Twin Cities Rail Trail Pedestrian Bridge Design

A Major Qualifying Project Report 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 purpose of this project was to evaluate an abandoned railroad bridge over Route 2 in Leominster, MA, and design structural improvements for Stantec Inc. Three redesign options were considered: repair of the existing structure, superstructure replacement, and full replacement. These options were evaluated based on aesthetics, material cost, constructability, and future impacts. The final recommendation selected was the full replacement. This design was further developed, including a demolition and traffic plan.

Executive Summary

The Twin Cities Rail Trail is a 4.5-mile long planned project that will connect the town centers of Fitchburg and Leominster in Massachusetts. The Massachusetts Department of Transportation (MassDOT) project will provide a protected, paved path for pedestrian and bicycle traffic along what is currently an abandoned segment of the Fitchburg and Worcester Railroad. This connection will promote sustainable transportation practices and is a project twenty years in the making (Dore, 2019).

The Twin Cities Rail Trail crosses directly over Route 2 in Leominster. Currently, there is an abandoned railroad bridge crossing the roadway. It was built in 1951 and officially abandoned in 2008. A pedestrian bridge will be necessary for the trail to be connected and cross Route 2.

The goal of this project was to learn the process of bridge design by analyzing multiple design options for a pedestrian bridge over Route 2 in Leominster and recommending the best option to the client. The following objectives outline the process for completing the project:

1. Evaluate Existing Conditions
2. Identify Design Criteria
3. Develop Design Options
4. Evaluate Design Options and Select Recommended Option
5. Develop Final Design and Recommendations

To assess the current condition of the bridge, the team evaluated the bridge through inspection reports, a site visit, and load rating calculations. This information, along with industry standards, and concerns specific to the project were used to establish design criteria for selecting a final design. Three design options were developed: a repair of the existing structure, a superstructure replacement, and a full structure replacement. Using evaluation criteria to score the options, the highest-scoring option was selected and further developed.

The **full replacement** option received the highest weighted total from the decision matrix, meaning it is the recommended option. Its longer span length allows for future Route 2 projects to occur without impacting the bridge structure (Figure I and Table I). Additionally, the width of the full replacement bridge will meet the trail design guidelines and allow the bridge to be easily integrated into the Twin Cities Rail Trail (Figure II).

Table I: Full Replacement Final Truss Geometry

	Span	Deck Width	Truss Height	Panel Spacing
Dimensions	140 ft	16ft	8.5 ft	10 ft

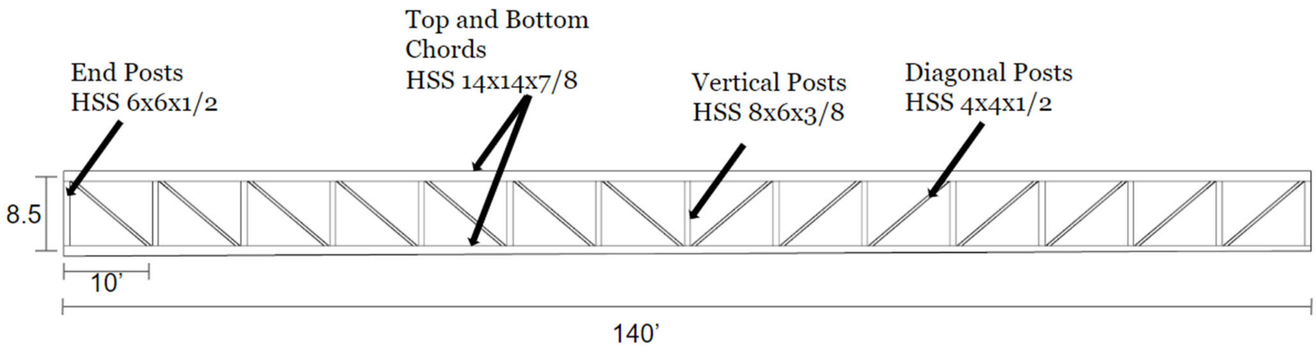

Figure I: Elevation View of the Full Replacement Superstructure Design

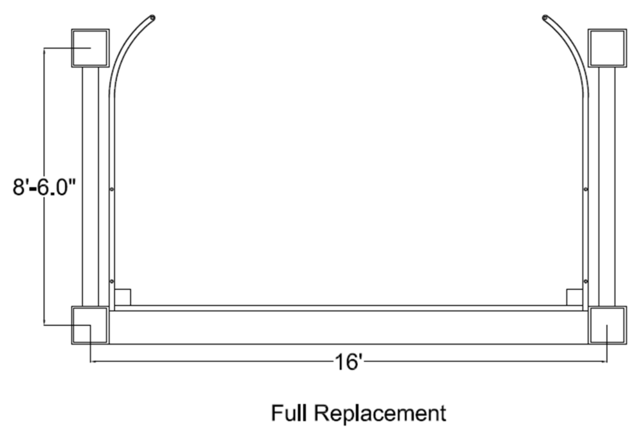

Figure II: Cross Section of the Full Replacement Superstructure Design

A final structural design, demolition plan, and construction and traffic management plan are included in the final recommendation for the full replacement design option.

The team recommends the full replacement of the existing bridge for the Twin Cities Rail Trail pedestrian bridge. If this design is adopted, the following should be considered before finalizing the design:

1. Gather new soil data, since the soil data used for the substructure was from a boring log produced in May 1936
2. Consider prefabrication and Accelerated Bridge Construction (ABC) costs and compare them to traditional construction method costs
3. Drive along proposed detour routes to check for impediments and weight limits
4. Discuss drainage plan with MassDOT and determine final spacing of drains, pipe sizes, and outlet path
5. Work with MassDOT to understand exactly what alterations will be made to the Route 2 interchange as to not constrain future plans

With the completion of these considerations, the pedestrian bridge design can be finalized, constructed, and integrated into the Twin Cities Rail Trail to service pedestrians.

Acknowledgements

We would like to acknowledge several people for their guidance and constructive criticism throughout the duration of the project. First, we would like to thank Worcester Polytechnic Institute for creating this project opportunity and allowing us to work in a professional environment. We would like to extend our gratitude to our advisors, Professor Leonard Albano and Professor Suzanne LePage for their valuable guidance and endless support in our project efforts. We appreciate their willingness to help and their consistent, quality feedback that contributed to our project. We further extend our gratitude to Stantec Consulting Services, Inc. for their valuable resources and workspace to complete the project. Also, we greatly appreciate all Stantec employees, especially Betsy Kirtland, Lauren Flanders, Erica Lotz, Rachel Santarsiero, and Frederick Moseley, for their guidance and valuable advice in completing this project.

Authorship

The team worked together to write the final report. All sections were reviewed and edited by all team members. The calculations were divided between group members, with an originator and helper, and a checker for the tasks. The division of the calculations is summarized in the table below.

	Originator	Helper	Checker
Existing Bridge	Nicole	Isabella	Hadi
Superstructure Replacement	Alex	Hadi	Nicole
Superstructure Full Replacement	Isabella	Nicole	-
Substructure Full Replacement	Jonathan	Alex	Isabella
Traffic Detour	Hadi	-	Alex
Demolition Plan	Alex	-	Isabella
Connection Design	Nicole	-	Isabella

Capstone Design Statement

This Major Qualify Project (MQP) for Worcester Polytechnic Institute (WPI) was completed by evaluating an existing railroad bridge and designing alternatives for replacement and rehabilitation to transform the structure into a pedestrian bridge. The selected alternative was developed into a detailed design. This bridge will be incorporated into the Twin Cities Rail Trail, and span Route 2 in Leominster, Massachusetts. The team designed multiple potential solutions for the pedestrian bridge and conducted structural calculations for the designs, in addition to considering the material cost, demolition, and traffic management for the project. In doing so, several design constraints were addressed: economic, environmental, social and political, ethical, health and safety, constructability, and sustainability. By considering these constraints in the design, this MQP satisfied the requirements for the Capstone Design Experience, as determined by the Accreditation Board for Engineering and Technology (ABET).

Economic

Economics is an important factor in determining if the construction project is within budget. The materials and methods selected for the project can help determine this cost and can be altered to find the most cost-efficient solution for the project. A material cost estimate was conducted to directly compare the different design options. Traffic, demolition, and construction costs associated with the pedestrian bridge were also considered to determine which design was the most cost-effective.

Environmental

The pedestrian bridge over Route 2 is connected to a trail surrounded by a wooded area. When considering bridge design and construction, solutions were promoted to mitigate the destruction of the natural habitat and material contamination. Due to limited space for construction equipment, the final design and construction proposals were made to minimize impacts on the environment and surrounding area. Drainage plans were conducted to properly divert and handle stormwater runoff from the bridge. The bridge and trail help to improve the public's relationship with the environment by encouraging people to walk and bike between Leominster and Fitchburg, instead of driving.

Social and Political

The construction of the bridge will serve as a connection between communities. During the design process, it was important to consider the social and political setting of the structure. Many people in the community have been waiting for the Twin Cities Rail Trail for decades and are very invested in the bridge. It was also important to be sensitive to local property owners and traffic in the surrounding area when designing the bridge, demolition plans, and traffic plans.

Ethical

The American Society of Civil Engineers (ASCE) Code of Ethics was followed throughout the duration of the project. It is an engineer's duty to be ethical and hold themselves to high standards as they directly impact the lives of people. It was necessary to ensure that all recommended designs are safe for public use.

Health and Safety

The health and safety of both the end users of the bridge and the laborers involved in its replacement were one of the main priorities in the design. The design and construction of the project must comply with the governing codes and laws. Recommendations for improving the pedestrian bridge ensured that the structure met or exceeded structural and serviceability requirements defined by the American Association of State Highway and Transportation Officials (AASHTO).

Constructability

Constructability was factored into the final pedestrian bridge design due to its restrictive location over Route 2. With Route 2 being heavily trafficked, materials for construction were selected for their ability to be easily shipped to the site and constructed in a short period of time. Custom members require more labor to install on site, an aspect that was considered due to the limited time windows for construction. The construction site also limited what vehicles could be considered for construction.

Sustainability

Different building materials were considered for their sustainability. Specifically, the team evaluated material durability to resist weathering. Maintenance costs and impacts were also considered.

Professional Licensure Statement

The National Council of Examiners for Engineering and Surveying states that professional licensure "protects the public by enforcing standards that restrict practice to qualified individuals who have met specific qualifications in education, work experience, and exams" (NCEES, 2019). In order to become professionally licensed in the United States, these qualifications must be met to ensure the safety and well-being of the public.

Becoming a Professional Engineer (PE) in the United States allows qualified individuals to certify, design, and sign off on engineering documents. Licensed engineers will be able to take on larger managerial roles in the industry and have more career opportunities to be a lead engineer on construction projects. In order to become a licensed Professional Engineer, the following requirements must be satisfied:

- Receive a four-year degree from an Accreditation Board for Engineering and Technology (ABET) accredited engineering program
- Pass the Fundamentals of Engineering (FE) exam to become an Engineering in Training (EIT)
- Complete four years of progressive engineering experience under a PE
- Pass the Principles and Practices of Engineering (PE) exam

Additional requirements may vary by state and can be found on the National Council of Examiners for Engineering and Surveying (NCEES) website.

A PE in Civil Engineering would be required to complete this MQP. Specifically, a PE would need to approve any structural designs and calculations performed by the team because the bridge structure can potentially impact the public's safety if calculations were not performed correctly.

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1.0 Introduction

The Twin Cities Rail Trail is a 4.5-mile long planned project that will connect the town centers of Fitchburg and Leominster in Massachusetts. This MassDOT project will provide a protected, paved path for pedestrian and bicycle traffic along what is currently an abandoned segment of the Fitchburg and Worcester Railroad. This connection will promote sustainable transportation practices and is a project twenty years in the making (Dore, 2019).

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2.0 Background

Railroads have long served as a common method of transportation. As certain railroads are abandoned or are no longer in use, cities can become disconnected. Now, there is an opportunity to reconnect the cities of Leominster and Fitchburg by using a former railroad but changing the mode of transportation. Instead of trains running between the communities, pedestrians will be able to travel between the cities by the means of the Twin Cities Rail Trail. With updates to its infrastructure, specifically a bridge over Route 2, the trail will be able to connect the cities again.

2.1 Proposed Trail Project

The Massachusetts Department of Transportation (MassDOT) is working to improve pedestrian transportation across the state. MassDOT's Pedestrian Transportation Plan identifies initiatives that address safety, critical gaps in connectivity, and accessibility in their pedestrian projects.

One of these initiatives is to consider pedestrian concerns more heavily when developing roadway projects. Typical pedestrian considerations are safety, comfort, and convenience, as stated in the first initiative of the plan. Additional concerns, such as a lack of lighting, are highlighted in Figure 1. Another goal being set by MassDOT is a year-round maintenance plan for pedestrian facilities that would encourage continuous improvement of safety and comfort for pedestrians (MassDOT 2019). MassDOT aims to support municipalities to implement maintenance plans, in order to ensure they are implemented.

Figure 1: Pedestrian Concerns (MassDOT Pedestrian Transportation Plan, 2019)

MassDOT is overseeing the construction of a proposed 4.5-mile long trail that will connect two city centers in Massachusetts: Fitchburg and Leominster. Currently, there are no pedestrian pathways between the cities. Called the "Twin Cities Rail Trail", the paved trail will be 12 feet wide with 2-foot shoulders, as depicted in Figure 2.

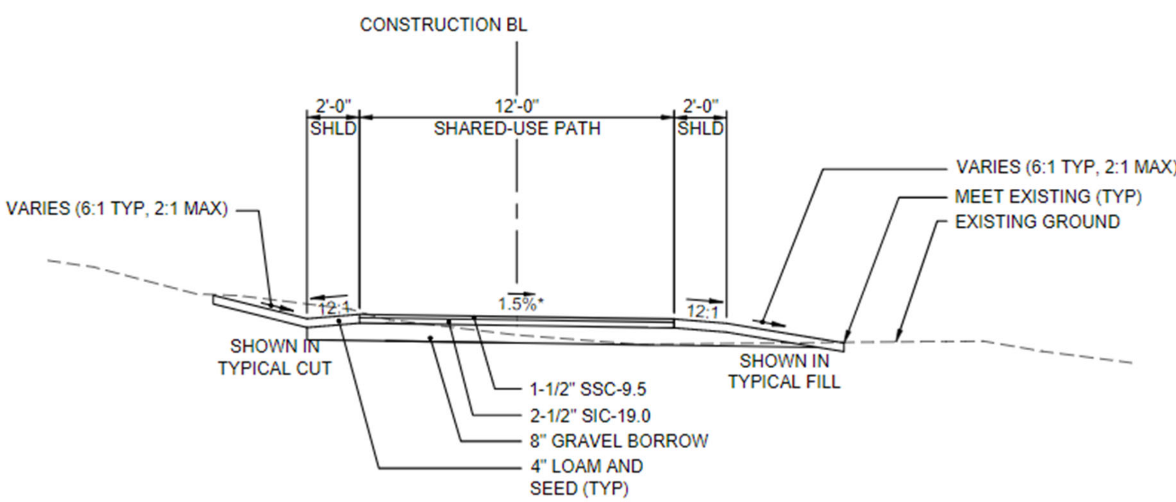

Figure 2: Typical Trail Cross Section (Stantec Drawings, 2019)

The trail will follow the abandoned commercial railroad corridor as shown in Figure 3. The goal of the trail is to "provide a non-motorized transportation and recreational alternative for people of all ages and abilities" (Dore, 2019), and to meet the initiatives set by MassDOT in the Pedestrian Transportation Plan. The project will be broken up into two phases, with the first phase of construction beginning in the Spring of 2020. The first phase of construction is outlined in red in Figure 3.

