 = 0.5\ ft^2$

Ballast: $SA_{exposed} = 9 \text{ members} \cdot (28")(2")/144 = 3.5\ ft^2$

Railing Beam: $SA_{exposed} = (7\frac{7}{8}")(\frac{1}{12}')(5') = 3.28\ ft^2$

+

$$\sum (SA_{exposed \text{ per } 5 ft \text{ of Railing}}) = 8.95\ ft^2$$

$$\Downarrow$$

Projected Vertical Area of Railing Per Linear Foot of Truss $= 8.95\ ft^2 / 5\ ft = \underline{\underline{1.79\ ft^2/ft}}$

Wind Load (WS) (continued)

Vertical Wind

Deck Width (W_{deck}) = 12.75 ft = 12'9"

Vertical Uplift Force (P_v) = 0.02 ksf (AASHTO LRFD 3.8.2)

Total Vertical Wind Force ($WS_{v,T}$) $= (P_v)(W_{deck})$

$= (0.02\ \text{ksf})(1000\ \text{lb/k})(12.75')$

$\rightarrow = 255\ \text{lb/ft}$

Breaks Down Into 2 Parts...

Vertical Wind Load On Windward Truss $= WS_{v,T}\left[\left(\frac{1}{4}(\text{Deck Width}) + \left(\frac{1}{2}\right)\left(\text{Bot. Chord Width}\right)\right) \Big/ \left(\text{℄-℄ of Trusses}\right)\right]$

$= 255\ \text{lb/ft}\left[\left(\frac{1}{4}(12.75') + \left(\frac{1}{2}\right)\left(\frac{9}{12}\right)\text{ft}\right) \Big/ (13.5\ \text{ft})\right]$

$\rightarrow = 67.29\ \text{lb/ft}$

Vertical Wind Load On Leeward Truss $= WS_{v,T}\left[\left(\frac{3}{4}(\text{Deck Width}) + \left(\frac{1}{2}\right)\left(\text{Bot. Chord Width}\right)\right) \Big/ \left(\text{℄-℄ of Trusses}\right)\right]$

$= 255\ \text{lb/ft}\left[\left(\frac{3}{4}(12.75') + \left(\frac{1}{2}\right)\left(\frac{9}{12}\right)\text{ft}\right) \Big/ (13.5\ \text{ft})\right]$

$\rightarrow = 187.71\ \text{lb/ft}$

Because The Vertical Wind Load On The Leeward Truss Exceeded That on The Windward Truss, It GOVERNS As The Vertical Force Applied To The Bridge.

$$\therefore WS_v = 187.71\ \text{lb/ft}$$

Load Combinations

Load Combinations

All Load Combinations Were Provided By AASHTO LRFD Table 3.4.1-1.

AASHTO Ped. Spec - 3.7 stated the following regarding the load combinations provided by AASHTO LRFD:

"Strength II, IV, and V need not be considered."
"Load Factor For Fatigue I LC Shall Be Taken As 1.0, & do not consider Fatigue II."

Strength I = γ_p DL + 1.75LL
= 1.25 (283.7 lb/ft) + 1.75 (540 lb/ft)
= 1.30 klf

Note:
γ_p = 1.25 (AASHTO LRFD 3.4.1)
γ_{LL} = 1.0 (AASHTO LRFD 3.4.1)
γ_{EQ} = 0.5 (AASHTO LRFD 3.4.1)

Strength III = γ_p DL + 1.00WS
= 1.25 (283.7 lb/ft) + 1.00 (187.71 lb/ft)
= 0.54 klf

Service I = 1.00 DL + 1.00 LL + 1.00WS
= 283.7 lb/ft + 540 lb/ft + 187.71 lb/ft
= 1.01 klf

GOVERNING LOAD COMBINATION: Strength I

Service II = 1.00 DL + 1.30 LL
= 1.00 (283.7 lb/ft) + 1.30 (540 lb/ft)
= 0.99 klf

Service III = 1.00 DL + γ_{LL} LL
= 1.00 (283.7 lb/ft) + 1.0 (540 lb/ft)
= 0.82 klf

Service IV = 1.00DL + 1.00WS
= 283.7 lb/ft + 187.71 lb/ft
= 0.47 klf

Fatigue I = 1.0 LL
= 1.0 (540 lb/ft)
= 0.54 klf

Extreme Event I = 1.00 DL + γ_{EQ} LL
= 1.00 (283.7 lb/ft) + 0.5 (540 lb/ft)
= 0.55 klf

Extreme Event II = 1.00 DL + 0.5LL
= 0.55 klf
«Same As Extreme Event I»

Strength Design

(Truss Members + Floorbeams)

Compression Strength Analysis

Top Chord Member 1 (TC1): (HSS 9x9x3/8)

i-Coordinate = (0 ft, 0 ft)
j-Coordinate = (10 ft, 4 ft) } Provided By RISA 2-D Truss Geometry

Member Length (Provided By Distance Formula)

$$= \sqrt{(x_j - x_i)^2 + (y_j - y_i)^2}$$

$$= \sqrt{(10'-0')^2 + (4'-0')^2}$$

$$= 10.77 \text{ ft}$$

Slenderness Ratio $= \dfrac{KL}{r}$

where: K = Effective Length Factor
L = Unbraced Length
r = Radius of Gyration

$$= \frac{(1.0)(10.77')}{(3.51'')(1/12)}$$

$$= 36.82$$

Note: Since All Truss Members Are Pinned @ Both Ends...

K = 1.0

Since $\left(\frac{KL}{r}\right) < 4.71\left(\sqrt{E/F_y}\right) = 113.4$... [For 50 ksi Steel]

These Columns Will Be Designed As Short-Intermediate Columns

Short-Intermediate Column

Euler Buckling Stress (F_e; ksi) $= F_e = \dfrac{\pi^2 E}{(KL/r)^2} = \dfrac{\pi^2 (29{,}000 \text{ ksi})}{(36.82)^2} = 211.1$ ksi

Critical Buckling Stress (F_{cr}, ksi) $= F_{cr} = \left[0.658^{(F_y/F_e)}\right] F_y = \left[0.658^{(50/211)}\right] 50$ ksi

$= 45.28$ ksi

Compression Strength Analysis (cont.)

Top Chord Member 1 (TC1) continued:

Nominal Compression Strength (ϕP_n; K) $= \phi P_n = \phi F_{cr} A$

$= 0.9(45.28 \text{ ksi})(11.8 \text{ in}^2)$

$= 480.87 \text{ K}$

$\phi P_n > P_{required}$ — Provided By RISA 2-D Analysis For Strength I Load Combo

$480.87 \text{ K} > 210.0 \text{ K}$ ✓

Thus, HSS 9x9x3/8 Satisfies The Strength Requirement For Strength I LC.

This Calculation Process Was Completed For The Remainder of Compressive Truss Members, Including:

TC 2 thru TC13

D1 thru D3

D10 thru D12

Tension Strength Analysis

Bot. Chord Member 1 (BC1): (HSS $9 \times 9 \times 3/8$)

Effective Cross-Sectional Area (A_e ; in^2) $= 0.75 A_g$
$= 0.75 (11.8 \, in^2)$
$= 8.85 \, in^2$

(A_e is calculated as $0.75 A_g$ in order to capture the effect of typical end connections, as specified by AISC Steel Construction Manual, 15th Ed. Pt.5 - Pg 3.)

Nominal Tensile Strength ($\phi_t P_n$; K) $= \phi_t P_n = \phi F_u A_e$
$= 0.75 (62 \, ksi)(8.85 \, in^2)$
$= 411.53 \, K$

$$\phi_t P_n > P_{req}$$

$$411.53 \, K > 195 \, K \checkmark$$

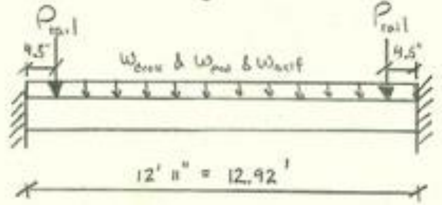
Thus, HSS $9 \times 9 \times 3/8$ Satisfies The Strength Requirement For Strength I LC.