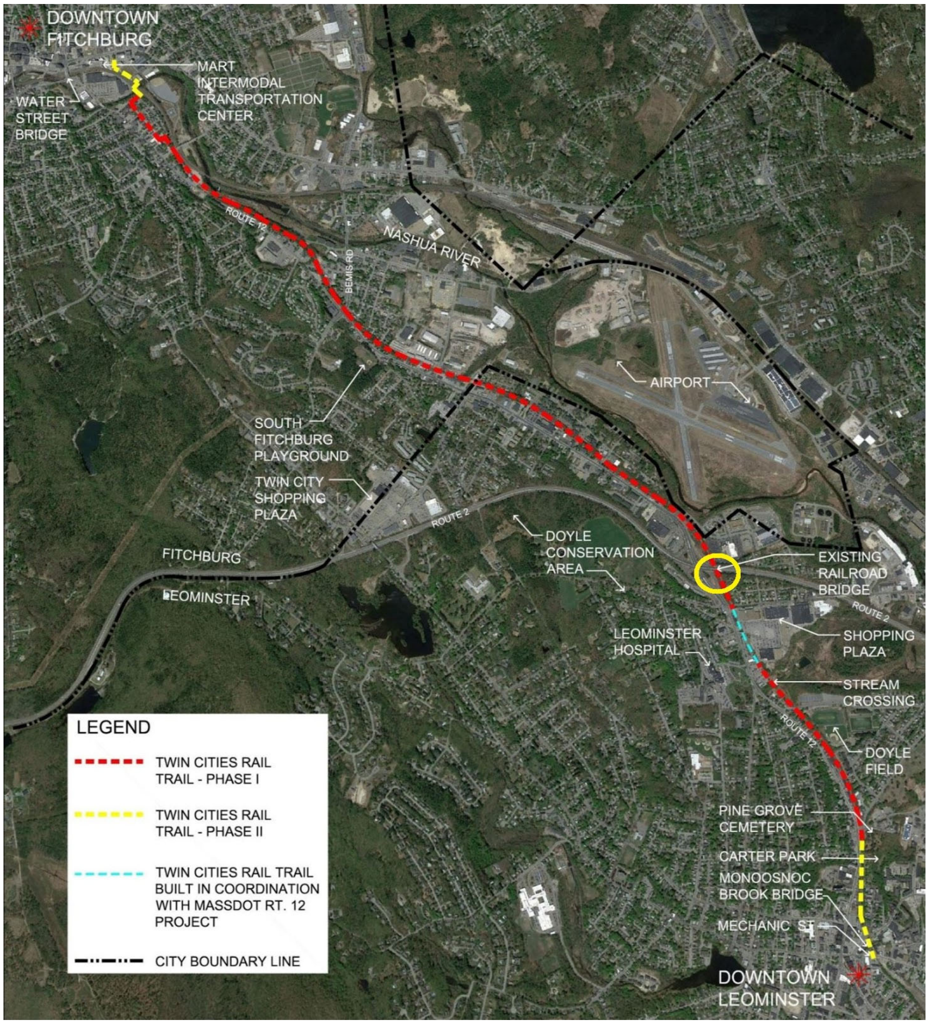

Figure 3: Complete Map of the Twin Cities Rail Trail (Core, 2019)

Included in the first phase of construction is the replacement or rehabilitation of the existing abandoned railroad bridge over Route 2 in Leominster, circled in yellow on Figure 3. The development of the bridge is essential in transporting pedestrians along the designated trail. The bridge is also located near an important junction of Route 2 with on-ramps and is seen by large volumes of commuters. The eastbound on-ramp has no significant acceleration lane, making it difficult for cars to merge onto Route 2. Based on past MassDOT highway updates, it is likely that this section of Route 2 will be expanded to include an acceleration lane and a third lane on Route 2 east. While not announced by MassDOT, it is possible that a third lane may also be added on the westbound side in the future, due to the large volumes of traffic Route 2 regularly sees.

2.2 Railroad and Bridge History

The Fitchburg and Worcester Railroad was established in 1840 to provide a rail connection between Fitchburg and Worcester. Service of the rails began on February 11, 1850, running 18 miles from Fitchburg through Leominster to Sterling Junction, and connecting with the Worcester and Nashua Railroad. The railroad was controlled by three different owners from its initial integration until the Surface Transportation Board approved the buyout of the Worcester and Fitchburg Railroad by CSX in 1998 (Revolvy, 2019).

The section of Route 2 spanning from Route 12 in Leominster to the Concord Rotary was constructed from 1950-1953 (Carr, 2007). According to the as-built plans, the bridge was constructed in 1951. CSX filed for abandonment of the 4.2-mile section of the railroad between Fitchburg and Leominster in 2008. This section includes a bridge crossing over Route 2, as seen in Figure 4.

Figure 4: Framingham and Worcester Railroad Bridge Over Route 2 in Leominster (Google Maps, 2019)

2.3 Impacted Area

Leominster used federal funds to purchase railway space, including that of the proposed bridge. Federal rules and regulations apply to the land purchased, meaning “abutting property owners who have used the land over the years must remove anything that they have placed there” (Sentinel and Enterprise, 2019). The acquisition of land has caused some local residents to feel uneasy about the encroachment of the trail onto their properties. Such encroachments onto properties require the city to pay for temporary and permanent easements. “In total, [Leominster] is expected to establish seven permanent easements and 24 temporary easements along the trail, which will extend the length of the old Fitchburg & Worcester Railroad line" (Busch, 2019). Fitchburg has approved one permanent easement and six temporary easements.

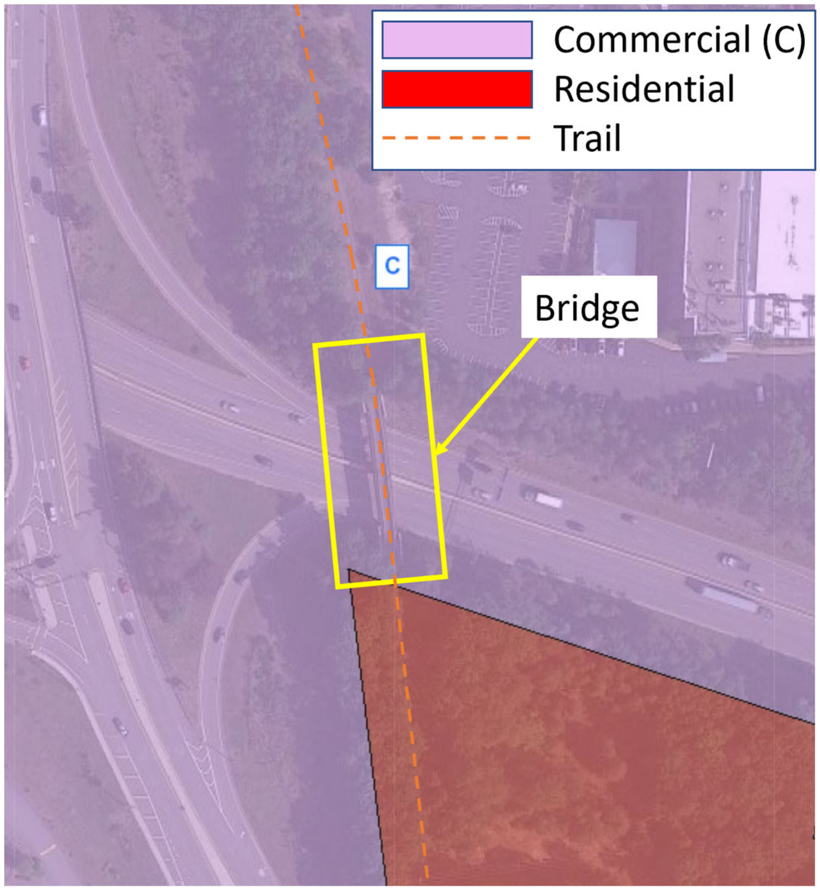

Figure 5: GIS Map Highlighting Zoning Areas Around the Railroad Bridge over Route 2 in Leominster (Leominster GIS, 2019)

The area surrounding the abandoned railroad bridge over Route 2 in Leominster is of particular concern to this project. Figure 5 outlines the bridge and how the land around it is zoned. This information is pertinent, since the rehabilitation or replacement of the bridge may require land near the bridge for construction staging. Figure 5 indicates that the majority of the land is commercially zoned with some residential zoning. This means that the town has identified these areas for growth of businesses and residences. The Twin Cities Rail Trail will provide pedestrian access to both of these areas. Recently, residents of the town, especially those who do not have land abutting the trail, have shown support for the trail.

2.4 Existing Conditions

The current structure being evaluated is a riveted plate girder bridge consisting of two simply supported single spans. Inspection reports that were conducted by MassDOT officials were sent to the MQP team by Stantec. The 2015 and 2017 Routine Inspection Reports identified and evaluated the integrity of the deck, substructure, and superstructure using a condition rating guide with a scale from 0-9, with 0 indicating the component failed and 9 indicating excellent condition. A score of "N" denoted an item that does not exist as part of the structure. Included in the rating guide is a deficiency rating guide with categories of deficiency (minor, severe/major, and critical) and urgency of repair (prioritize, ASAP, and immediate).

A summary comparison of the ratings from 2015 to 2017 can be seen in Table 1. A rating of at least a 6 indicates that the component is in a satisfactory or better condition. The only components that rated below a 6 were the parapets and paint, which were rated as fair and poor, respectively. Both the parapets and the paint repairs were indicated to be in severe condition and should be repaired as soon as possible. Differences between the 2015 and 2017 inspection reports are highlighted in yellow.

Table 1: Inspection Report Ratings Comparison Between 2015 and 2017 Inspection Reports

	2015	2017
Deck	**6**	**6**
Deck Condition	6	6
Parapets	N	5
Superstructure	**6**	**6**
Stringers	6	6
Floorbeams	6	6
Girders or Beams	6	6
Conn Plt's, Gussets & Angles	6	6
Bearing Devices	N	6
Rivets & Bolts	6	6
Member Alignment	8	8
Paint/Coating	4	4
Substructure	**6**	**6**
Abutments	6	6
Bridge Seats	*7*	*6*
Backwalls	*7*	*7*
Breastwalls	*6*	*6*
Wingwalls	*7*	*7*
Pointing	*6*	*6*
Settlement	*7*	*7*
Piers or Bents	6	6
Stems/Webs/Pierwalls	*6*	*6*
Pointing	*7*	*7*
Settlement	*7*	*7*

On the underside of the deck, moderate longitudinal hairline cracking with heavy efflorescence and efflorescence icicles were noted in the inspection report. At the top of the deck, sections that are up against the girders showed longitudinal and transverse cracking up to 1/16-inch-wide in many areas. Areas of minor to moderate map cracking and minor scaling with efflorescence to the wingwalls and breastwalls were indicated in the inspection report as well.

2.5 Construction Materials

There are several important factors to consider when selecting construction materials for a bridge design. The ability to prefabricate and easily assemble a structure addresses constructability concerns. The durability of certain materials may make them more appealing than others, especially in a variable New England climate. Vibration control is a concern during any bridge design, especially one for pedestrian use when small vibrations can be easily felt. Finally, the cost of materials is an important consideration for any project. A summary of the pros and cons of common pedestrian bridge materials is found in Table 2.

Prefabrication and Ease of Assembly

Recently, the prefabrication of materials, even entire structures, has become popular, especially for the construction of pedestrian bridges. Prefabrication offers an expedited construction process, since the parts of the bridge will be shipped to the site, and only need to be assembled. Accelerated Bridge Construction (ABC) often utilizes prefabricated components to help reduce the traffic impacts, onsite construction time, and weather-related delays that are usually encountered during traditional construction. ABC also improves total project delivery time, which is usually very appealing for communities (FHWA, 2019).

While the benefits of ABC are clear, the cost associated with both ABC and prefabricated components is typically higher than traditional construction methods. Components can be expensive to ship, depending on the section weight and size (Lin, 2017). If components cannot be prefabricated away from the site, materials such as concrete may be poured into forms in construction staging areas, and then placed. Each material has different limitations on its ability to be prefabricated and assembled.

Durability

The durability of a material is its ability to withstand the conditions for which it was designed, without compromising its structural integrity (PCA, 2019). Some materials, such as aluminum, are appealing because they have reduced susceptibility to corrosion during the service life of the bridge. Other materials such as wood or steel must be treated to avoid such degradation.

Vibration Control

Controlling vibration is important in all structures, especially pedestrian bridges. Pedestrians can easily feel structural vibrations and often find them unsettling. Since providing a serviceable structure is an important design goal, controlling vibration is necessary. Some materials provide little vibration resistance. Steel bridges exhibited generally higher responses to vibrations, with acceleration levels about twice as large as those exhibited by reinforced or prestressed concrete bridges (Gaunt). The use of aluminum for pedestrian bridges is increasingly becoming popular due to its high strength-to-weight ratio and reduced susceptibility to corrosion during the service life of a bridge. However, aluminum structures have low intrinsic damping and mass. As a result, they tend to be lively under operational loads and often exhibit large amplitude vibrations (Dey, 2015).

Cost

Cost is an influential factor for all projects. If materials are too expensive, it may not be feasible to construct the project. However, some more expensive materials are selected because their strength, durability, and vibration control are superior to other materials. Steel is often used even though it is more expensive than materials like concrete because it has a high strength to weight ratio and is a durable material, especially when compared to wood or aluminum. Concrete can be used with steel to create a composite material. In this case, the cost can be reduced by utilizing the ability of steel to handle tensile forces and the ability of concrete to resist compressive forces through composite action. While wood is one of the most inexpensive materials, it is generally only used for pedestrian bridges as it is not the most durable material, even when specially treated.

Table 2: Comparison of Construction Materials for Pedestrian Bridges Based on Four Main Elements

		Prefabrication and Ease of Assembly	**Durability**	**Vibration Control**	**Cost**
Aluminum	Pros	Low specific weight	Corrosion resistant and remains elastic under extreme cold temperatures	High elasticity and malleability reduce chance of brittle failure by excessive vibrations or deflection	Minimal maintenance reduces cost
	Cons	Larger member cross sections might be needed, complicating assembly	Prone to pitting corrosion by halide ions	Low Young's modulus creates members with low rigidity, members susceptible to deflection and vibrations	Most expensive
Concrete	Pros	Easily prefabricated or formed on-site	Highly adaptable to different environments using different compositions	High vibrational resistance	Widely used, low cost materials with various construction options
	Cons	Heavy sections and formwork impede construction	Susceptible to weathering and cracking, especially under freeze/thaw conditions	Continuous vibration conditions can reduce overall strength	Prefabrication, shipping and heavy members can increase construction costs
Steel	Pros	Variety of prefabrication options and low self-weight allows for easy assembly	Galvanizing increases weather resistance	High tensile strength controls deflections over long spans	Widely used and available, lower shipping and construction costs due to low member weights
	Cons	Fabrication is time consuming and not completed on site	Susceptible to weathering and rust formation	Low resistance to vibrational loads and effects (double the effect seen in reinforced concrete)	More expensive than concrete and timber
Timber	Pros	Light weight leads to easy transportation and assembly	Resistance to deicing agents and insects if treated properly	Good energy absorption and high strength to weight ratio	Less expensive than steel and concrete
	Cons	Timber frames may require additional time for design and fabrication	Low durability if unprotected from weather and insects	N/A	Maintenance and shorter life span can increase future costs

2.6 Bridge Types

There are many types of bridge designs and several different techniques that can be used to construct them. Common bridge designs include truss, arch, and simple girder. Techniques such as ABC and utilizing prefabricated bridges can be employed to improve upon the impacts from traditional construction methods.

2.6.1 Truss

A truss is a type of bridge with connected elements that form triangular units. Truss systems are used because of their rigid nature and their ability to transfer loads from a single point to a much wider area (History of Bridges, 2019). They use materials efficiently and effectively for the amount of load that may be carried, meaning the construction of a truss bridge is very economical. In a truss bridge, two long, usually straight members known as chords, form the top and bottom; they are connected by a web of vertical posts and diagonals. A truss will distribute stresses (tension and compression) throughout the structure, allowing the bridge to safely support vertical and lateral loads. A truss does not support the roadway below it, like a suspension bridge, or above it, like an arch bridge, rather, it makes the roadway stiffer and stronger, helping it resist the various loads acting on the structure (TDOT, n.d.). Figure 6 shows the configuration of a Pratt Truss and the stresses acting on the members.