This Calculation Process Was Completed For The Remainder of Tension Truss Members, Including:

BC2 thru BC13
V1 thru V12
D4 thru D9

Floorbeam Strength Design

Pedestrian Loading:



Tributary Width = 10 ft

$w_{deck} = (3/12)'(60^{lb}/ft^3)(10') = 100^{lb}/ft$ (DL)

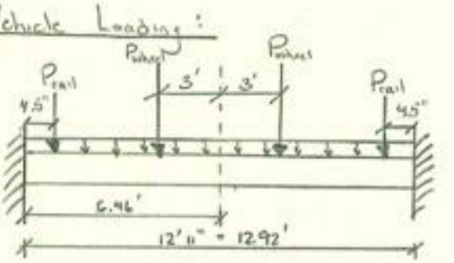
$P_{rail} = (30.74^{lb}/ft)(10') = 307.4$ lb (DL)

$w_{ped} = (90^{lb}/ft^2)(10') = 900^{lb}/ft$ (LL)

Using Strength I LC...

$M_{max} = 23.788$ k-ft (@ Fixed Ends)

Vehicle Loading:



Rear Axle Load Will Govern Over Front Axle Load.

Tributary Width = 10 ft

$w_{deck} = 100^{lb}/ft$ (DL)

$P_{rail} = 307.4$ lb (DL)

$P_{wheel} = 8^k$ (LL)

Using Strength I LC...

$M_{max} = 37.346$ k-ft (@ Fixed Ends)

[Note: No Need To Recalculate Maximum Moments & Recheck Strength Capacity Due To Large Strength Over-design of This Member.]

W10x30 Was Selected For Floor Beam Based On Top Chord Stability Design; This Section Satisfies This Strength Criteria. ✓

$(\phi M_p = 137 \text{ k-ft}) > 37.346 \text{ k-ft}$ & $(\delta = 0.10") < L/360 = \frac{(12.92')(12"/')}{360} = 0.431"$

Serviceability Design

Deflections
Lateral Frame Design
Top Chord Support
Vibrations

Deflections

As Per Ped. Spec - 5.0, Deflections Were Analyzed At The Service I Load Combination.

Vertical Deflection Due To Unfactored Ped. Live Loading $< \left(\frac{1}{360}\right)$(Span Length)

Provided By RISA - 2D $\longrightarrow$ $\left(\begin{array}{l} 1.792" < \left(130'(12"/1')/360\right) \\ 1.792" < 4.33" \checkmark \end{array}\right)$

Horizontal Deflection Due To Unfactored Wind Loading $< \left(\frac{1}{360}\right)$(Span Length)

Provided By RISA-3D $\longrightarrow$ $\left(\begin{array}{l} 2.317" < \left(130'(12"/1')/360\right) \\ 2.317" < 4.33" \checkmark \end{array}\right)$

Lateral Frame Design:

The Truss Verticals Were Assumed To Be Adequate To Resist The Lateral Force Specified By Ped. Spec. - 7.1.1, In Order For The Purposes of The "Top Chord Stability" Check.

Once The "Top Chord Stability" Check Passed, This Assumption Was Verified Through The Following Manner.

Minimum Lateral Force (K):

$$H_f = 0.01/K \quad \text{(Ped. Spec. - 7.1.1)}$$

Where: K = Effective Length Factor For Top Chord Members

$$= 0.01/2.566$$

$$= 0.0039 > 0.003 \checkmark$$

(Limiting Value Established By Ped. Spec - 7.1.1)

Top Chord Support

As Per Ped. Spec - 7.1.2, Floor Beam & Verticals Must Have Matched Member Widths In Simple Unreinforced HSS Connections

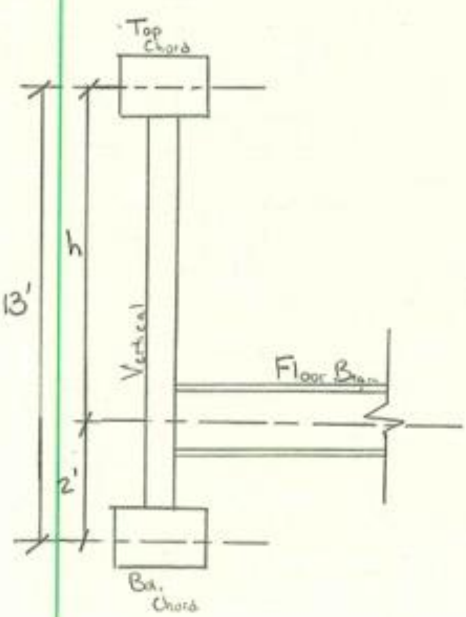
Thus, Floor Beam Flange Width Was Matched As Close As Possible To The Workable Flat Width of The HSS Vertical Member

Vertical Member:		Floor Beam:
HSS 7x7x3/16	→	W10x30
Workable Flat = 6 3/16"		Flange Width = 5 3/4"

Due To Lack of Overhead Lateral Support @ Tops of Trusses, Floorbeams Act As Springs When Trusses Are Exposed To Lateral Loading. Thus, A Spring Constant Was Encorporated Into This Analysis.

Transverse Frame Spring Constant (C; k/in) $= C = \dfrac{E}{h^2\left[(h/3I_c)+(b/2I_b)\right]}$

(Equation Based On "Guide To Stability Design Criteria For Metal Structures" By Galambos, 1968)



I_b = Moment of Inertia of Floor Bean (W10x30)
↳ = 170 in⁴

I_c = Moment of Inertia of Vertical (HSS 7x7x3/16)
↳ = 36 in⁴

b = Floor Beam Span (Based on ℄-℄ of Trusses)
↳ = 13.5' = 162"

h = Effective Height of Vertical
= 13' - 2'
= 11'
↳ = 132"

Top Chord Support (cont.)

Transverse Frame Spring Constant (K/in) $= C = \dfrac{29{,}000\text{ ksi}}{(132'')^2\left[\dfrac{132''}{3(36\text{ in}^4)} + \dfrac{162''}{2(170\text{ in}^4)}\right]}$

$= 0.9798$ K/in

Calculating CL/Pc Value To Be Used In Ped. Spec. Table 7.1.2-1:

$\dfrac{CL}{P_c}$ (Ped. Spec. - 7.1.2)

where: C = Transverse Spring Constant
L = Unbraced Length of Top Chord
P_c = Max Factored Compressive Force In Top Chord Multiplied By 1.33.

$= \dfrac{(0.9798\text{ K/in})(10\text{ ft})(12\text{ in/ft})}{(1.33)(214.383\text{ K})}$

$= 0.4124$

Calculating Effective Length Factor Based On Ped. Spec. Table 7.1.2-1:

Given n = # of panels = 13 and CL/Pc = 0.4124...

CL/Pc Values For "n=12,14" From Ped. Spec. Table 7.1.2-1

1/K	n=12	n=13*	n=14
0.45	0.624	0.581	0.537
0.40	0.454	0.441	0.428

* CL/Pc Values For n=13 Were Tabulated By Averaging The Corresponding Values From n=12 & n=14.

$\left(\dfrac{1}{K}\right)_{n=13} = \left(\dfrac{0.45-0.40}{0.581-0.441}\right)(0.4124-0.441) + 0.40$

$= 0.3897$

$\therefore$

Effective Length Factor For Top Chord Members : K = 2.566

Top Chord Support (cont.)

Calculating Top Chord Compressive Resistance:

For HSS 9x9x3/8 ... $A_s = 11.8\text{ in}^2$, $r = 3.51\text{ in}$

$$KL/r = (2.566)(10\text{ ft}) / 3.51\text{ in} = 87.73$$

(This Value Satisfies The Limiting Value of 120 For Truss Members Provided By AASHTO LRFD - 6.9.3)

$$\lambda = \left(\frac{KL}{r_s \pi}\right)^2 \frac{F_y}{E} \quad \text{(AASHTO LRFD, Eq. 6.9.4.1-3)}$$

$$= (87.73)^2 \left(\frac{1}{\pi}\right)^2 \left(\frac{50\text{ Ksi}}{29{,}000\text{ Ksi}}\right)$$

$$= 1.34$$

Because $\lambda \le 2.25$, The Following Equation Is Used To Calculate Factored Compressive Resistance ..