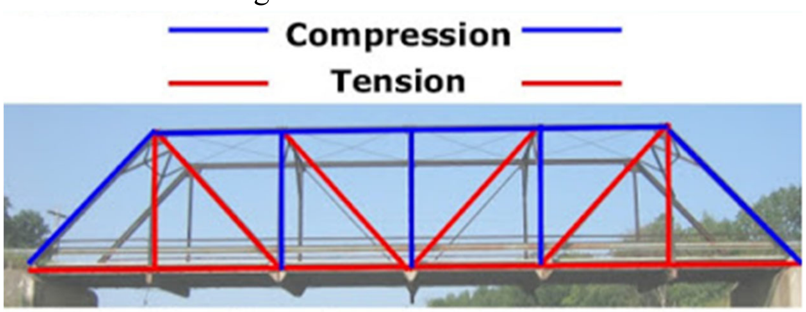

Figure 6: Diagram of Pratt Truss Bridge Forces (Holth, n.d.)

There are various types of truss bridges that are classified by their deck location and the geometric arrangement of their chords, vertical posts, and diagonals. Also, some truss configurations can carry loads differently. For example, a Warren truss is identified by its construction from equilateral triangles and is used due to its ability to carry distributed loads. A Pratt Truss, whose vertical members are in compression and diagonal members are in tension, is most effective when loads are in the vertical direction. Figure 7 shows the different configurations of truss bridges.

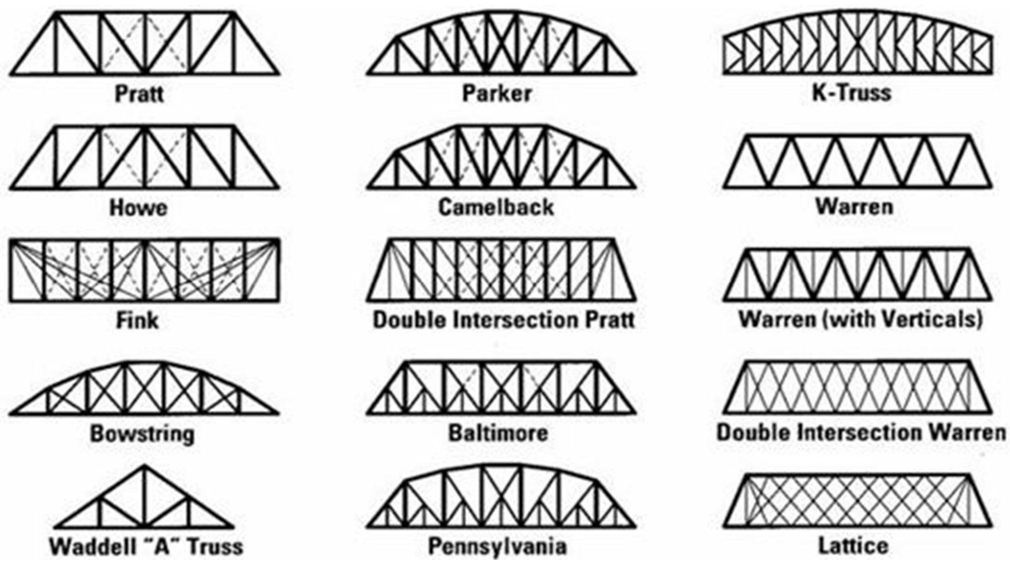

Figure 7: Types of Truss Bridges (Femia, 2013)

2.6.2 Arch

Arch bridges came into use over three thousand years ago and are still one of the most popular bridge types today. The weight of an arch bridge is carried outward along the curve of the arch to the supports or abutments at each end, instead of pushing straight down. This greatly reduces the effects of tension on the underside of the arch. The abutments carry the loads and keep the ends of the bridge from spreading out. Thus, the arch's semicircular structure distributes compression through its entire form (NOVA, 2000). Figure 8 shows the state of the arch under loading.

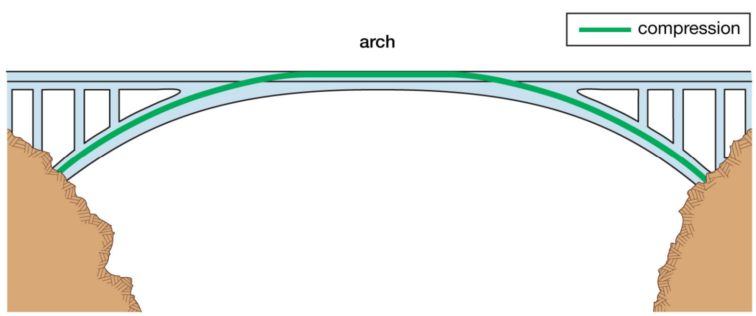

Figure 8: Diagram of Arch Bridge (Britannica, 2012)

2.6.3 Simple Girder

A girder bridge is the simplest, most common, and most inexpensive type of bridge. The bridge deck is built on top of the supporting beams, which are placed on piers and abutments that support the span of the bridge (Haskins, 2015). However, the girders must be able to support their own weight and any loads between the piers. Under loading, the beam's top surface is compressed, and the bottom edge is placed under tension (Goode, 2006). Figure 9 illustrates the internal forces acting on a girder bridge.

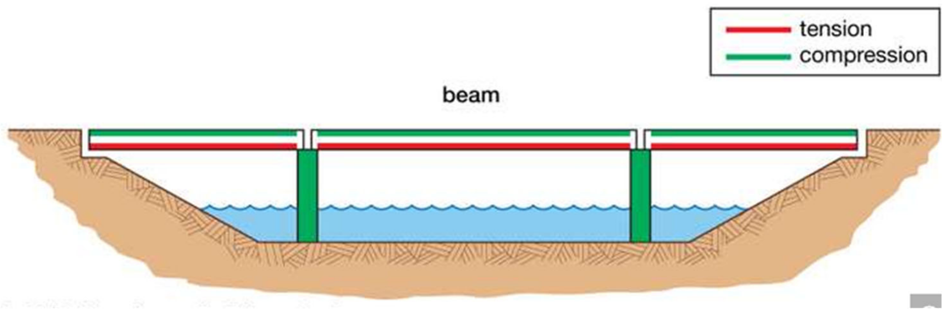

Figure 9: Diagram of Girder Bridge (Britannica, 2012)

2.7 Abutments

Abutments are designed to support the bridge deck so that the lateral and vertical forces can be safely transferred to the ground. The four primary types of abutments are full height abutments, stub abutments, spill through abutments (open abutments), and integral abutments. Less common types of abutments are mechanically stabilized earth and geosynthetic reinforced material. Abutments are typically made of concrete or stonemasonry (Rossow, 2012). The elevations and sections of the abutment types are illustrated in Figure 10.

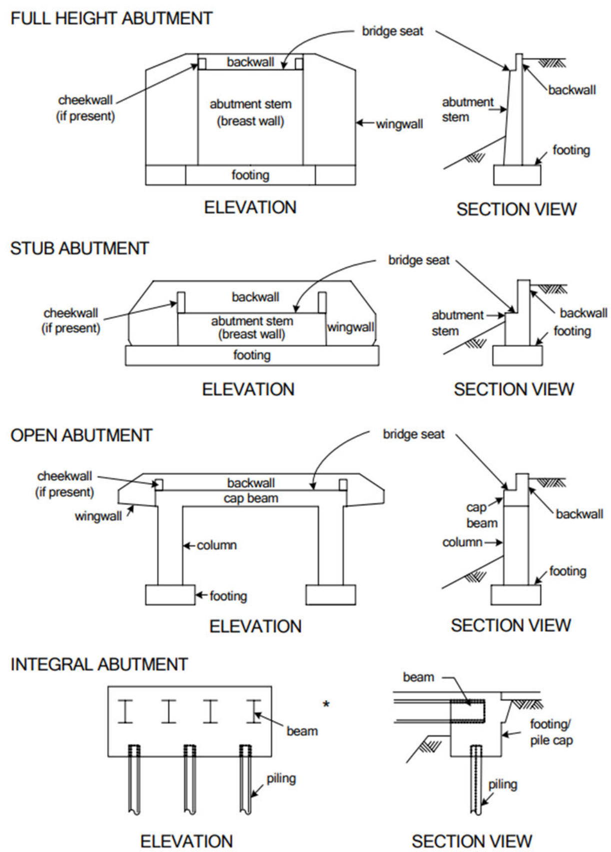

Figure 10: Types of Abutments and Their Components (Rossow, 2012)

Abutment types provide different advantages and should be selected based on design and cost considerations for a project. For example, full-height abutments are generally used for shorter spans or if a right-of-way or terrain issue is present. These types of abutments reduce the initial cost of the superstructure. Stub abutments are used for longer spans and increase the cost of the superstructure but reduce the cost of the substructure.

2.8 Codes

The project was conducted in compliance with standard industry practices. Typical methods were followed in accordance with AASHTO and MassDOT specifications. The goal of these specifications is to guide design practices to achieve safe, serviceable, and constructible bridges. Serviceability is important in any bridge but is especially important for a pedestrian bridge, where users can feel vibrations and deflections of the structure. Making a bridge serviceable includes making pedestrian users comfortable by decreasing vibration effects. While the structure may be structurally sound with large deflections, pedestrians crossing Route 2 will not feel comfortable on the bridge. Constructability is important when considering inspectability, economics, and aesthetics. Table 3 shows the main codes and specifications used for the design of the pedestrian bridge.

Table 3: Main Codes and Specifications Utilized for Pedestrian Bridge Design

Main Codes and Specifications	**Purpose**
MassDOT Load and Resistance Factor Design (LRFD) Bridge Design	Massachusetts-specific guidelines
AASHTO LRFD Bridge Design 8th Edition	Superstructure and substructure design
AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges	Pedestrian bridge design
AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals 6th Edition	Reference for pedestrian bridge design
Manual for Bridge Evaluation	Existing bridge load development
Americans with Disabilities Act (ADA)	Compare against design to ensure pedestrian access for all
Federal Highway Administration Bridge Preservation Guide	Guidelines for bridge preservation
AISC Steel Construction Manual 15th Edition	Steel member design
NHI LRFD for Highway Bridge Substructure and Earth Retaining Structures Reference Manual	Substructure design
Design of Concrete Structures	Substructure design
Army Field Manual 3-34-343	Substructure design

3.0 Methodology

The MQP team met the goal of producing a recommended bridge design by completing the following objectives.

1. Evaluate Existing Conditions
2. Identify Design Criteria
3. Develop Design Options
4. Evaluate Design Options and Select Recommended Option
5. Develop Final Design and Recommendations

These steps were achieved by following the methodology described in the sections below.

3.1 Evaluate Existing Conditions

The existing conditions of the bridge were analyzed to evaluate the structural integrity of the bridge. The team accomplished this by visiting the site and conducting a visual inspection. The previous plans and inspection reports of the railroad bridge (Appendix B) were also used to develop the load capacity of the existing bridge.

3.1.1 Site Visit

The team visited the bridge site on October 28, 2019, with two Stantec engineers, Lauren Flanders, PE, and Betsy Kirtland, EIT. The team viewed the general condition of the structure and took pictures of the superstructure, substructure, and surrounding trail area. Traffic conditions, such as the volume of traffic and noise of traffic, were noted. The procedure in the field involved two group members taking notes on the bridge's condition, one member comparing the original bridge plans to the current bridge layout, and two members taking pictures for further analysis. Notes based on information provided by the Stantec engineers, observations made by the team, and photos taken at the site are collected in Appendix C.

3.1.2 Load Rating Factors

The loads on the existing bridge were calculated in accordance with *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges.* The dead loads acting on the existing bridge, or the self-weight of the bridge and anything on it that remains stationary, were calculated using the as-built plans and assumptions from photos of the bridge. The weight of all the steel beams was multiplied by 1.06, to account for miscellaneous steel, such as stiffeners or connections (MBE, 2018).

The live load was taken as the maximum of either the pedestrian load or the vehicle load. The pedestrian load was taken as 90 pounds per square foot over the entire bridge. The vehicle load was taken as an H10 truck, based on the width of the bridge, as pictured in Figure 11. The pedestrian load governed over the vehicle load for the girders, and the vehicle load governed for the floor beams, so the appropriate loads were used in each of the load combinations and ratings.

Clear Deck Width	Design Vehicle
7 to 10 ft	H5
Over 10 ft	H10

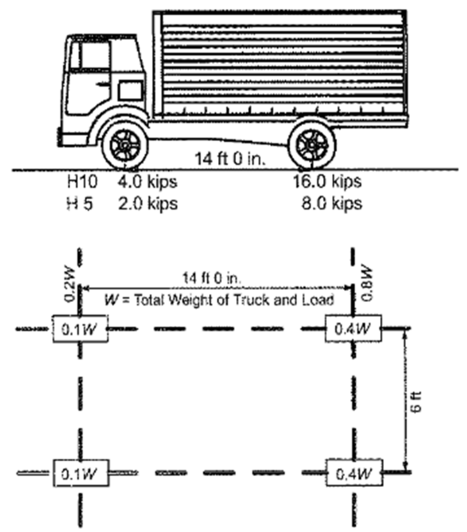

Figure 11: Design Maintenance Vehicle Loading and Configuration (AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges, 2009)

The load selection and calculations for equestrian loading, horizontal and vertical wind load, and fatigue load followed the process outlined in *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges*. Details of the calculations can be found in Appendix D.

Table 4: Load Combinations

Load Combination	Equation
Strength I	1.25*DC + 1.50*DW + 1.75*LL
Strength II	1.25*DC + 1.50*DW + 1.35*LL
Strength III	1.25*DC + 1.50*DW + 1.00*WS
Strength IV	1.25*DC + 1.50*DW
Strength V	1.25*DC + 1.50*DW * 1.35*LL +1.00*WS
Extreme Event I	1.00*DC + 1.00*DW + 0.50*LL
Extreme Event II	1.00*DC + 1.00*DW + 0.50*LL
Service I	1.00*DC + 1.00*DW + 1.00*LL + 1.00*WS
Service II	1.00*DC + 1.00*DW + 1.30*LL
Service III	1.00*DC + 1.00*DW + 1.75*LL
Service IV	1.00*DC + 1.00*DW + 1.00*WS
Fatigue I	1.75*LL
Fatigue II	0.80*LL

The load combinations were taken from *AASHTO LRFD Bridge Design Specifications* and can be seen in Table 4. The load combinations shown in red were not considered, per *AASHTO LRFD Design Specifications for the Design of Pedestrian Bridges,* as they will never govern in the case of pedestrian bridges. Although AASHTO did not specify against using Service III for pedestrian bridges, it was not applied to the steel superstructure, as Service III is used for checking tensile forces in prestressed concrete. The governing load factors from the combinations were then used for the load rating.

The maximum dead load and live load were found for each of the four girders that support the bridge and for the floor beams. From these loads, the moment at the center of each span was taken, as this was the governing scenario. The plastic capacities of the beams and girders were calculated and multiplied by a condition factor and a system factor dependent on the conditions and design of the bridge, as specified in the *Manual for Bridge Evaluation.* The load rating factor equation can be seen in Figure 12 with an explanation of the variables in Table 5. A rating of at least 1.0 is needed for a bridge to be considered structurally stable and capable of supporting the loads applied to it.

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_{LL})(LL+IM)} \tag{6A.4.2.1-1}$$

Figure 12: Manual for Bridge Evaluation Load Rating Equation

Table 5: Manual for Bridge Evaluation Load Rating Equation Variables

Variable	**Description**
C	Capacity
γ_{DC}	Load factor for structural components and attachments
DC	Dead load of structural components and attachments
γ_{DW}	Load factor for wearing surfaces and utilities
DW	Dead load of wearing surface and utilities
γ_P	Load factor for permanent loads other than dead loads
P	Permanent loads other than dead loads
γ_{LL}	Evaluation live load factor
LL	Live load
IM	Dynamic load allowance (not applicable for pedestrian bridges)

3.2 Identify Design Criteria

The team developed design criteria prior to the design of the bridge. Criteria specific to the bridge and site were considered, such as required bridge length and foundation concerns. Since the pedestrian bridge will be part of the Twin Cities Rail Trail, trail-specific criteria were determined, to ensure that the bridge will fit with the trail's design. *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges*, *Americans with Disabilities Act (ADA),* and *MassDOT LRFD Bridge Manual* were referenced during the establishment of design criteria.