Nominal Compressive Resistance → $P_n = 0.66^{\lambda} F_y A_s$ (AASHTO LRFD, Eq. 6.9.4.1-1)

$$= (0.66^{1.34})(50\text{ Ksi})(11.8\text{ in}^2)$$

$$= 338.1\text{ K}$$

Factored Compressive Resistance → $\phi_c P_n = 0.9(338.1\text{ K})$ (As Per AASHTO LRFD Eq. 6.5.4.2, $\phi_c = 0.9$)

$$= 304.29\text{ K}$$

$\phi_c P_n$ > Max Factored Compressive Force In Top Chord

$(304.29\text{K} > 214.383\text{ K})$ ✓

$\therefore$

Thus, The HSS 9x9x3/8 Satisfies The Top Chord Stability Requirement.

Vibrations

As Per "Ped. Spec. - Article 6"...

Fundamental Frequency In Vertical Mode > 3.0 Hz
(In Order To Avoid First Harmonic)

9.973 Hz > 3.0 Hz ✓

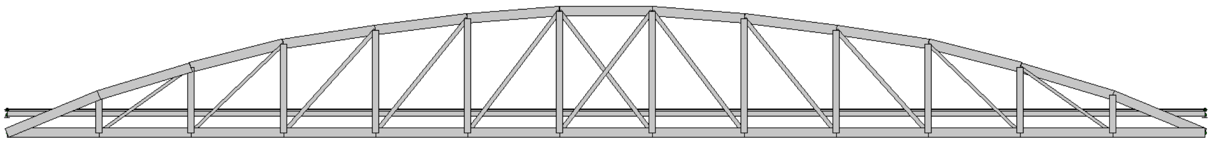
(As Provided By RISA-3D Eigensolution Nodal Analysis Based On Two Unit Loads, Each Applied In The Downward Direction @ Center Span of Each Truss)

Fundamental Frequency In Horizontal Mode > 1.3 Hz

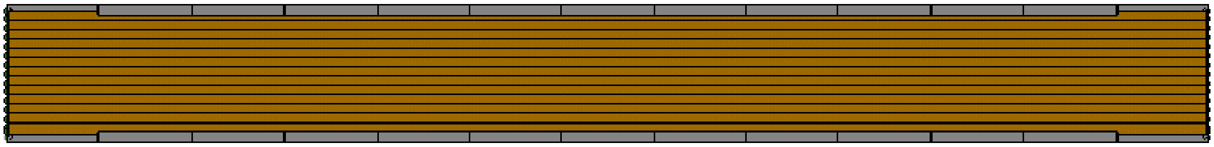
2.209 Hz > 1.3 Hz ✓

As Provided By RISA-3D Eigensolution Nodal Analysis Based On A Single Unit Load, Applied In The Lateral Direction @ Center Span of One Truss

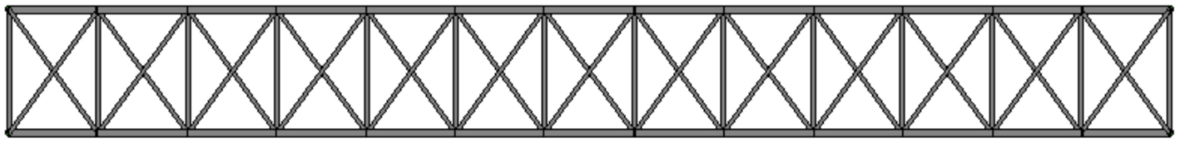
Appendix H: Bridge Renderings



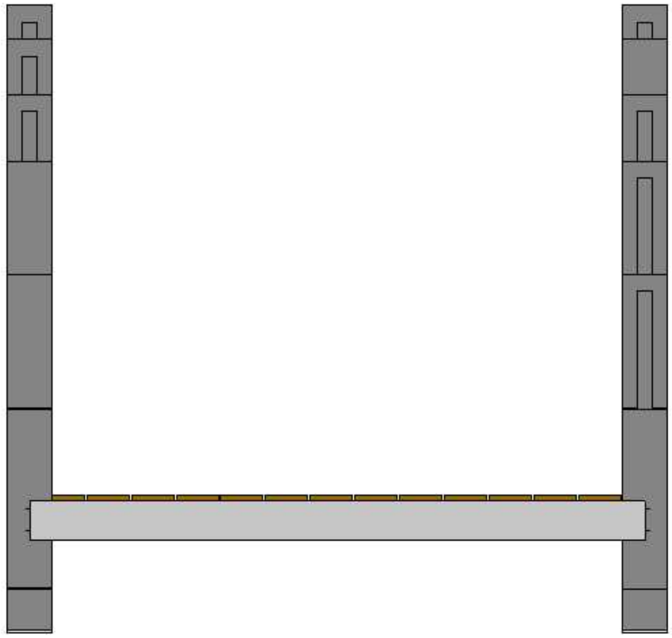
Elevation View of the Final Bridge Design



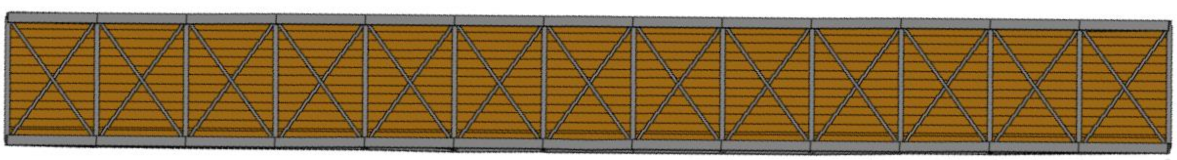
Plan View of the Final Bridge Design with Deck



Plan View of the Final Bridge Design without Deck



Cross-Section View of the Final Bridge Design



Bottom View of Final Truss Bridge Design with Deck

Appendix I: Abutment Spreadsheet Calculations

Design Step 1 - Design Criteria

Material Properties

Conc. density	Wc	kcf	0.15
Conc. 28 day comp. strength	f'c	ksi	3.00
Reinforcement strength	fy	ksi	60.00
Sandy gravel unit weight	yg	kcf	0.12
Sandy gravel fill friction angle	ϕ_g	degrees	40.00
Active lateral earth pressure coefficient	K_g		0.36

Soil Properties

Friction angle	ϕ_b	degrees	25.00
Specific weight of bearing soil	y	kcf	0.10
Active lateral earth pressure coefficient	K_b		0.41

Design Step 2 - Select Preliminary Abutment Dimensions

Abutment Dimensions

Footing depth	ft	9.00
Footing length	ft	13.00
Footing height	ft	1.00
Footing heel	ft	5.00
Footing toe	ft	3.00
Stem depth	ft	1.00
Stem length	ft	13.00
Stem height	ft	6.00

Design Step 3 - Compute DL Effects

Superstructure DL	DL	k	31.82
Effective DL on abutment	DL_{brg}	klf	2.45
Stem DL	DL_{stem}	klf	0.90
Footing DL	DL_{ftg}	klf	1.35
Earth DL	DL_{earth}	klf	5.76

Design Step 4 - Compute LL Effects			
Total LL	LL	k	70.20
Effective LL on abutment		klf	2.70
Surcharge load	Qs	ksf	0.24

Design Step 5 - Limit State

Stem

Load Factors	
	Strength I
Loads	γ_{max}
DC (DL of structural comps. and nonstructural comps.)	1.25
DW (DL of wearing surfaces and utilities)	1.5
LL (vehicular live load)	1.75
EH (horizontal earth pressure load)	1.5
LS (live load surcharge)	1.75

Factored Loads		
Stem DL	k	1.13
Effective LL on abutment	k	4.73
Effective DL on abutment	k	3.06
Lateral earth load (stem)	k	1.15
Lateral surcharage load (stem)	k	0.89
Stength I		0.88
Factored vertical force acting on base of st	k	8.91
Factored horizontal force acting on base o	k	2.05
Factored moment acting on base of stem	k-ft	4.98