3.3 Develop Design Options

The team developed three design options for the pedestrian bridge. The team researched different design and construction methods and considered how they could be beneficial to each option. Chapter 2 of the *MassDOT LRFD Bridge Manual* was referenced when researching types of bridge construction. The team applied knowledge and research results about construction materials to determine the best material for each design option. Additionally, the team evaluated the material cost, traffic implications, and constructability for each design option. Cost estimates were completed using MassDOT average item values, material calculations from the option designs, and spreadsheets. All structural calculations were performed by an originator and a helper, and then independently checked by another team member. MathCAD was used to organize calculations and provide a platform to easily test different member sizes for designs. RISA-3D was used for structural analysis, and force values obtained from RISA-3D were input to MathCAD routines for iterative designs. AutoCAD was used to create drawings of cross sections, elevations, and other details.

3.3.1 Repair Existing Bridge

The team analyzed the option to repair the existing bridge by first looking at the existing structure. The inspection report, load rating, and site visit were referenced to better understand the structural integrity of the bridge and to determine the extent of repairs necessary. The *Federal Highway Administration Bridge Preservation Guide* was referenced when the team looked at substructure repair options. Based on the findings from the site visit, the team determined what repairs were needed on the bridge and calculated the new load rating (Appendix E) with the loads from the minimum required repairs on the structure. Additionally, the team considered the traffic impacts of repairs, such as the road space needed for lead paint removal. The repair costs were considered in a material cost estimate.

3.3.2 Replace Superstructure

Superstructure replacement required the team to consider both prefabricated bridge options and traditional construction options. The team also considered different materials for the superstructure replacement by considering the design criteria (Section 4.2 Identify Design

Criteria). Steel was ultimately chosen for the superstructure when compared to other common construction materials. The decision was made due to the strength-to-weight ratio of steel allowing a small section depth for the longer span length provided, since the original bridge was a two-span bridge, and the replacement design is a single-span bridge. The accessibility and constructability of steel was also appealing. Multiple bridge configurations were considered, and a Pratt truss design was chosen due to its strength, aesthetics, and availability for prefabrication. *AASHTO LRFD Design Specifications for the Design of Pedestrian Bridges* was followed during the structural design process.

The team analyzed a truss design for the superstructure replacement. Similar steps to load rating were followed as in Section 3.1.2 Load Rating Factors. Initial member sizes were selected based on the example truss shown in *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges*. These members were used for the initial calculations prior to the team using an iterative process to find member sizes that satisfied strength and serviceability requirements. The wind load was calculated by using *AASHTO Signs*, per *AASHTO Pedestrian Bridges*. The initial bridge design was established in RISA-3D to determine the loading on each member. Load combinations and member sizes were updated in RISA-3D, in conjunction with the designated MathCAD calculations, in order to complete the iteration process. The full structural design process can be found in Appendix F.

An Eigensolution analysis through RISA-3D was utilized to analyze the superstructure for both horizontal and vertical vibrations. According to *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges*, the frequency in the vertical direction must be greater than 3 Hertz to avoid the first harmonic, and the frequency in the horizontal direction must be greater than 1.3 Hertz. The frequency in the vertical direction was calculated using the bridge's dead load, while the lateral frequency analysis was completed using the bridge's self-weight and the horizontal wind load. The software analysis generated vibrations throughout the structure and allowed for each node to be tested with six degrees of freedom. Hand calculations only accounted for one degree of freedom for each node and produced overly conservative frequencies as a result. Therefore, the software analysis was selected due to its more accurate results and was used to determine if the superstructure design was adequate for vibration limits. In addition to deflection and vibration calculations, camber at the midspan of the bridge was determined in accordance with *ADA* specifications. An approximate 1% camber of the bridge span length plus 100% of the dead load deflection must yield a maximum 5% slope at bridge ends to meet *ADA* requirements (Excel Bridge, 2019). Once all strength and serviceability requirements were met with updated loads and member sizes, the model was rendered.

3.3.3 Replace Superstructure and Substructure

Design of the full replacement option began with the comparison of different types of bridges and materials for construction. The team utilized design criteria (presented in Section 4.2) to make the decision on style and material for the design. The team also considered prefabricated structures and traditional building methods for the replacement of the existing structure.

A similar process to Section 3.3.2 Superstructure Replacement was followed to obtain final member sizes and geometry for the superstructure (Appendix G). Initial member sizes and geometry were selected for the truss, with the increased span length prompting the team to increase the truss height to support the greater loads. These member sizes and the geometry of the structure were used to determine loads on the bridge and input load combinations into RISA-3D, using a 2D analysis. The team went through an iterative process to change member sizes and geometry in RISA-3D and MathCAD to ensure that the superstructure could satisfy strength and service requirements. Similarly, to the superstructure replacement option, an Eigensolution analysis through RISA-3D was used to analyze horizontal and vertical vibrations in the structure.

The substructure dimensions were established from the required road clearance and the truss dimensions (Appendix H). The team utilized suggestions from a U.S. Army Corps of Engineer field manual to help select initial abutment dimensions and adjusted them as needed to meet design criteria and structural needs. Soil properties were assumed to not have changed since the original boring sample from May 1936, which is included in the as-built plans. A two-dimensional modelling and analysis approach was used to design the abutment per foot of length. Moments and shear forces were calculated to prevent the abutment from sliding and overturning. A value of at least 1.5 had to be achieved for a factor of safety for both overturning and sliding. Values such as concrete to soil friction, internal friction angle of the soil, and unit weight of soil were selected based on suggestions from a table in *Design of Concrete Structures* (Darwin, 2016). Selected values are summarized in Table 6. Lastly, the soil bearing pressure was checked to see if it could withhold the pressure from the loads acting down on it.

Table 6: Unit Weights w, Effective Angles of Internal Friction, and Coefficients of Friction with Concrete for Two Types of Soil (from Design of Concrete Structures)

Soil	**Unit Weight w, pcf**	**Φ, deg (internal friction angle)**	**f (soil to concrete friction coefficient)**
Sand or gravel with silt mixture, low permeability	120-130	25-35	0.4-0.5
Silty sand, sand and gravel with high clay content	110-120	23-30	0.3-0.4

Steel reinforcement was designed and added to the abutment stem to control cracks due to expansion and contraction. AASHTO requires that the following equation be satisfied for minimum area of temperature reinforcement:

$$Area_{min} \geq \frac{1.30bh}{2(b+h)fy}$$

where b = least width of the component; h = least thickness; f_y = yield strength of steel

The reinforcement was placed along the faces of the stem per foot. Steel reinforcement was added in a spiral column under each truss bearing area to resist bulking in the concrete. Reinforcement was added to the footing to resist bending. The amount of steel required to resist bending in the footing was determined from a graph of moment capacity for rectangular sections in *Design of Concrete Structures*. Four 8-inch diameter weep holes were added to both abutments to prevent hydrostatic pressure build up against the abutment. One cubic foot of crushed stone was also added to the end of each weep hole to help prevent blockage.

The full replacement of superstructure and substructure will be costly and require a substantial amount of site work. The demolition and construction for such a design will be more challenging and have more traffic implications to Route 2 than the options to rehabilitate or replace the superstructure. Accelerated Bridge Construction (ABC) and potentially a prefabricated superstructure could be used to accelerate the speed of construction. ABC will be more costly than traditional construction, but the savings it will provide for traffic impacts may be worth the cost, especially on Route 2. A prefabricated structure could allow for a faster build than traditional construction.

3.4 Evaluate Design Options and Select Recommended Option

The team evaluated the design options by considering the evaluation criteria and applying a decision matrix. The team established the evaluation criteria to structure the decision-making process for selecting the best design option. The evaluation criteria were used to determine the scores each design option would receive and are defined as follows.

Aesthetics

The pedestrian bridge must serve as an appealing bridge over a heavily trafficked road, fit the image of the Twin Cities Rail Trail, and blend into the area. To fit the trail image, the bridge must be inviting to users. The continuation of a smooth surface, such as asphalt, onto the bridge, inclusion of lighting and a new railing system will contribute to the aesthetic appeal of the bridge. MassDOT specifies that designers should also consider how the public experiences a bridge:

1. “The overall view of a bridge and how it relates to its setting.
2. The personal experience of someone driving over or under a bridge.
3. The human level experience of a pedestrian walking over, under or beside a bridge.”

(*MassDOT Bridge Manual*, 2013)

When scoring the aesthetics of the bridge, the options were compared against one another and scored accordingly.

Cost

Cost is an important consideration for a pedestrian bridge project. If the cost of the bridge is too high, a community may not be able to afford it, therefore not getting the benefit of a pedestrian bridge. Keeping the cost of the project low, while still selecting quality materials, is a necessary part of design. However, some costs, such as those from ABC, will pay off quickly in the amount of traffic interruption they help to avoid for the community. The cost scores were based off of the comparison of the material cost estimates for each option, as well as assumed additional costs due to construction, labor and traffic.

Constructability

The construction of the bridge is one of the most crucial aspects of the project's completion. Route 2 presents challenges for construction. The highway is busy with traffic every day, meaning long-term construction or detours would disrupt thousands of motorists' commutes. Negative traffic impacts must be avoided as much as possible during the construction process. This means that expedited construction is encouraged for this project, so that traffic impacts are limited to a smaller time period. Scores were determined by comparing the potential construction options for each design with one another. Assumed length of time for construction and ability to employ accelerated forms of construction were considered during the scoring process as well.

Future Impacts

Several future improvements to the area around this bridge are planned or likely to occur. This encourages a flexible design that can be adapted to these changes. The bridge height must be at least equal to its current height to comply with state regulations. However, the bridge height currently controls the corridor, so with any replacement options, it will be beneficial to increase the clearance height of the bridge. Additionally, increasing the span and widening the abutments will enable lane-widening on Route 2 in the future. The design options were scored on their ability to accommodate for more lanes on Route 2 under the bridge.

Weights were assigned to each evaluation criterion based on their importance to the project, as determined by the team from input from Stantec engineers. The weights were defined on a scale of 1-3; a larger weight indicated a more impactful criterion. For example, future impacts was given the highest weight because widening Route 2, or other larger future plans, could require a rebuild for some of the design options in the future. The high weight reflects the importance of this criterion. Each design was then assigned a score on a scale of 0-5 for each criterion. A score of 1 represented that a design poorly satisfied a criterion, which in turn negatively impacted the design's feasibility. A score of 5 represented that the specified design had an excellent level of satisfaction for a criterion and positively impacted the design's feasibility. Table 7 shows a spreadsheet that was created for the decision matrix.

Table 7: Decision Matrix Template

Weights	Criteria	Design Options		
		Repair Existing Bridge	Replace Superstructure	Full Replacement
1	Aesthetics			
2	Cost			
2	Constructability			
3	Future Impacts			
	Weighted Totals			

Score	How well does it meet the criteria?
0	Does not meet criterion
1	Poor
2	Fair
3	Satisfactory
4	Good
5	Excellent

3.5 Develop Final Design and Recommendations

The recommended design was further developed. The team designed typical connection details, outlined a drainage system for stormwater runoff, created detours for traffic impacts during construction, and drafted a demolition plan.

3.5.1 Final Structural Design

Connections

Once the final member sizes and geometry were defined, typical connection details were designed for the selected bridge design. Welds were selected over bolted connections. While welds are initially more expensive, they require less maintenance in the future, and are more aesthetically pleasing than plates. For HSS welded members, Appendix K of the *AISC Steel Construction Manual* was used to check for added constraints, such as punching shear (Figure 13). The available strength of the weld in the center of the truss was calculated, and it was assumed that it acted as a gapped K-connection. The load from the vertical post was then applied in the available strength check. Table K3.2 in the *AISC Steel Construction Manual* was used to find the available strength. Only the chord wall plastification limit state was checked, as the other limit states did not apply due to the chords and branches being square HSS members. Table K3.2A was used to check the applicability of Table K3.2, and the design passed all the checks.

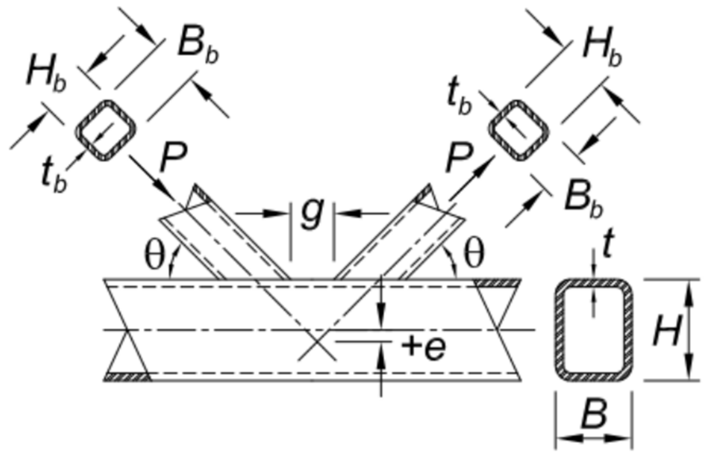

Figure 13: Connection Detail (AISC Steel Construction Manual)

Drainage

MassDOT LRFD Bridge Manual, Part II (2013) standard details were referenced to determine a drainage system for the pedestrian bridge. Camber and crown were added to the bridge to facilitate runoff toward the drains.

3.5.2 Demolition

The demolition plan was created based on discussions with professionals in the industry. Demolition of the existing superstructure and substructure must comply with the Occupational Safety and Health Administration (OSHA) demolition standards 29 CFR 1926 (OSHA, 1998). Such standards denote proper removal of hazardous materials and overall mechanical demolition, specifically applicable to crane usage. Site excavation estimates were determined utilizing existing condition plans and following sloping standards from 1926 Subpart P App B (OSHA, 1998). A Google Maps image of the bridge and the surrounding area was scaled in AutoCAD in order to test different crane sizes and the picking lengths required to remove the existing girders. Weights of the structural steel members for the existing bridge were previously calculated for the activities outlined in Section 3.1.2 and were used to determine necessary pick capacity of cranes examined. Different company's websites were then referenced to gather information on available crane sizes and their pick capacities based on pick length and boom heights. The cost of the demolition materials and labor were not included in the cost estimate for the final design.

3.5.3 Traffic

The team identified traffic impacts based on meetings and discussions with professionals at Stantec and available traffic data. Both eastbound and westbound traffic on Route 2 would be affected by construction and demolition, and detour solutions were necessary for each path. MassDOT's Transportation Data Management System (MassDOT, 2019) was used to find daily traffic counts for Route 2 and the surrounding roadways. This data allowed the team to identify possible detour paths for vehicles during demolition and construction. Google Maps was utilized to illustrate the possible detour routes and identify potential exits, interchanges, and on/off ramps. The length of detour paths, roadway capacity and condition, and geometric conditions (roadway width, layout, weight restrictions, and low clearance bridges) were considered in order to identify the quickest route for motorists. The team obtained intersection records and data regarding traffic volumes from the Traffic Operations Engineer at MassDOT's District 3 office and the Principal Planner at the Montachusett Regional Planning Commission. These records allowed the team to determine which detour paths would be reasonable for effective detours.

4.0 Findings

The goal of this project was to learn the process of bridge design by analyzing multiple design options for a pedestrian bridge over Route 2 in Leominster and recommending the best option to the client. This goal was met by achieving the following objectives.

1. Evaluate Existing Conditions
2. Identify Design Criteria
3. Develop Design Options
4. Evaluate Design Options and Select Recommended Option
5. Develop Final Design and Recommendations

The findings from objectives 1 through 4 are discussed in this chapter; those from objective 5 are presented in chapter 5.

4.1 Evaluate Existing Conditions

The team evaluated the existing conditions through a site visit, inspection reports, and load rating factors. Through these methods, the team found the characteristics of the existing bridge and site, and used this information to guide their designs.