Footing Heel

Load Factors	
	Strength I
Loads	γ_{max}
DC (DL of structural comps. and nonstructural comps.)	1.25
LL (vehicular live load)	1.75
EH (horizontal earth pressure load)	1.5
EV (vertical pressure from dead load of earth fil)	1.35
LS (live load surcharge)	1.75

Factored Loads		
Heel DL	k	0.94
Vertical earth load	k	4.86
Vertical surcharge on heel	k	2.08
Stength I		
Factored vertical force acting on the heel	k	7.88
Factored vertical force acting at the stem and heel	k-ft	19.70

Design Step 6 - Wall Stability Against Overturning

Earthquake Load	Location is in Seismic Zone so no analysis is necessary		
Lateral earth load on abuttement	P1	klf	1.57

Restoring Moment (Mr)			
	w (k)	x (ft)	Mr (k-ft)
Earth load on heel	4.86	6.50	31.59
Earth load on toe	2.70	1.50	4.05
Footing DL	1.69	4.50	7.59
Stem DL	1.13	3.50	3.94
Bridge LL+DD	5.85	3.50	20.48
Total restoring moment			67.65
Overturning Moment (Mo)			
	w (k)	x (ft)	Mr (k-ft)
Earth pressure (P1)	1.57	2.33	3.67
Surcharge load	1.04	3.50	3.64
Total			7.31

Check moment resistance

Restoring Moment	Mr	67.65
Overturning Moment	Mo	7.31
Factor of Safety		9.26
		PASS

Design Step 7 - Pressure Under Base			
Vertical Forces	V	K	16.22
Restoring Moment	Mr	k-ft	67.65
Overturning Moment	Mo	k-ft	7.31
Net Moment	Mnet	k-ft	60.34
Eccentricity	e	ft	0.78
B/6		ft	1.50
Max pressure at toe	Qmax	ksf	2.74
Maximum allowable bearing pressure	Qallow	ksf	3.00
Check for resistance			
Max pressure at toe	Qmax	ksf	2.74
Maximum allowable bearing pressure	Qallow	ksf	3.00
Factor of safety	FS		1.09
			PASS

Design Step 8 - Resistance to Sliding			
Coefficiant of friction	μ		0.47
Friction force	F	k	7.56
Sliding force	S	k	2.61
Check for resistance			
Factor of safety			2.90
			PASS

Design Step 9 - Reinforcement			
Stem			
Stem Cover	$Cover_s$	in	3.00
Preliminary bar diameter (#6)		in	0.75
Bar area		in^2	0.44
Effecetive Depth	d	in	3
Required area of reinforcement	As	in^2	0.50
Minimum required reinforcement	As min	in^2	
Spacing	S	in	13.00

Footing Heel

Bottom footing cover	$Cover_b$	in	3.00
Preliminary bar diameter (#10)		in	1.27
Bar area		in^2	1.27
Check moment requirement			
Effective depth	d	in	5
Required area of reinforcement	As	in^2	1.18
Check shear requirement			
Shear at heel and stem	V	k	7.88
Cracking factor	B		2
Modification Factor			0.9
Solve for d	d	in	7.00
Required area of reinforcement	As	in^2	0.84
Reinforce for moment resistance			
Spacing	S	in	11.00
Moment in the footing is most critical at heel so reinforcement is sufficient to extend to the toe			

Appendix J: Abutment Hand-Calculations

Abutment Design

Design step 1 – Design criteria

- Concrete density (W_c) = 0.15 kcf
- Concrete compressive strength (f'_c) = 3 ksi
- Reinforcement strength (f_y) = 60 ksi
- Gravel fill unit weight (γ_g) = 0.12 kcf

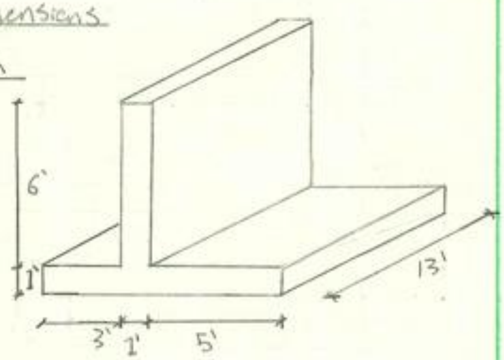
Gravel fill friction angle (ϕ_s) = 30°

Bearing soil properties

- Friction angle (ϕ_b) = 25°
- Specific weight (γ) = 0.10 kcf

Design Step 2 – Abutment dimensions

Section	Height	Length	Depth
Stem	6'	13'	1'
Footing	1'	↓	9'
Heel	1'	↓	5'
Toe	1'	↓	3'



Design step 3 – Compute dead load effects

- Bridge DL = 0.489 $\frac{k}{ft}$
- Bridge length = 130 ft
- Effective DL of bridge per abutment per length foot:

DL × length ÷ 2 abutments ÷ abutment length

$$\Rightarrow \frac{0.489\frac{k}{ft} \times 13ft}{2 \times 13ft} = 2.45\frac{k}{ft}$$

- DL of Stem per length foot: $W_c \times Stem_{height} \times Stem_{depth}$

$$\Rightarrow 0.15\frac{k}{ft^3} \times 6ft \times 1ft = 0.90\frac{k}{ft}$$

- DL of footing per length foot: $W_c \times \text{footing}_{height} \times \text{footing}_{depth}$

$$\Rightarrow 0.15\,\frac{k}{ft^3} \times 1ft \times 9ft = 1.35\,\frac{k}{ft}$$

- DL of Soil on Footing per length foot: $(\gamma_g \times \text{heel}_{Depth} \times \text{Stem}_{height}) + (\gamma_g \times \text{Toe}_{Depth} \times \text{Stem}_{height})$

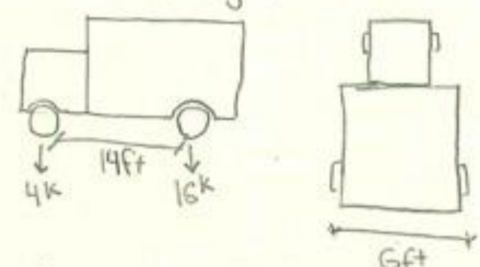
$$\Rightarrow \left(0.12\,\frac{k}{ft^3} \times 5ft \times 6ft\right) + \left(0.12\,\frac{k}{ft^3} \times 2ft \times 6ft\right) = 5.04\,\frac{k}{ft}$$

Design step 4 - Compute live load + Surcharge

- LL on bridge $\Rightarrow 0.54\,\frac{k}{ft} \times 130ft = 70.20\,K$
- Effective LL on abutment per length foot = LL ÷ (2 abutments × abutment length)

$$\Rightarrow \frac{70.20\,k}{2 \times 13ft} = 2.70\,\frac{k}{ft}$$

- Surcharge of H10 design vehicle



$$Q = \frac{16^k + 4^k}{14ft \times 6ft} = 0.24\,\frac{k}{ft^2}$$

Design step 5 - Limit state: Strength I

Load	Strength I load factor
Dead load	1.25
Live load	1.75
horizontal soil pressure	1.5
vertical soil pressure	1.35
Surcharge	1.75

modified loads

Stem DL $\Rightarrow 0.9\,\frac{k}{ft} \times 1.25 = 1.13\,\frac{k}{ft}$

Bridge LL $\Rightarrow 2.70\,\frac{k}{ft} \times 1.75 = 4.73\,\frac{k}{ft}$

Bridge DL $\Rightarrow 2.45\,\frac{k}{ft} \times 1.25 = 3.06\,\frac{k}{ft}$

Footing DL $\Rightarrow 1.35\,\frac{k}{ft} \times 1.25 = 1.69\,\frac{k}{ft}$

Vertical Surcharge $\Rightarrow 0.24\,\frac{k}{ft^2} \times 5ft \times 1.75 = 2.10\,\frac{k}{ft}$