4.1.1 Site Visit

The MQP team traveled to the site and visually evaluated the structural condition of the bridge. The notes from the site visit can be found in Appendix C. Overall, the bridge was in good condition. The main issues were the paint on the girders, the cracked areas on the substructure, and the collapsing fence and concrete barriers on the bridge as seen in Figures 14 and 15. The Stantec engineers who accompanied the MQP team informed the team that the paint is made with lead, so it would need to be removed and repainted during the bridge repair process. Additionally, the ballast is contaminated, so it must be treated as a hazardous material during removal. The girders and barriers on the bridge attract graffiti, much of which is offensive. Repainting the girders and parts of the abutments will help make the bridge more inviting and aesthetically appealing. The Stantec engineers reminded the team that salt from Route 2 is often thrown up against the abutments and increases the wearing of concrete, so the repair of cracked concrete is important to reduce negative effects of salt on the concrete.

Figure 14: Existing Bridge Conditions-Map Cracking on East Side of North Abutment and Peeling Lead Paint on East Side of Girder (Hamdan, 2019)

The team noted the low height of the girders on the bridge with respect to the pathway. Currently, there are barriers with a fence on the bridge, providing a higher rail height for pedestrians, as seen in Figure 15. With the barriers removed, the low height of the girders in comparison with the path height presents a risk for pedestrians.

Figure 15: Existing Bridge Railing on West Side of Bridge (Morrison Ouellette, 2019)

Structurally, the current bridge is composed of two simply supported single spans. The team was not able to see the entirety of the abutments, due to a fence restricting access to Route 2 near the bridge. The team relied on information from the 2015 and 2017 inspection reports and information from Stantec engineers to make structural determinations about the substructure and inaccessible parts of the superstructure.

The open area surrounding the bridge was somewhat limited based on the team's observations, but there were spaces for staging of construction vehicles. During the site visit, there were no obstructions behind the existing abutments, indicating that additional excavation to set new abutments back further from the road for a new bridge design is feasible. More pictures and notes from the site visit can be found in Appendix C.

4.1.2 Load Rating Factors

Strength I controlled for the load combinations. The load rating factor is a ratio between the capacity and the demand on the bridge. Both girders and floorbeams were rated for their moment capacity. The rating factors of the existing structure at various locations can be seen in Table 8. As these both rated above 1.0, the current bridge conditions, with the current dead loads and the required live loads, can safely carry the required loads. The detailed calculations are found in Appendix D.

Table 8: Load Rating Factors for the Existing Structure

Location	Pedestrian Load Case	H10 Truck Load Case
Floorbeam Midspan	19.86	10.92
Girder Midspan	2.18	4.49

4.2 Identify Design Criteria

The team established design criteria based on the specific needs of the project. The design criteria are summarized in Table 9, with a standard pedestrian fence detail shown in Figure 16.

Table 9: Design Criteria

Design Category	Numerical Design Criteria	Design Criteria
Bridge Length	≥115'	Same as current span for existing road width, greater than existing span for increased road width
Vertical Clearance	≥16.2'	Equal to or greater than current vertical clearance, FHWA encourages at least 1-foot greater clearance than the clearance needed for the corridor for pedestrian bridges
Foundation Design	-	Collision-force protected since foundation is <30 feet from roadway
Skew	26º	Skew must remain due to space constraints of site
Construction	-	Must be constructed quickly to reduce traffic impacts- prefabrication and accelerated bridge construction should be considered
Trail Width	≥12'	Must match typical trail width
Surface	-	Able to service non-motorized modes of transportation
Fence	See Figure 16	Must meet MassDOT standard details for pedestrian fence over highway

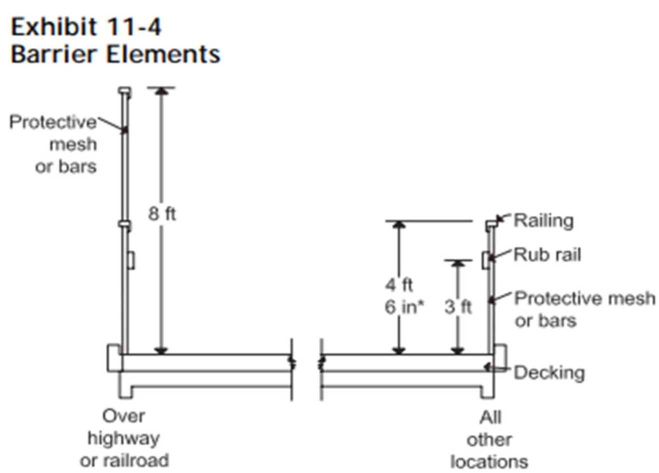

Figure 16: Pedestrian Rail Height (MassDOT, 2006)

Safety was not specified in the design criteria table, as all design options met or exceeded the safety requirements as directed by the codes and specifications referenced during design (outlined in Section 2.8 Codes).

4.3 Develop Design Options

The team developed three design options for the pedestrian bridge:

1. Repair Existing Bridge
2. Replace Superstructure
3. Replace Superstructure and Substructure

Each option was developed to address the design criteria within the constraint of each design. Table 10 compares the maximum forces due to the applied loads for each of the three design options. The DW Dead Load was not applied to the replacement options because all dead loads were considered to be structural components (DC) on the replacement designs.

Table 10: Comparison of Maximum Loads from Load Types Applied to the Three Design Options

Load Types	Repair		Superstructure Replacement		Full Replacement	
	Moment (kip*ft)	Shear (kip)	Moment (kip*ft)	Shear (kip)	Moment (kip*ft)	Shear (kip)
DC Dead Load	567.02	80.17	831.85	28.06	1674	47.82
DW Dead Load	122.84	17.37	-	-	-	-
Pedestrian Load*	262.21	36.57	1089	37.04	1799	50.9
Vehicle Load**	127.46	10.99	552	10.48	672	10.4
Wind Load	42.52	6.01	174.58	12.04	291.03	16.63
Fatigue Load	184.6	26.1	426.39	29.41	753.63	43.07

*Includes equestrian load
**Only used for floorbeams

The AASHTO load combinations are compared for the three design options in Table 11. For all three options, the Strength I load case, highlighted in yellow, governed. The Strength I results were then used to calculate the load rating factors for the existing structure. The Strength I load combination was also input into RISA-3D to analyze the superstructure for both replacement options.

Table 11: Comparison of AASHTO Load Combinations for the Three Design Options

Load Combination	**Repair**		**Superstructure Replacement**		**Full Replacement**	
	Moment (kip*ft)	Shear (kip)	Moment (kip*ft)	Shear (kip)	Moment (kip*ft)	Shear (kip)
Strength I	1352.00	190.30	5241.00	148.85	2922.00	99.90
Strength III	935.60	132.20	2383.00	76.41	1192.00	47.12
Extreme Event I	821.00	115.80	2573.00	73.27	1358.00	46.58
Extreme Event II	821.00	115.80	2573.00	73.27	1358.00	46.58
Service I	994.60	140.10	3764.00	115.35	2077.00	77.14
Service II	1031.00	145.10	4013.00	113.99	2229.00	76.22
Service IV	732.40	103.50	1965.00	64.45	988.00	40.10
Fatigue I	323.00	45.70	1319.00	75.36	746.00	51.46

4.3.1 Repair Existing Bridge

In order to repair the existing bridge to make it fit for pedestrian use, several issues must be addressed. The lead paint and contaminated ballast must be removed, a permanent fence needs to be erected along the length of the bridge, and the concrete abutments must be patch repaired.

Since the ballast is contaminated, it will be removed and treated as hazardous waste. After the ballast is removed, large concrete curbs will be visible. With the curbs, the existing bridge will have a width less than the required trail width. It may be necessary to remove these curbs in order to match the proposed trail. After removing the curbs, the 12-foot trail width can be maintained, however, there will be no shoulders. The bridge will be paved with a 2-½-inch layer of Superpave Intermediate Course 19.0 (SIC-19), and then covered with a 1-½-inch layer of Superpave Surface Course 9.5 (SSC-9.5), in accordance with the proposed trail plans. The final cross-section of the repaired bridge is shown in Figure 17. This pavement will help integrate the bridge into the trail and improve serviceability of the bridge. A fence will need to be added, in accordance with the standard detail in Section 4.2 Design Criteria. This fence will improve safety for pedestrians on the bridge.

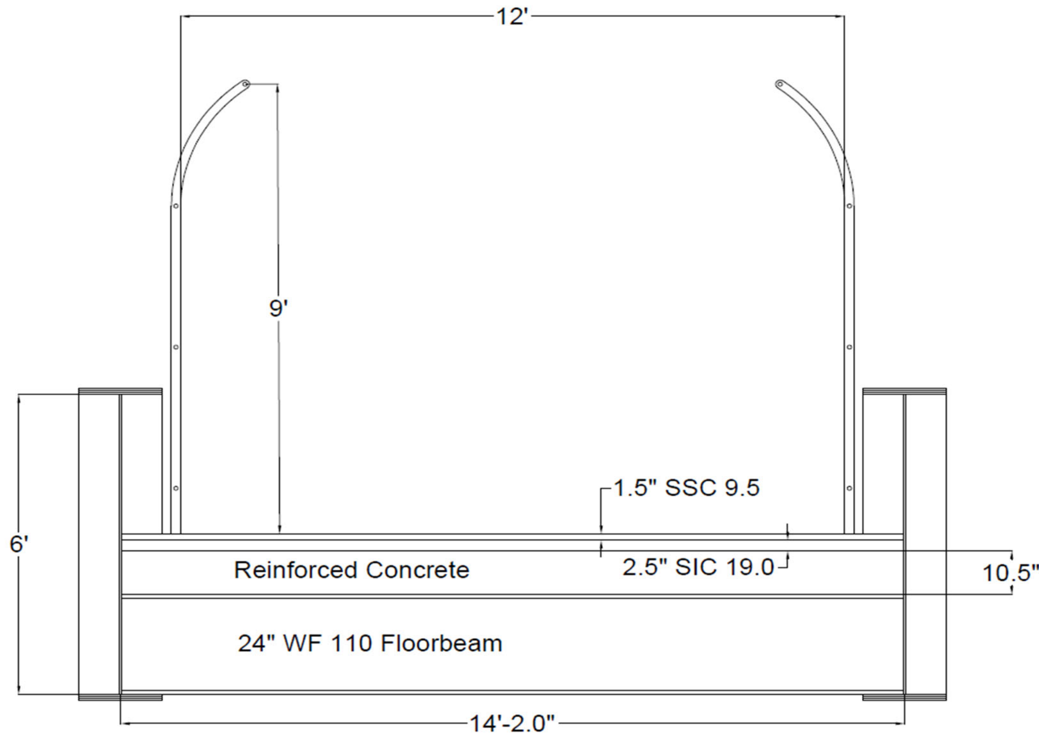

Figure 17: Existing Bridge Superstructure Repair Cross Section

The girders will have new unleaded paint, which will help with the aesthetic appeal of the bridge. An anti-graffiti paint may be beneficial to this bridge, as it has been a popular graffiti location. The lead paint removal process for this bridge will disrupt traffic because the lanes below the work area will need to be blocked off. Patch repairs may be done to the cracking concrete that is part of the substructure. Map cracking and scaling of the concrete can be chipped away and repaired to reduce the amount of salt and water infiltrating the concrete through the cracks. This will also require some amount of roadway to be closed during repairs.

With the removal of ballast and the addition of asphalt and fencing, the existing structure is still more than capable of supporting the new loads. As seen in Table 12, the load rating factors in all areas increased with the updated loading, as much of the dead load was removed from the structure. The calculations for the loading on the repaired bridge can be found in Appendix E.

Table 12: Load Ratings Comparison of Bridge Repair and Existing Bridge

Location	**Pedestrian Load**		**H10 Truck**	
	Repair	**Existing**	**Repair**	**Existing**
Floorbeam Midspan	22.31	19.86	12.27	10.92
Girder Midspan	5.04	2.18	10.37	4.49

4.3.2 Replace Superstructure

The superstructure replacement option allows for more aesthetic changes and serviceability updates than the repair option can offer. The bridge height will be increased, and cast-in-place concrete pedestals will be added to the abutments to increase the height of the bridge, so that it will not be the controlling member of the corridor. Raising the superstructure six inches will allow the clearance height to be larger than the adjacent bridge to the west, whose clearance height is 16.4 feet (National Bridge Inventory, 2018), providing more clearance height to vehicles on this section of Route 2. The pier will be cut down to a Jersey barrier height for the superstructure replacement, since it will not be necessary for the support of the new superstructure. This will help to provide a more open and inviting look to the bridge from the roadway. In addition, the current pier is in line with the concrete barrier, and therefore it is unprotected. There is little space to improve on this, so it is not beneficial to preserve it.

Figure 18 shows the final design of the superstructure replacement option generated through RISA-3D. This render shows the truss height, which extends to 7 feet, is much higher than the existing girders, providing a feeling of safety to pedestrians as they walk over Route 2. Additionally, the height helps with structural stability over the 116-foot span of the roadway.

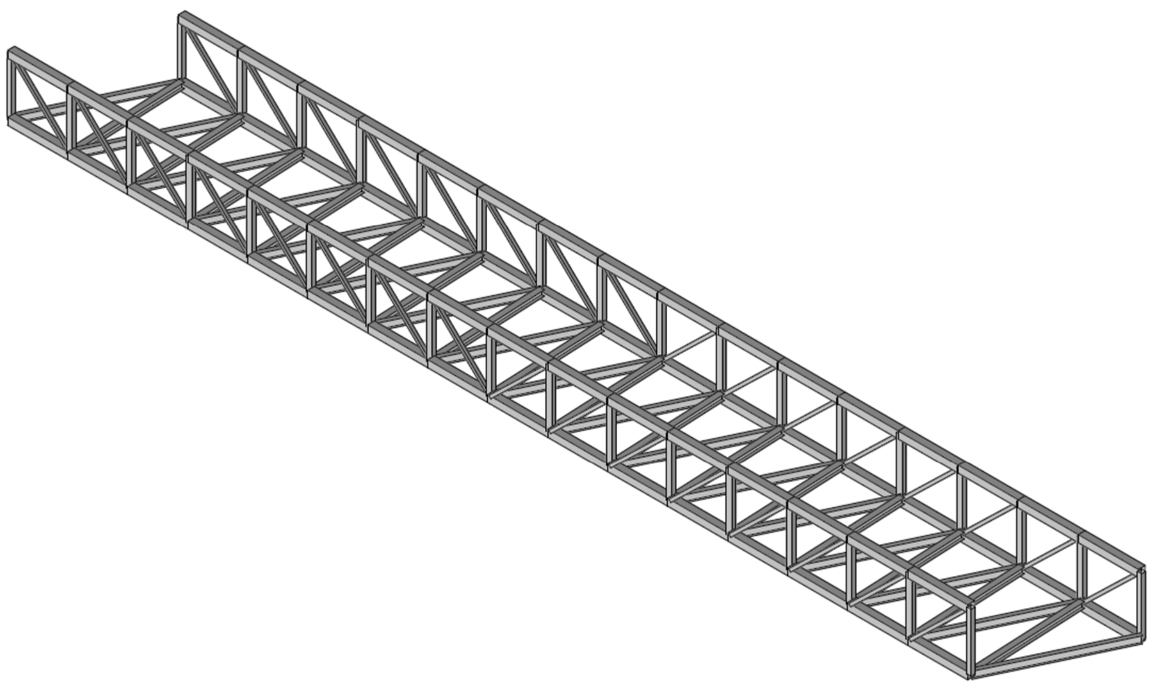

Figure 18: Isometric View of Final Superstructure Replacement Design

The unfactored loads applied to the bottom chords can be seen in Table 13. The live load, however, does not include the equestrian load, which was added on as a concentrated load of 1 kip in the center of the bridge. The associated load factors and limit states were determined according to *AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges*.

Strength I served as the governing load combination for structural analysis of the superstructure replacement. A factor of 1.25 was applied to the dead load and a factor of 1.75 was applied to the live load. Strength I generated a vertical load of 1.71 kips per linear foot and was applied to the bottom chords.

Table 13: Superstructure Replacement Applied Loads

Dead Load	0.484 klf
Live Load	0.630 klf
Wind Load	0.507 klf

The member sizes were established using an iterative design-by-analysis approach. The calculations for the final member sizes can be found in Appendix F. Table 14 shows the final superstructure geometry, and the final member sizes are found in Table 15. Table 15 also shows the capacity that each member can hold, and the demand placed on these members, as taken from the RISA model, based on the Strength I loads applied on the chords. HSS members were selected for the truss based on their structural, aesthetic, and maintenance performance.