Design Step 6 - Wall stability against overturning

- Lateral earth load: $\frac{1}{2} \times k_g \times \gamma_g \times$ abutment height$^2 \times$ Load factor

$\Rightarrow k_a$ (active lateral earth pressure coefficient) $\Rightarrow \frac{1-\sin\phi_g}{1+\sin\phi_g} = \frac{1-\sin 40}{1+\sin 40} = 0.36$

$\Rightarrow P1 = \frac{1}{2}(0.36)(0.12\frac{k}{ft^3})(7ft)^2(1.5)(1ft) = 1.57^k$

- Lateral surcharge: $k_g \times Q \times$ abutment height $\times$ Load factor

$\Rightarrow (0.36)(0.24\frac{k}{ft^2})(7ft)(1.75)(1ft) = 1.05^k$

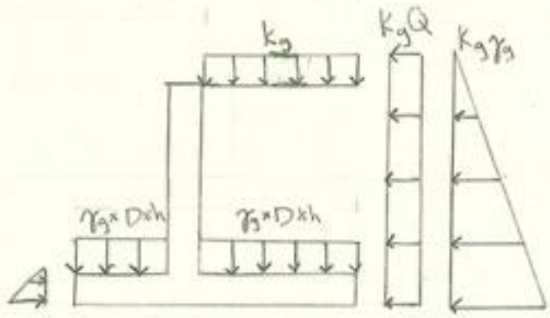
- Calculate restoring moment about the Toe

Load	K	Moment arm (ft)	Moment (k-ft)
Earth load @ heel	4.86	6.5	31.59
Earth load @ Toe	2.7	1.5	4.05
footing DL	1.69	4.5	7.61
Stem DL	1.13	3.5	3.95
Bridge LL+DL	5.85	3.5	20.48

Total = 67.68 ← All forces calculated using Strength I modification factors

- Calculating overturning moment about the toe

Load	K	ft	moment (k-ft)
Lateral earth load	1.57	2.33	3.66
Lateral surcharge	1.05	3.5	3.67



- Check resistance against overturning

$$\Rightarrow \frac{M_r}{M_o} \Rightarrow \frac{67.68 \text{ k-ft}}{7.33 \text{ k-ft}} = 9.23 \Leftarrow \text{FS is sufficient} \checkmark$$

Design step 7 - Pressure under footing

- Sum of vertical forces $= 4.46^k + 2.7^k + 1.69^k + 1.13^k + 5.85^k = 16.23^k$
- Net moment $\Rightarrow M_r - M_o \Rightarrow 67.64 \text{ k-ft} - 7.33 \text{ k-ft} = 60.35 \text{ k-ft}$
- Eccentricity $\Rightarrow \frac{\text{footing length}}{2} - \frac{\text{Net moment}}{\text{Vertical force}} = \frac{9\text{ft}}{2} - \frac{60.35 \text{ kft}}{16.23^k} = 0.78 \text{ ft}$
- Max pressure at toe $(Q_{max}) \Rightarrow \frac{\text{Vertical force}}{\text{footing surface}} \times \left(1 + \frac{6 \times \text{eccentricity}}{\text{footing length}}\right)$

$$\Rightarrow \frac{16.23^k}{9\text{ft} \times 1\text{ft}} \times \left(1 + \frac{6 \times 0.78}{9\text{ft}}\right) = 2.74 \frac{k}{\text{ft}^2}$$

- Check against allowable bearing capacity
  - Q_{allow} for silty sands = 3 ksf

$$\Rightarrow \frac{Q_{allow}}{Q_{max}} = \frac{3 \frac{k}{\text{ft}^2}}{2.74 \frac{k}{\text{ft}^2}} = 1.09 \Leftarrow \text{FS is sufficient} \checkmark$$

Design step 8 - Resistance to sliding

- Coefficient of friction $(\mu) = \tan(\phi_b) \Rightarrow \tan(25) \doteq 0.47$
- Friction force $(F) =$ Vertical forces $\times \mu = 16.23 \times 0.47 = 7.57^k$
- Sliding force $(S) =$ Lateral earth force + Lateral surcharge

$$\Rightarrow 1.57^k + 1.04^k = 2.61^k$$

- Check resistance $\Rightarrow \frac{F}{S} \Rightarrow \frac{7.57^k}{2.61^k} = 2.90 \Leftarrow \text{FS is sufficient} \checkmark$

Design Step 9 - Reinforcement

- Following equations were taken from LRFD Bridge Design Specifications

Eqn 1 · $M = \phi A_s f_y \left(d - \frac{a}{2}\right)$

Eqn 2 · $A_s = 0.85 \frac{f'_c}{f_y} a b$

ϕ = Concrete resistance factor = 0.9
A_s = area of prestressed reinforcement
d = distance from extreme compression fiber to nonprestressed tensile reinforcement
a = stress block factor
b = Width of the compression face = 1ft

- Strain compatibility : $\frac{a}{d} = \frac{0.003}{0.003 + \varepsilon_{c1}} \beta_1$

$\Rightarrow$ Given a yield strength of 60 ksi $\varepsilon_{c1} = 0.002$ and $\beta_1 = 0.85$

$\Rightarrow a = 0.51d \Leftarrow$ Eqn 3

- Substitute Eqn 2 into Eqn 1 $\Rightarrow M = \phi\left(0.85 \frac{f'_c}{f_y} ab\right) f_y \left(d - \frac{a}{2}\right)$

- Substitute Eqn 3 $\Rightarrow M = \phi \; 0.85 \frac{f'_c}{f_y} \, 0.51 d b f_y 0.745 d$

Simplify using $f'_c = 3$ksi, $f_y = 60$ksi, $b = 12$in, $\phi = 0.9$

$\Rightarrow M = (0.9)(0.85)(3\text{ksi})(12\text{in})(0.51)(0.745) d^2$

$\Rightarrow M = 10.5 \frac{k}{in} d^2 \Leftarrow$ Eqn 4

flextural resistance

$\Rightarrow$ Using Eqn 1 $\Rightarrow M = \phi A_s f_y 0.745 d \Rightarrow M = 40.2^{ksi} A_s d \Leftarrow$ Eqn 5

Design Stem Reinforcement

- Solve for d using eqn 4 and setting flextural resistance equal to moment at the base of stem

$\Rightarrow$ Moment @ base of stem = (Lateral earth load $\times \frac{h}{3}$) + (Surcharge $\times \frac{h}{2}$)

$= 4.98^{k\text{-}ft}$

Plug in $\Rightarrow d = \sqrt{\frac{4.98 \text{ k-in}}{10.5}} = 2.39\text{in} \Rightarrow \underline{3\text{in}}$

Plug into Eqn 5 $\Rightarrow A_s = \frac{4.98\text{k-ft} \times 12\frac{in}{ft}}{40.2\frac{k}{in} \times 3\text{in}} = 0.5\text{in}^2 \Leftarrow$ Per foot of wall

- Using #6 bar: Area = 0.44in^2

- Determine spacing $\Rightarrow \frac{0.44\text{in}^2}{0.5\frac{\text{in}^2}{ft}} \times 12\frac{in}{ft} = 13$in spacing

Design Footing Reinforcement

- Set flextural resistance equal to moment where the heel meets the stem. This location will experience highest moment forces

$\Rightarrow$ Flextural resistance = (heel DL + Surcharge + earth load) $\times \frac{h}{2}$

$= 19.7$ k-ft

Plug in $\Rightarrow$ $d = \sqrt{\frac{19.7\text{ k-ft} \times 12\frac{in}{ft}}{10.5}} = 4.75$ in $\Rightarrow$ 5 in

Plug into eqn 5 $\Rightarrow$ $A_s = \frac{19.7\text{ k-ft} \times 12\frac{in}{ft}}{40.2\frac{k}{in^2} \times 3\text{ in}} = 1.18\text{ in}^2$ $\Leftarrow$ Per foot of wall

- Using #10 bar: Area = 1.27 in²

- Determine spacing $\Rightarrow$ $\frac{1.27\text{ in}^2}{1.18\frac{in^2}{ft}} \times 12\frac{in}{ft} = 11$ in spacing

- Check shear reinforcement requirements

Eqn 1 - $V = \phi V_c$ | V = Shear resistance = max shear in heel
V_c = Shear resistance of concrete
ϕ = modification factor = 0.9

Eqn 2 - $V_c = 0.0316\beta\sqrt{f'_c}\,bd$

- Substitute Eqn 2 into Eqn 1 $\Rightarrow$ $V = \phi\, 0.0316\beta\sqrt{f'_c}\,bd$

- Solve for d using: $\phi = 0.9$, $\beta = 2$, $f'_c = 3000$ psi, $b = 12^{in}$,

and max shear = heel DL + Earth load + Surcharge = 7880 lb

$\Rightarrow$ $7.88\text{K} = (0.9)(0.0316)(2)\sqrt{3\text{ ksi}}\,(12\text{ in})\,d$

$\Rightarrow d = \frac{7.88^K}{(0.9)(0.0316)(2)(\sqrt{3\frac{k}{in^2}})(12\text{ in})} = 6.67\text{ in} \Rightarrow 7\text{ in}$

$\Rightarrow$ Plug in $\Rightarrow$ $M = 40.2\text{ksi}\,A_s d$ $\Rightarrow$ $A_s = \frac{19.7\text{k-ft} \times 12\frac{in}{ft}}{40.2\frac{k}{in^2} \times 7\text{ in}} = 0.84\text{ in}^2$

Max moment at heel

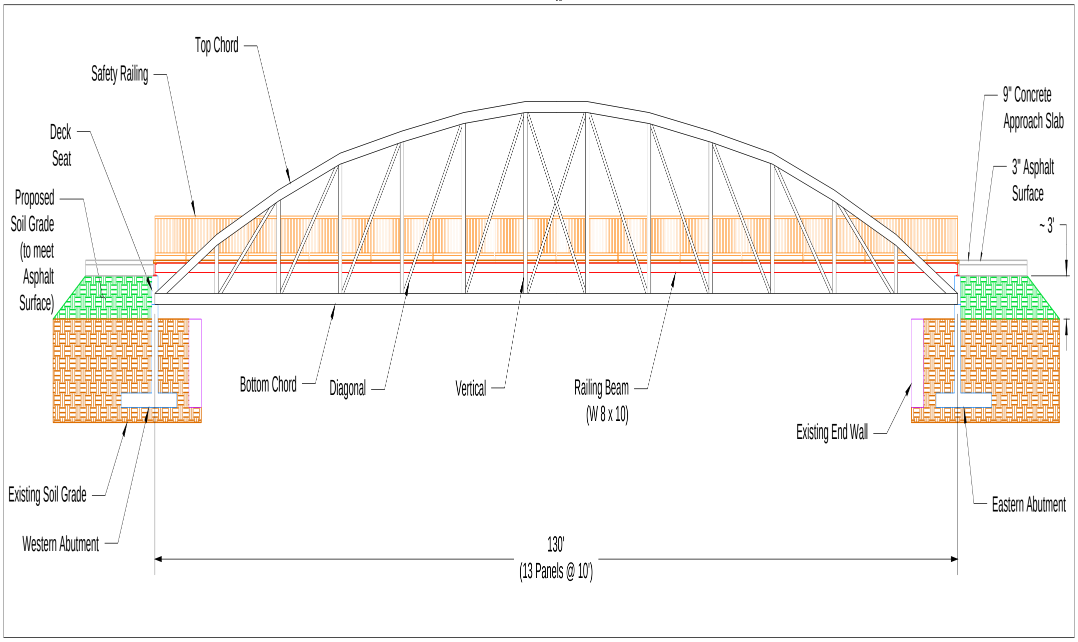
Required shear reinforcement is less than flextural reinforcement

$\therefore$ Use #10 bar @ 11 in spacing for footing

Appendix K: Bridge and Abutment AutoCAD Drawings

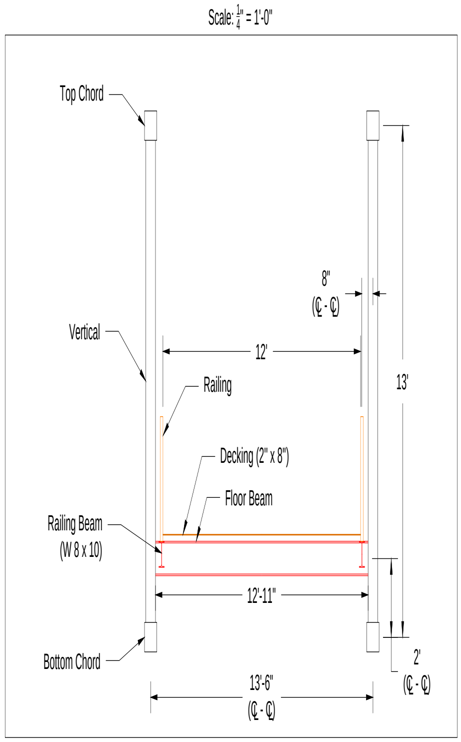
Note: The AutoCAD drawings placed in this appendix are not to the scales indicated on the sheets. These drawings are meant to be printed on 11’’ x 17’’ paper, and were reduced in size in order to fit on the 8.5’’ x 11’’ pages of this report. For properly scaled drawings, please refer to the Supplementary Files that were submitted along with this report.

ELEVATION VIEW
(of Bridge and Abutments)
Scale: $\frac{3}{32}$" = 1'-0"



BRIDGE SECTION
(Located in Center Panel)
Scale: $\frac{1}{4}$" = 1'-0"

SCHEDULE OF MEMBERS	
TOP CHORD	HSS 9 x 9 x 3/8
BOTTOM CHORD	HSS 9 x 9 x 3/8
VERTICAL	HSS 7 x 7 x 3/16
DIAGONAL	HSS 4 x 4 x 1/8
FLOOR BEAM	W 10 x 30
FLOOR DIAGONAL	W 6 x 12



NOTE:

HSS MEMBERS SHALL BE MADE OF A500 GRADE C, WITH YIELD STRESS OF 50 KSI AND TENSILE STRESS OF 62 KSI.

CONCRETE SHALL BE TESTED TO HAVE COMPRESSIVE STRENGTH OF 3000 PSI.

CONNECTIONS SHALL BE DESIGNED IN ACCORDANCE TO AISC STEEL CONSTRUCTION MANUAL, 15TH EDITION - SPECIFICATION CHAPTER K.

Structural Improvement Design for Northern Strand Community Trail Bridge over Saugus River

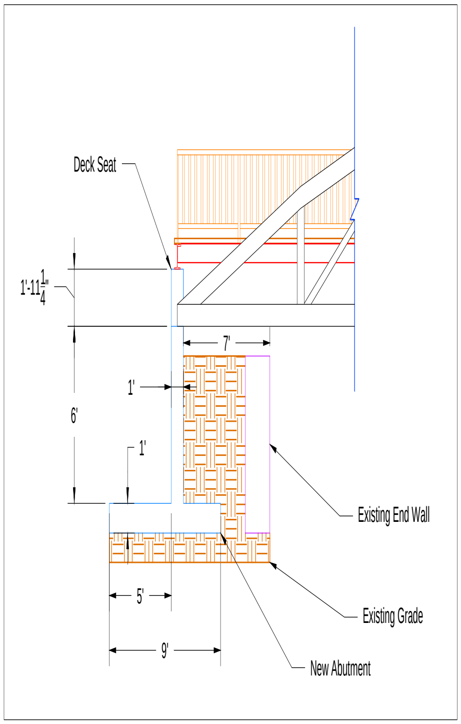
Team Members:
Sean Burke
Marlies de Jong
Ben Leveillee