Table 14: Superstructure Replacement Final Truss Geometry

	Span Length	Deck Width	Truss Height	Panel Spacing
Dimensions	116 ft	14 ft	7 ft	7.25 ft

Table 15: Superstructure Replacement Member Sizes and Capacities

	Member Size	Capacity	Demand
Top Chords	HSS 10x10x5/8	772.7 kip	406.4 kip
End Posts	HSS 6x6x1/2	59.3 kip*ft	6.21 kip*ft
Vertical Posts	HSS 8x6x3/8	77.3 kip*ft	12.37 kip*ft
Diagonals	HSS 4x4x1/4	152 kip	119.31 kip
Floorbeams	W 10x22	60 kip*ft	37.9 kip*ft
Diagonal Bracing	W 8x31	N/A	N/A

The vertical and horizontal deflections and vibrations of the superstructure were also calculated to meet serviceability requirements as shown in Table 16.

Table 16: Superstructure Replacement Serviceability Requirements

	Calculated	Required
Slenderness Ratio	52.658	<120
Horizontal Deflections	0.440 in	<2.784 in
Dead Load Deflection	1.501 in	N/A*
Live Load Deflection	1.954 in	<3.867 in
Horizontal Vibrations	2.596 Hz	>1.3 Hz
Vertical Vibrations	3.183 Hz	>3.0 Hz
Camber	1.29 ft	**

*Factored into vibration calculations

**1% of bridge span length plus 100% of dead load deflection

4.3.3 Replace Superstructure and Substructure

Replacing the superstructure and the substructure of the bridge will allow for the widening of Route 2 underneath the structure. This allows for more flexibility in future plans, removing the limitations on the road imposed by the existing abutments. Building new abutments also allows for increased clearance height below the bridge and a wider bridge deck.

A Pratt truss geometry was used for the new superstructure, similar to the superstructure replacement option, as seen in Figure 19. The full replacement option was designed to have a longer span length. This extra span of 24 feet provides space for up to two additional lanes along Route 2 under the bridge. The longer Pratt truss with new abutments will also provide an aesthetic final product that will be inviting to pedestrians and attractive to Route 2 traffic driving under it. The geometry of this superstructure is summarized in Table 17.

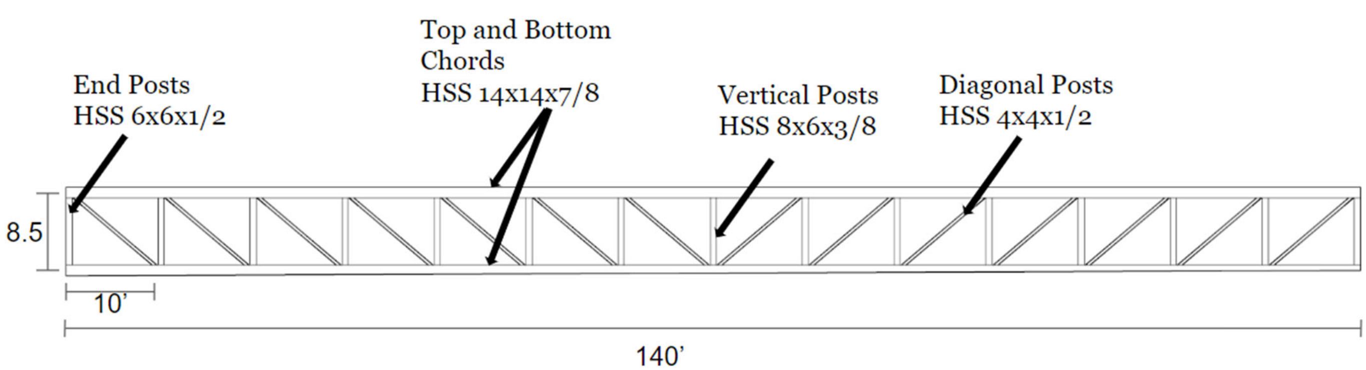

Figure 19: Elevation View of the Full Replacement Superstructure Design

Table 17: Full Replacement Final Truss Geometry

	Span	Deck Width	Truss Height	Panel Spacing
Dimensions	140 ft	16ft	8.5 ft	10 ft

The unfactored loads applied to the bottom chords can be seen in Table 18. Due to the longer span of the full replacement, the loading created higher demands than those for the superstructure replacement design. Similar to the superstructure replacement loads, the equestrian load was not included in the live load, instead a 1-kip concentrated load was applied to the center of the bridge. The truss and beam sizes are organized with their capacities in Table 19.

Table 18: Full Replacement Applied Loads

Dead Load	0.683 klf
Live Load	0.720 klf
Wind Load	0.615 klf

Table 19: Full Replacement Member Sizes and Capacity

	Member Size	Capacity	Demand
Top Chord	HSS 14x14x7/8	1418 kip	754.6 kip
End Post	HSS 6x6x1/2	59.33 kip*ft	14.35 kip*ft
Vertical Post	HSS 8x6x3/8	73.1 kip*ft	23.76 kip*ft
Diagonal	HSS 4x4x1/2	271 kip	248.5 kip
Floorbeam	W 12x50	192 kip*ft	40 kip*ft
Diagonal Bracing	W 10x45	N/A	N/A

The serviceability requirements were taken into account when designing the updated superstructure and can be seen in Table 20. Increasing floor beam and diagonal bracing sizes improved vibration performance of the full replacement superstructure, especially when loads were applied to the increased span length.

Table 20: Full Replacement Serviceability Requirements

	Calculated	Required
Slenderness Ratio	60.79	<120
Horizontal Deflections	0.487 in	<3.36 in
Dead Load Deflection	2.703 in	N/A*
Live Load Deflection	1.787 in	<4.67 in
Horizontal Vibrations	3.05 Hz	>1.3 Hz
Vertical Vibrations	3.02 Hz	>3.0 Hz
Camber	1.63 ft	**

*Factored into vibration calculations

**1% of bridge span length plus 100% of dead load deflection

Figure 20 shows the cross section of the superstructure for the full replacement. The 16-foot width of the bridge provides space for pedestrians and bikers to comfortably transverse the bridge, while still allowing room for curbs and fencing. See Appendix G for superstructure calculations.

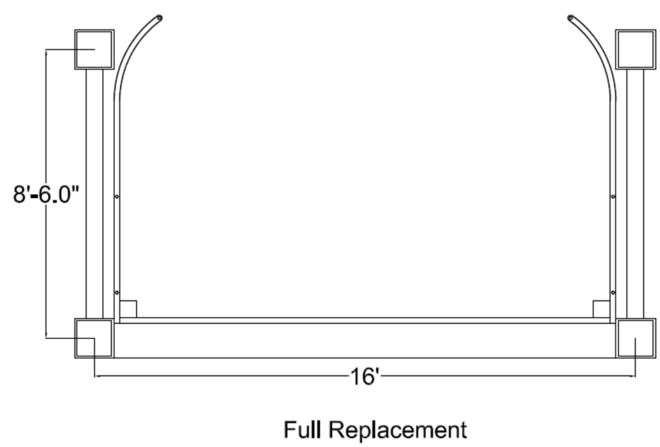

Figure 20: Cross Section of the Full Replacement Superstructure Design

The substructure was designed to satisfy the full replacement bridge design criteria and the governing pedestrian live load of 90 psf. The overall length of 20 feet for the bearing seat ensured that the skewed bridge can sit properly on the bearing area. The overall height of the abutment allowed for 3 feet of fill over the footings and the desired clearance height of 17 feet. The geometry of the gravity abutment influenced weights for the different tributary areas, and the final abutment section geometry can be seen in Figure 21.

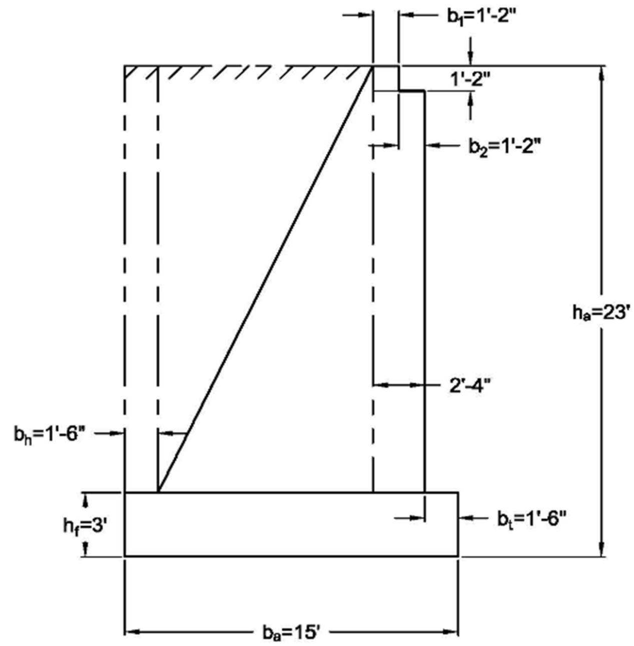

Figure 21: Final Abutment Section Geometry

Abutments were designed for temperature and shrinkage by placing adequate reinforcing in both transverse and longitudinal directions in the stem and footing, as seen in Figure 22. The abutment was designed for bulking stresses induced by the transfer of loads from the truss by placing spiral reinforcement under the bearing pads that extend down to the footing. The diameter of the column reinforcement is 16 inches.

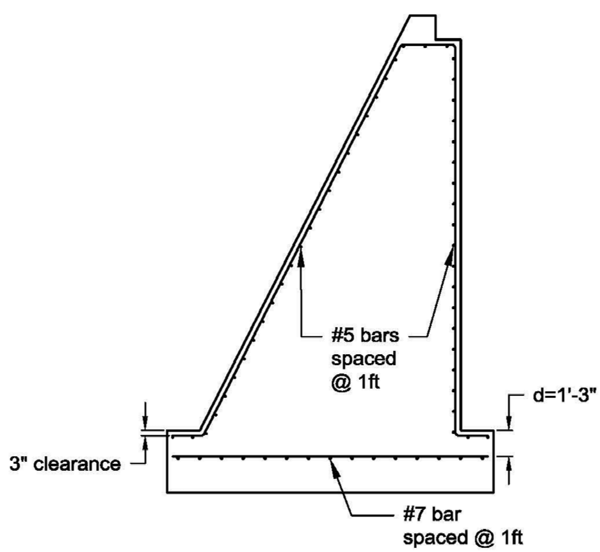

Figure 22: Abutment Reinforcement

Table 21 shows calculated safety factors for overturning moments and sliding between the concrete abutment and soil, with both considerations requiring a safety factor of at least 1.5, per *AASHTO LRFD Bridge Design Specifications*. The substructures maximum pressure on the soil was checked against the bearing capacity of the existing soil. See Appendix H for substructure calculations.

Table 21: Abutment Factor of Safety and Soil Pressure Checks

	Calculated	Required
Overturning Safety Factor	3.447	>1.5
Maximum Soil Pressure	5,258 psf	<8,000 psf
Sliding Safety Factor	1.535	>1.5

4.4 Evaluate Design Options and Select Recommended Option

Table 22 shows preliminary cost estimates of the three options based on the materials needed for each design. The breakdown and calculations from these can be found in Appendix I. While the repair option is the least expensive in terms of material cost, the other options offer some benefits that cannot be gained with the repair of the existing bridge. It should be noted that the demolition cost was not included in either replacement option cost estimate.

Table 22: Material Cost Estimates

Repair	Superstructure Replacement	Full Replacement
$926,132.30	$1,035,021.20	$2,129,811.18

The Federal Highway Administration estimates that pedestrian bridges range from $150 to $250 per square foot, totaling a cost of approximately $1 million to $5 million per complete installation (UNC, 2016). Based on the material costs of the designs, the final project will be on the higher end of this spectrum, after demolition costs for the existing bridge and labor costs for the new construction are factored into the estimate.

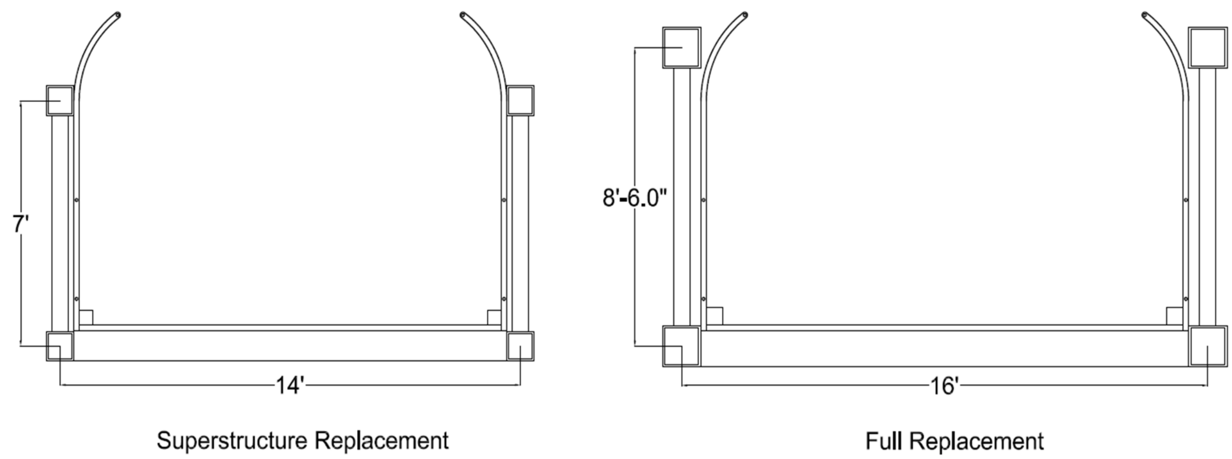

Figure 23: Superstructure Replacement and Full Replacement Cross Sections

Figure 23 shows a cross section of the full replacement design and the superstructure replacement design. The 8.5-foot truss height for the full replacement is due to the higher loads and wider panels than the superstructure replacement. With the full-replacement option, the width of the bridge is greater than the superstructure replacement option. This allows for curbing, the full 12-foot trail width, and shoulders. The superstructure replacement design was designed to fit on the existing abutments, so the width of the bridge was constrained to 14 feet. This does not allow for the same flexibility to add curbs and shoulders, while maintaining the desired 12-foot trail width. The repair option does not allow for a redesigned, more appealing superstructure. The pathway width is also limited based on the constraints of the existing superstructure and abutments.

Replacement of a superstructure over Route 2 would be disruptive to traffic. The use of Accelerated Bridge Construction (ABC) with a prefabricated bridge could reduce this impact significantly. In addition, night work could reduce traffic impacts. Keeping the existing substructure will help reduce overall costs, but limits future Route 2-widening projects.

The final decision matrix is seen in Table 23. The full replacement option received the highest weighted total, meaning it is the recommended option. The flexibility this design provides for future impacts is very important. Additionally, the width of the full replacement bridge is larger than that of the other options, meaning that it will be more user-friendly and can easily fit into the style of the Twin Cities Rail Trail.

Table 23: Final Decision Matrix

Weights	Criteria	Design Options		
		Repair Existing Bridge	Replace Superstructure	Full Replacement
1	Aesthetics	3	4	5
2	Cost	4	4	2
2	Constructability	4	3	2
3	Future Impacts	1	2	5
	Weighted Totals	22	24	28

Score	How well does it meet the criteria?
0	Does not meet criterion
1	Poor
2	Fair
3	Satisfactory
4	Good
5	Excellent

5.0 Final Design and Recommendations

Based on the previous findings and design matrix, the team chose to further develop the full replacement option (Figure 24) to present to Stantec. A final structural design, demolition plan, and construction and traffic management plan are included in the final recommendation for the full replacement design option.