Date: March 1, 2019

SHEET 1 OF 3

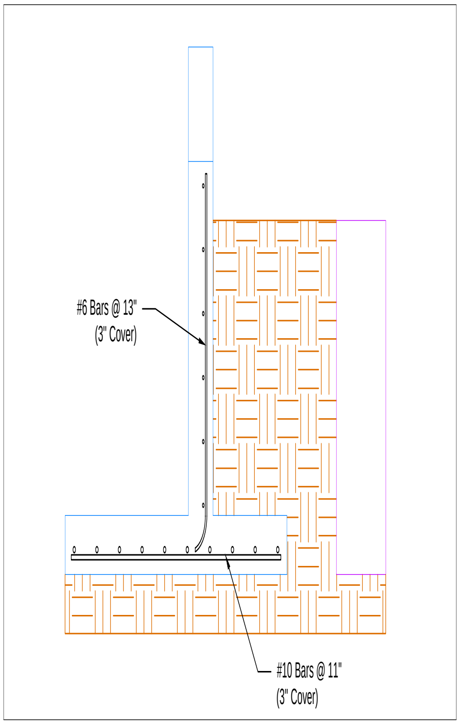
ABUTMENT DETAIL

Scale: $\frac{3}{16}$" = 1'0"



ABUTMENT REINFORCEMENT DETAIL

Scale: $\frac{3}{8}$" = 1'0"



NOTE:

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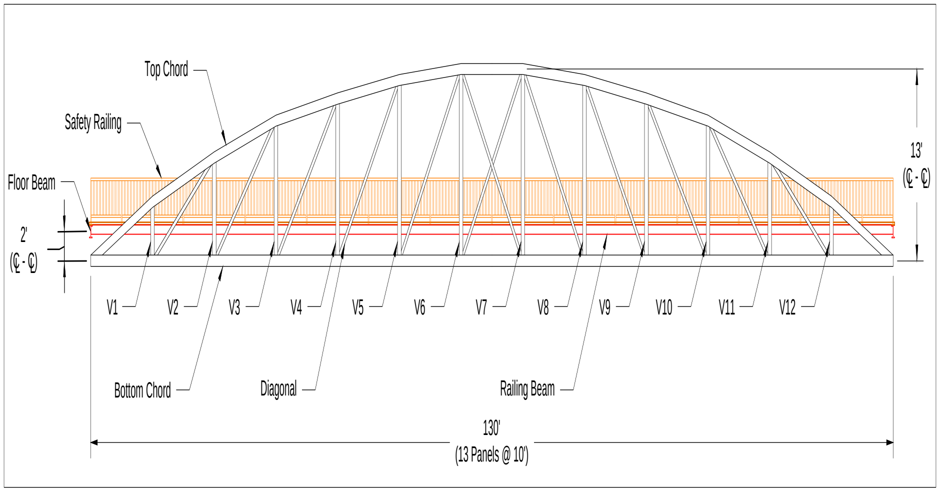
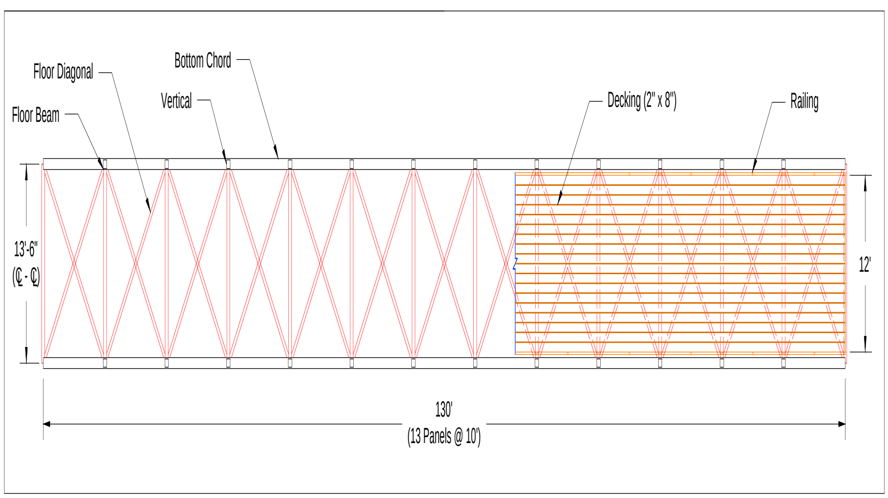
SHEET 3 OF 3

ELEVATION VIEW
(of Steel Superstructure)
Scale: $\frac{3}{32}$" = 1'-0"



VERTICAL DISTANCE ℄ - ℄ BET. TOP & BOTTOM CHORDS	
Vertical 1 (V1)	4'
Vertical 2 (V2)	7'
Vertical 3 (V3)	9.5'
V4	11'
V5	12.25'
V6	13'
V7	13'
V8	12.25'
V9	11'
V10	9.5'
V11	7'
V12	4'

PLAN VIEW
(at Deck Level)
Scale: $\frac{3}{32}$" = 1'-0"



NOTE:

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CONCRETE SHALL BE TESTED TO HAVE COMPRESSIVE STRENGTH OF 3000 PSI.

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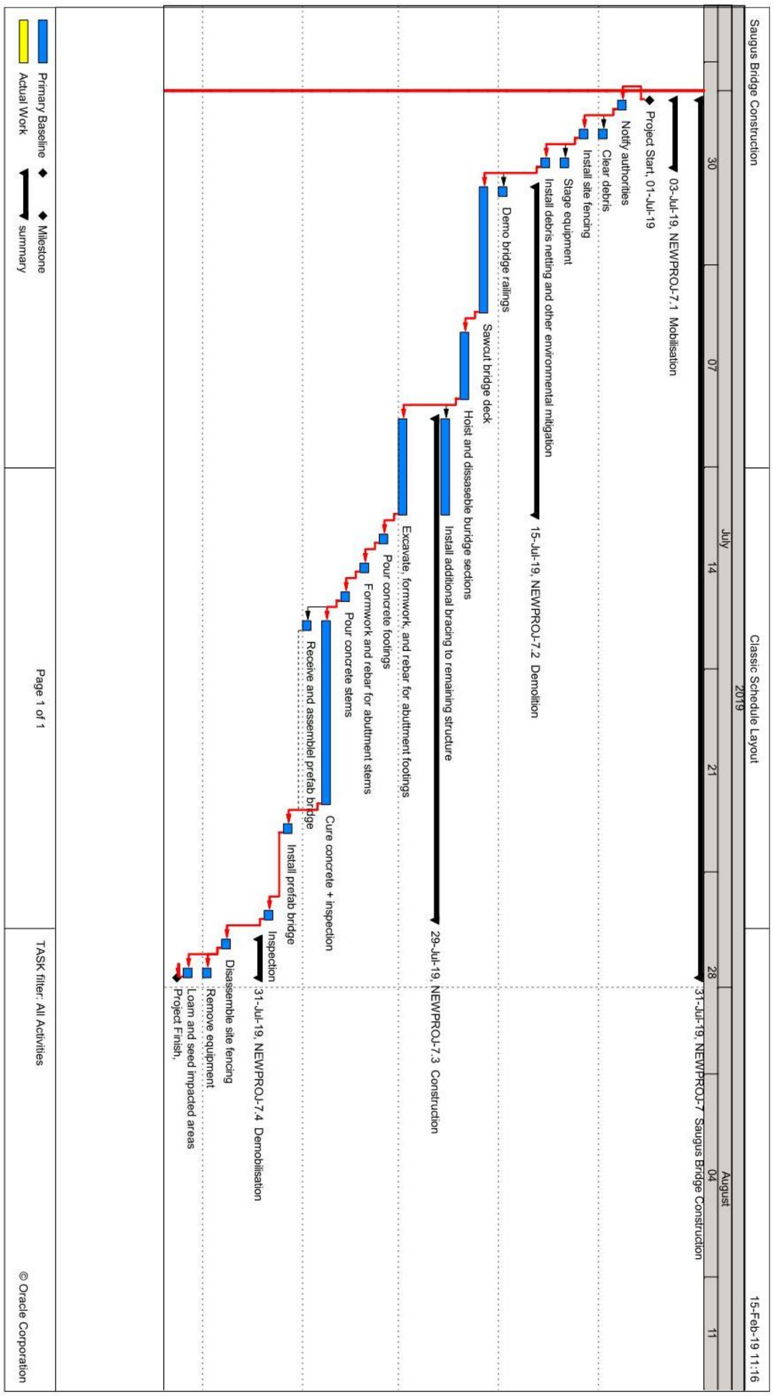
Ben Leveillee