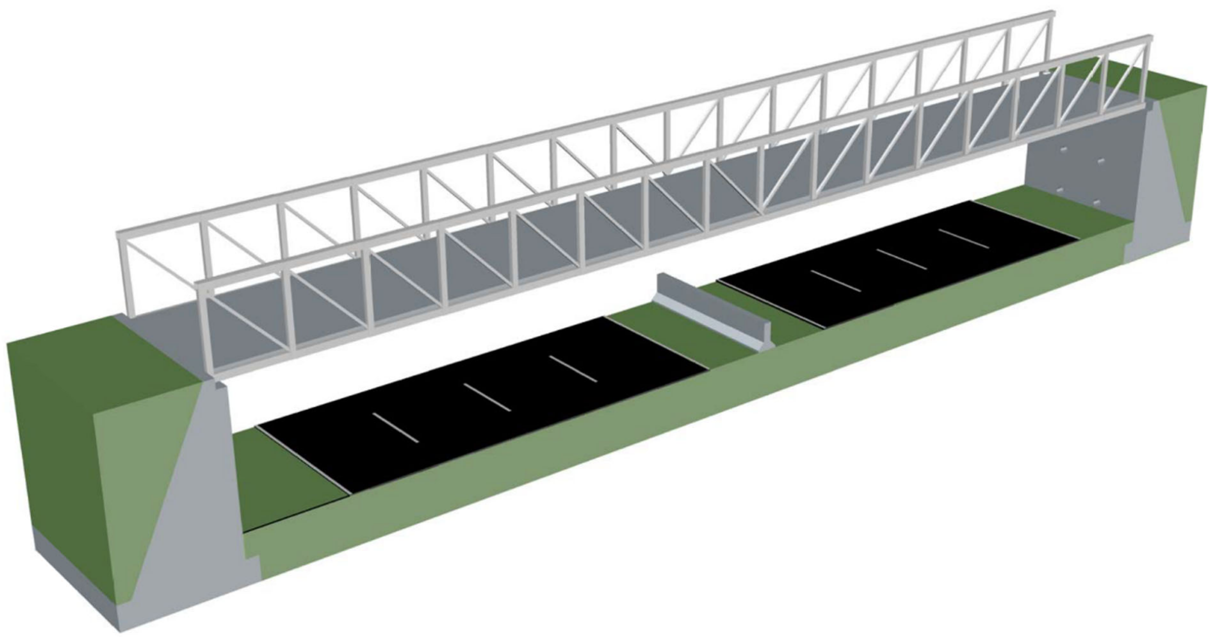

Figure 24: Conceptual AutoCAD Drawing of Final Full Replacement Bridge Design

Figure 24 shows a 3D rendering of the final full replacement design. While not pictured, the trail will continue to be paved on the grassy areas atop the abutments. Curbs will be on the bridge to assist with stormwater management, and a pedestrian fence will run along the length of each side of the bridge.

5.1 Final Structural Design

The geometry and member sizes for the final full replacement design can be found in Section 4.3.3 Replace Superstructure and Substructure, along with the abutment design.

Connections

The chosen connections for the full replacement design were fillet welds, as gusset plates require considerable maintenance after construction. The angle between the chord and branch is 40°, which is greater than the minimum 30° angle recommended for welds (AISC, 2017). Anything less than the minimum angle increases the difficulty of welding and later inspections. The design of HSS-to-HSS welds differs from typical weld design because there are limitations, such as punching shear, member collapse, or lamellar tearing. Due to these limitations, member size has an important role in the success of the connections. The minimum fillet weld size for members with a thickness greater than ¾ inch is 5/16 inch, per Table 5.7 in the *American Welding Society* (*AWS*) *D1.1.*

The factored available strength of the center connection is 633 kips, and the demand on the connection is 41 kips. The calculations for the connections can be found in Appendix J. Figure 25 shows the controlling connection, which would be in the center of the bridge, as this connection has two diagonal posts and a vertical post.

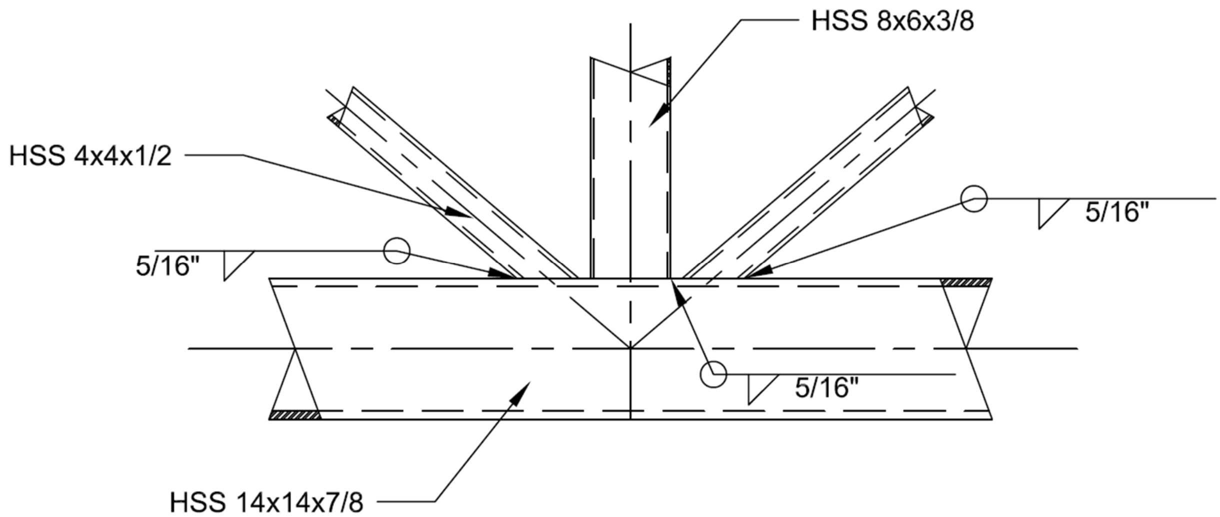

Figure 25: Center Connection Detail

Drainage

Drainage is an important consideration for this bridge because runoff from stormwater and snow melt cannot be allowed to freely flow down from the deck onto Route 2. The water must be controlled using a drainage system. In order to dissipate stormwater, the bridge will be crowned and cambered. The crown will be 1% and the camber will be 2.3%. Bridge drains, following the MassDOT standard detail in Figure 26, will be spaced at 20 feet on the bridge. These drains will connect into a larger carrier pipe that will convey the water to the abutments. The pipe will continue down the side of the abutments and deposit water into existing drainage systems for Route 2.

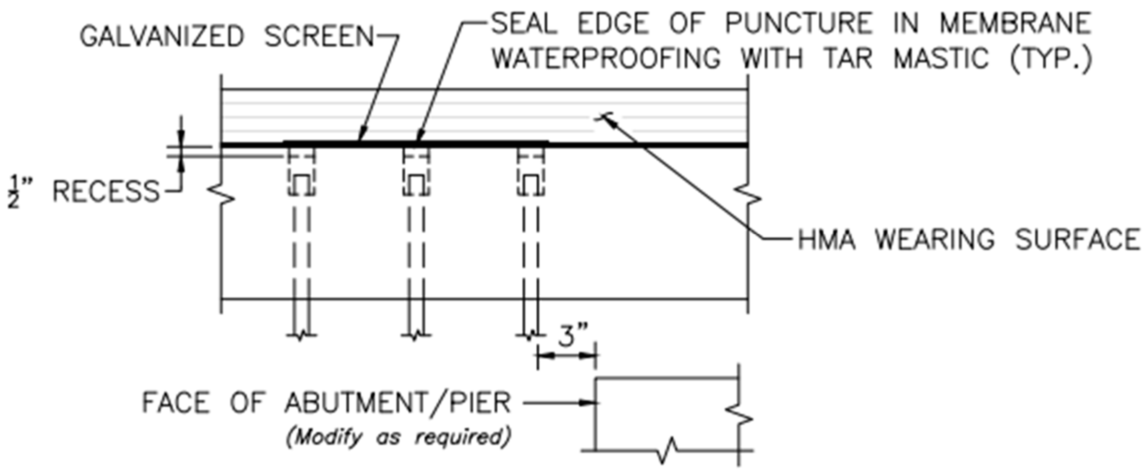

Figure 26: MassDOT Standard Detail of Drains for Bridge No.7.3.1 (MassDOT Bridge Manual Part II, 2013)

5.2 Demolition Plan

A site layout was created before demolition could begin as seen in Figure 27. Construction staging areas are flat, usable areas where construction vehicles, equipment, and formworks can be set up and stored.

Figure 27: Project Site Layout

The suggested sequence for demolition will begin by removing the contaminated ballast and disposing of it as hazardous waste. A third-party organization will be hired to remove and dispose of the waste properly, complying with 310 CMR 30.000 Massachusetts Hazardous Waste Regulations (Mass.gov, 2015).

After the ballast is removed, the concrete deck will need to be removed from the existing bridge. Shielding, made from timber planking or corrugated metal decking, will need to be put in place to catch any falling debris and protect traffic on Route 2. The steel diaphragm members will be cut and removed after the concrete deck has been demolished. The interior and exterior girders can then be individually picked out. Depending on the capacity of the cranes available from the contractor, the girders may need to be cut before they are picked out.

Based on information from manufacturer's websites and an AutoCAD model (Figure 28), a 165-ton all-terrain crane can be used to pick each girder out. After removal, the girders can be placed on a flat-bed semi-truck located along the pick arc. The girder picks will be at a maximum of 105 feet and crane capacities were checked for their capability to raise the boom an extra 20 feet to avoid any trees in the arc path, as necessary. It will be necessary to establish easy access to the bridge for construction vehicles. Possible options include the adjacent Double Tree parking lot or Erdman Way for access to the north side of the bridge, and Hamilton Street to access the south side. Minor brush and tree clearing may be necessary on each side of the bridge to account for this.

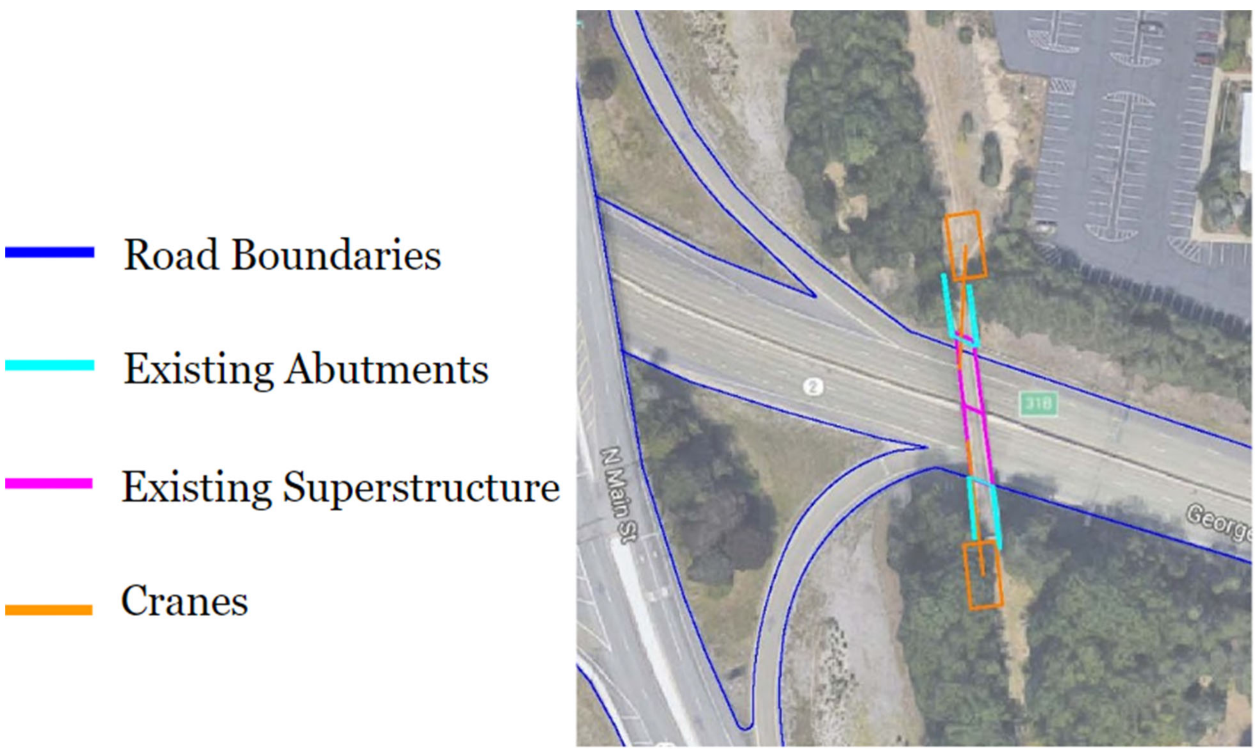

Figure 28: Crane Layout

Lane closures will be conducted as necessary during shielded construction. During parts of the demolition, such as the process of picking out the girders, an entire side of Route 2 may need to be closed. This should happen overnight to reduce traffic impacts during peak hours. The pier and abutments will need to be demolished with excavators using demolition hammer attachments. Soil will have to be excavated from behind the existing abutments at a maximum slope of 1:1, as determined by the existing soil type, to keep the soil from falling in the road during demolition. The abutments will be demolished from the approach side as much as possible to minimize disturbance of traffic flow on Route 2. The site work performed to demolish the existing abutments can be utilized to place the new abutments 12 feet back on either side of Route 2, and the excavated soil can be used to backfill the new abutments to the appropriate grade. The pier can be demolished at night and can be brought down to the height of the jersey barriers.

5.3 Construction and Traffic Management Plan

Due to the bridge's position over Route 2, traffic will be heavily impacted by the construction of the new bridge. MassDOT's Transportation Data Management System estimated an average annual daily traffic (AADT) count of 55,309 total motorists (circled in black in Figure 29) traveling eastbound and westbound on Route 2 in 2018 near the project site. In order to minimize delays for motorists, the team discussed multiple local detours for each direction based on records and data of intersection traffic volume provided by the MassDOT District 3 office and the AADT data. Detour routes were also assessed based on efficient use of exits, interchanges, and on/off ramps.

Figure 29: Average Annual Daily Traffic Count for Route 2 (Location of Bridge in Red Circle) (MassDOT, 2018)

The team met with a traffic engineer at Stantec, Fred Moseley, who offered insight and advice on how traffic could be impacted during construction. The engineer suggested that highway closures should be overnight, as there are fewer vehicles on the road, and that one or two detour options for each path would be reasonable. He explained that peak hour traffic volume is usually 10% of the daily traffic volume. Also, detouring traffic is not limited to roadways owned by the Massachusetts Department of Transportation (MassDOT). If a portion of the detour is on local roads, then it should be coordinated with local officials.

Using Google Maps, the team defined 2 detour paths for motorists traveling through Leominster. Vehicles traveling westbound can utilize paths like Main Street and Hamilton Street to direct vehicles back onto Route 2. Vehicles traveling eastbound can utilize North Main Street (Route 12) and Priest Street to be rerouted back onto Route 2. Figures 30 and 31 show two maps that outline the team's recommendations for detour paths for motorists to take during the construction process. Alternative detour maps can be found Appendix K.