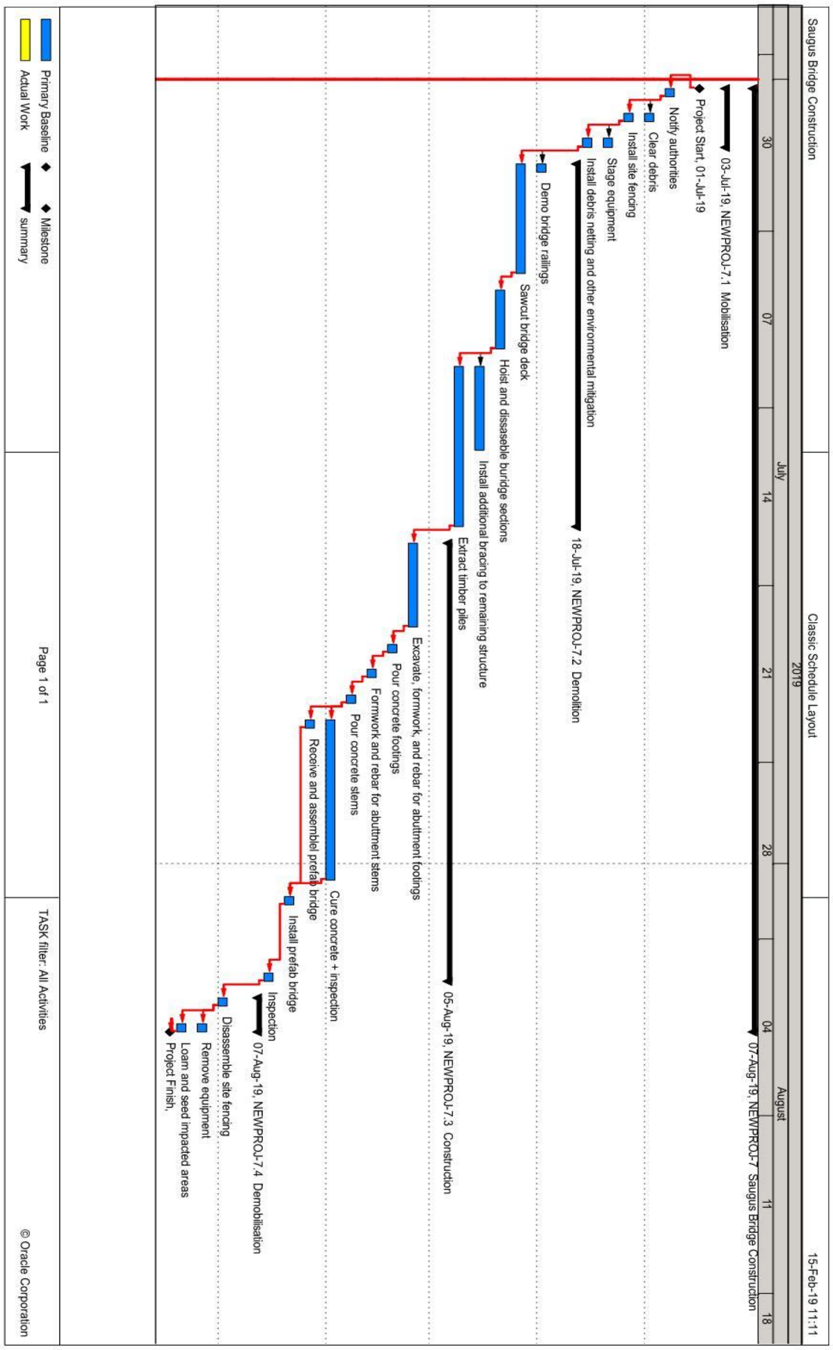
Date: March 1, 2019

SHEET 2 OF 3

Appendix L: Replacement Alternative A Schedule



Appendix M: Replacement Alternative B Schedule



Appendix N: Estimate Cost Item Breakdown

Bridge Demolition A

	Unit	Cost	QTY		
50' snorkel crane with work platform	DAY	$ 350.00	3	$	1,100
Truck mounted hydraulic boom - 40 ton	DAY	$ 932.00	3	$	2,800
Wood Bridge Demo	SF	$ 13.60	1560	$	22,000
			Total	$	25,900
			20% Contingency	$	5,200
				$	31,100

Bridge Demolition B

	Unit	Cost	QTY		
40' snorkel crane with work platform	DAY	$ 294.00	3	$	890
Truck mounted hydraulic boom - 40 ton	DAY	$ 932.00	3	$	2,800
Wood Bridge Demo	SF	$ 13.60	1560	$	22,000
Diving crew	DAY	$5,000.00	5	$	25,000
Pile Removal Crew	DAY	$5,500.00	5	$	28,000
Dredging	CY	$ 380.00	120	$	46,000
			SUM	$	125,000
			20% Contingency	$	25,000
				$	150,000

PREFABRICATED TRUSS BRIDGE

	Unit	Cost	QTY	
Prefabricated Bowstring Truss Bridge	LS	$375,000.00	1	$375,000.00
Truck mounted hydraulic boom - 40 ton	DAY	$ 932.00	2	$ 1,864.00
Lattice boom crawler crane - 40 ton	DAY	$ 1,179.00	2	$ 2,358.00
Front-end loader	DAY	$ 455.00	2	$ 910.00
			SUM	$380,132.00
			10% Contingency	$ 38,013.20
			Total	$418,145.20

Item No. 151.1 GRAVEL BORROW FOR BACKFILLING STRUCTURES

Backfill is equal to excavation minus concrete

Material	Unit	QTY
Excavation	cy	130
Concrete	cy	25
Backfill	cy	105
SAY	cy	120

Item No. 120.1 UNCLASSIFIED EXCAVATION (CY)

Excavate existing abuttment walls plus
addition area to pour concrete footings

Per Abutment	Unit	QTY
Height:	ft	8
Width:	ft	14
Depth:	ft	15
Volume:	cy	62.2
TOTAL	cy	124
SAY	cy	130

Item No. 903 3000 PSI, 1.5 INCH, 470 CEMENT CONCRETE (CY)

Per Abuttment	Unit	QTY
Stem Height:	ft	6
Stem Width:	ft	12
Stem Depth:	ft	2
Volume:	cy	5.3
FTG Height:	ft	2
FTG Width:	ft	12
FTG Depth:	ft	7
Volume:	cy	6.2
TOTAL	cy	23.1
SAY	cy	25

Item No. 181.14 DISPOSAL OF HAZARDOUS WASTE (TON)

Members	Length (ft)	QTY	Unit Weight (lb/ft3)	Weight (lb)
Ties	10	80	55	22000
Piles	25	50	55	68750
Pile caps	24	10	55	13200
Girders	110	4	55	24200
		TOTAL	64 ton	
		SAY	70 ton	

Item No. 910.1 STEEL REINFORCEMENT FOR STRUCTURES - EPOXY COATED

Type	length per bar	# of bars	total ft	lb per ft	total lb
#10 footing	8	30	240	4.3	1032.0
#6 stem	7	24	168	1.5	252.0
				TOTAL	2568.0 lbs
				SAY	2600 lbs

Item No. 181.12 DISPOSAL OF REGULATED SOIL - IN-STATE FACILITY (CY)

Excavated soil is to be used on site for transition

Per Abuttment	Unit	QTY
Height:	ft	7
Width:	ft	13
Depth:	ft	15
Volume:	cy	101.1
TOTAL	cy	101.1
SAY	cy	120

<u>Item No. 181.12 DISPOSAL OF REGULATED SOIL - IN-STATE FACILITY (CY)</u>

Excavated soil is to be used on site for transition

Per Abuttment	Unit	QTY
Height:	ft	7
Width:	ft	13
Depth:	ft	15
Volume:	cy	101.1
TOTAL	cy	101.1
SAY	cy	120