Figure 30: Route 2 Westbound Reroute using Main Street/Hamilton Street (Location of Bridge in Red Circle) (Google Maps, 2019)

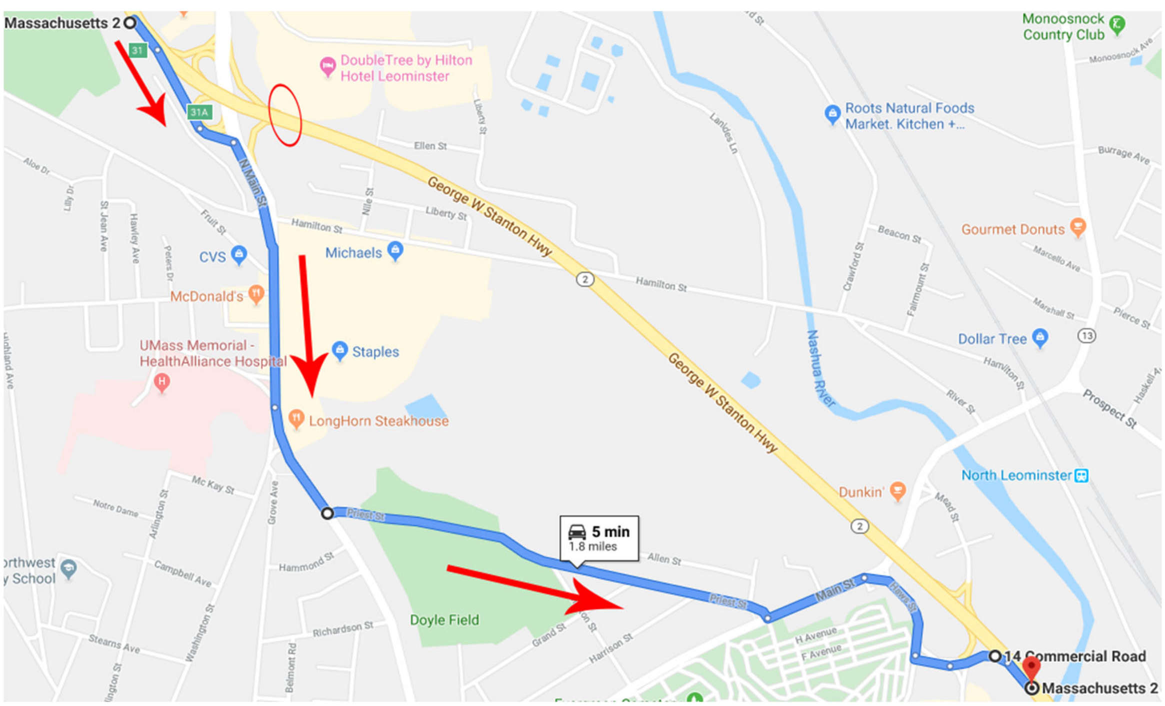

Figure 31: Route 2 Eastbound Reroute using Route 12/Priest Street (Location of Bridge in Red Circle) (Google Maps, 2019)

5.4 Additional Considerations

The team recommends the full replacement of the existing bridge for the Twin Cities Rail Trail pedestrian bridge. If this design is adopted, the following should be considered before finalizing the design:

1. Gather new soil data, since the soil data used for the substructure was from a boring log produced in May 1936
2. Consider prefabrication and Accelerated Bridge Construction (ABC) costs and compare them to traditional construction method costs
3. Drive along proposed detour routes to check for impediments and weight limits
4. Discuss drainage plan with MassDOT and determine final spacing of drains, pipe sizes, and outlet path
5. Work with MassDOT to understand exactly what alterations will be made to the Route 2 interchange as to not constrain future plans

With the completion of these considerations, the pedestrian bridge design can be finalized, constructed, and integrated into the Twin Cities Rail Trail to service pedestrians.

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Appendix A: Proposal

Twin Cities Rail Trail Bridge Design

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

Nicole Barrett
Jonathan Benoit
Alex Duffield
Hadi Hamdan
Isabella Morrison Ouellette

September 30, 2019

Advisors:
Leonard Albano
Suzanne LePage

Table of Contents

Capstone Design Statement

This Major Qualify Project (MQP) for Worcester Polytechnic Institute (WPI) will be completed by evaluating a current railroad bridge and designing alternatives for replacement or rehabilitation to transform the structure into a pedestrian bridge. The selected alternative will be developed into a detailed design. This bridge will be incorporated into the Twin Cities Rail Trail, and cross over Route 2 in Leominster, Massachusetts. The team will determine multiple potential solutions to the development of the pedestrian bridge and conduct structural calculations for the final design. In doing so, there are several design constraints that need to be addressed: economic, environmental, social and political, ethical, health and safety, constructability, and sustainability. By considering these constraints in the design, this MQP will satisfy the requirements for the Capstone Design Experience, as determined by the Accreditation Board of Engineering and Technology (ABET).

Economic
Economics is an important factor in determining if the construction project is within budget. The materials and methods selected for the project can help determine this cost and can be altered to keep the project within budget. When determining design options, the overall cost will be a consideration when choosing the final design. The design, materials, and construction cost associated with the pedestrian bridge will be considered to determine which design is the most cost-effective. A preliminary cost estimate will be conducted using previous statewide average bid prices and by analyzing past Stantec projects.

Environmental
The pedestrian bridge over Route 2 is connected to a trail surrounded by a wooded area. When considering bridge design and construction, destruction of the natural habitat and material contamination must be mitigated. Due to limited space for construction equipment, the final design and construction proposal will be made to minimize impacts on the environment and surrounding area.

Social and Political
The construction of the bridge will serve as a connection between communities. During the design process, it is important to consider the social and political setting of the structure. Many people in the community have been waiting for the Twin Cities Rail Trail for decades, and are very invested in the bridge. It will be important to be sensitive to property owners and traffic in the surrounding area when designing the bridge, demolition plans, and traffic plans.

Ethical
The American Society of Civil Engineers (ASCE) Code of Ethics will be followed throughout the duration of the project. It is an engineer’s duty to be ethical and hold themselves to high standards as they directly impact the lives of people. It is necessary to ensure that all recommended designs are safe for public use.

Health and Safety

The health and safety of the users and laborers of the bridge will be one of the main priorities in the design. The design and construction of the project must comply with all building codes and laws. Recommendations for improving the pedestrian bridge will ensure that the structure can meet or exceed structural and serviceability requirements defined by the *AASHTO Guide Specifications for the Design of Pedestrian Bridges*.

Manufacturability/Constructability

Constructability must be factored into the final pedestrian bridge design due to its restrictive location over Route 2. With Route 2 being heavily trafficked, chosen materials must be able to be shipped to the site and constructed in a short period of time. Alternatively, the construction process could be more drawn out, but be less invasive to the area around the bridge. Custom members could require more labor to install on site, an aspect that needs to be considered with limited time windows for construction. The construction site may also limit what vehicles can be used and if members can be fabricated on-site.

Sustainability

Different building materials will be looked at for their sustainability. Specifically, the team will be evaluating material durability to weathering. Maintenance costs and the environmental impacts from the making of the material will also be considered.

1.0 Introduction

The Twin Cities Rail Trail is a 4.5-mile long planned project that will connect the town centers of Fitchburg and Leominster in Massachusetts. The MassDOT project will provide a protected, paved path for pedestrian and bicycle traffic along what is currently an abandoned segment of the Fitchburg and Worcester Railroad. This connection will promote sustainable transportation practices and is a project twenty years in the making (Dore, 2019).

The Twin Cities Rail Trail crosses directly over Route 2 in Leominster. Decisions must be made regarding an existing, abandoned railroad bridge crossing Route 2. The team must determine whether or not it can be rehabilitated and repurposed into a pedestrian bridge for the trail, or reconstruction should be recommended.

The goal of this project is to deliver a comprehensive proposal for the pedestrian bridge over Route 2. The following objectives outline the process for completing the project.

1. Evaluate existing conditions
2. Define and establish design criteria
3. Develop and screen preliminary design concepts
4. Develop alternative schematic designs
5. Evaluate alternatives and select the best option
6. Develop detailed design for the selected option

To assess the current condition of the bridge, the team will evaluate the bridge through a site visit and load rating calculations. This information will be used later to compare design options, including rehabilitation and full replacement alternatives. Unreasonable options will be eliminated based on the constraints of the project specifications. Schematic designs will be developed and evaluated using a decision matrix. The best design based on the decision matrix will be further developed into a final design.

2.0 Background

Railroads have provided a connection between people for many years. Now, there is an opportunity to reconnect the cities of Leominster and Fitchburg using the old railroad, but changing the mode of transportation. Instead of trains running between the towns, pedestrians will have the opportunity to travel between the cities by the means of the Twin Cities Rail Trail. With updates to its infrastructure, specifically a bridge over Route 2, the trail will be ready to connect the cities again.

2.1 Railroad and Bridge History

The Fitchburg and Worcester Railroad was incorporated in 1840 to provide a rail connection between Fitchburg and Worcester. Service of the rails began on February 11, 1850, running 18 miles from Fitchburg through Leominster to Sterling Junction and connecting with the Worcester and Nashua Railroad. The railroad was controlled by three different owners from its initial integration until the Surface Transportation Board approved the buyout of the Worcester and Fitchburg Railroad by CSX in 1998 (Revolvy, 2019). Eventually, the tracks between Leominster and Fitchburg were abandoned, leaving behind a trail and a railroad bridge over Route 2 in Leominster. The current section of Route 2 spanning from Leominster (Route 12) to Concord (Rotary) was constructed from 1950-1953 (Carr, 2007). Given that the abutments to the bridge are built on the sides of Route 2 and the construction entailed widening the previous route from 2 lanes to 4 lanes, it is likely possible that the bridge was built during or after 1950. The existing structure, as seen in Figure 1, was designed to be a two-span railroad bridge.

Fig 1: Framingham and Worcester Railroad Bridge Over Route 2 in Leominster (Pi.1415926535, 2015)

2.2 Proposed Trail Project

MassDOT is overseeing the construction of a proposed 4.5-mile long trail that will connect two town centers in Massachusetts: Fitchburg and Leominster. Called the “Twin City Rail Trail”, the paved trail will be 12 feet wide and follow the abandoned commercial railroad corridor shown in Figure 2. The goal of the trail is to “provide a non-motorized transportation and recreational alternative for people of all ages and abilities” (Dore, 2019). The trail, therefore, promotes more sustainable transportation alternatives and reconnects the two towns of Fitchburg and Leominster with a direct path. The project will be broken up into two phases, with the first phase of construction beginning in the Spring of 2020 and costing an estimated $8,081,000 (MassDOT, 2019).

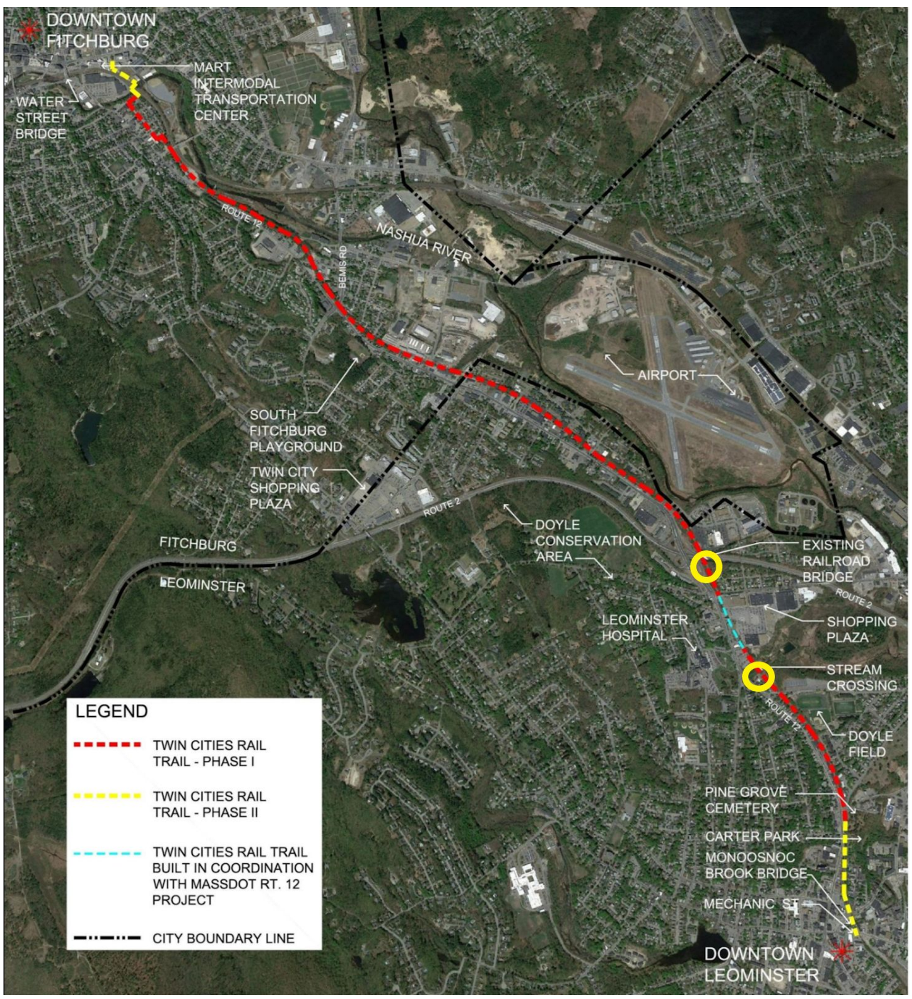

Figure 2: Complete Map of Twin City Rail Trail (Core, 2019)

Included in the first phase of construction, outlined in red in Figure 2, is the replacement or rehabilitation of the existing abandoned railroad bridge over Route 2 in Leominster. The development of the bridge is essential in transporting pedestrians along the designated trail. An additional bridge needs to be designed and constructed to allow pedestrians to cross over Hamilton Street Brook in Leominster. Both bridges are included in Phase 1 and their locations are circled in yellow in Figure 2.

2.3 Project Specifications

The project will be conducted in compliance with standard industry practices. Typical methods will be followed in accordance with AASHTO and MassDOT specifications. *MassDOT Load and Resistance Factor Design (LRFD) Bridge Design* Chapters 2 and 3 detail pedestrian bridge design specifications. Chapter 3 of the specifications guide engineers to refer to *AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges* and *Americans with Disabilities Act (ADA)* during the design process. Since the bridge design will be for pedestrian use, both specifications will need to be followed to design a safe and serviceable structure. Additionally, the bridge design must be constructible over Route 2 and be able to join with the existing trail area. These requirements, as well as those outlined in the specifications, will determine many of the parameters of the design.

3.0 Methods

The team will be achieving its goal of delivering a comprehensive proposal for the pedestrian bridge over Route 2 by following these objectives:

1. Evaluate existing conditions
2. Define and establish design criteria
3. Develop concepts
4. Develop schematic pedestrian bridge designs
5. Evaluate alternatives
6. Develop detailed design for selected option

The team will determine whether rehabilitation or reconstruction is more appropriate for the bridge design. Then, the team will complete a detailed design for the selected option. If the team feels that the time and resources are available after the first week of the project, the team will consider working on the design of the second bridge on the Twin Cities Rail Trail. This process will largely follow the same steps as those for the bridge over Route 2. The objectives are described in more detail in the following sections.

3.1 Evaluate Existing Conditions

First, the team will evaluate the existing conditions of the railroad bridge and project site. The team will initially travel to the bridge with Stantec liaisons to gather information to use during evaluation of the bridge. The purpose of the Stantec liaisons will be to provide a more experienced opinion when gathering information about the structural condition of the superstructure and substructure. Photos will be taken of the bridge and the surrounding area and trail, to get a better sense of the condition of the bridge and accessibility for potential construction.

The inspection report to be obtained from Stantec will be used to further evaluate the condition of the bridge. A load rating will be calculated to determine if the bridge is able to support the required loading. The load rating calculations will be in accordance with the *AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges*.

If the load rating of the bridge is sufficient and meets AASHTO standards, the remaining fatigue life will be evaluated in accordance with AASHTO requirements. An expired or soon to expire fatigue life would be taken into account when comparing options for rehabilitation or replacement.

3.2 Define and Establish Design Criteria

The team will determine the public demands of the bridge. Public meeting minutes, town ordinances, and codes will be referenced to determine the expected uses and required design loads. If special uses, such as biking or horseback riding are expected on the bridge, the team will factor them into the design concepts. The associated loading of the intended use will be determined in conjunction with the *AASHTO LRFD Guide Specification for the Design of Pedestrian Bridges*, as referenced in the MassDOT specifications. Additionally, the team will determine if there are aesthetic considerations or limitations based on historic bridge standing or Leominster ordinances for the structure. The established design criteria will serve as the basis for the decision matrix.

3.3 Develop Concepts

The team will develop several concepts for the design of the bridge moving forward. These will include various options for rehabilitation and full replacement of the bridge. Initially, the team will determine if existing conditions or historic rulings could impact the design and prohibit rehabilitation. If rehabilitation is determined to be a possible design option, aesthetic changes, superstructure replacement, and miscellaneous repairs will be considered. Full replacement options would include various designs and materials used, for example, concrete slab, steel girder, wood, arch, etc.

Concept designs can be eliminated from consideration if they are deemed unreasonable, meaning that the particular design is not feasible. Feasibility of a design may rely on span length requirements or existing conditions limitations. The feasibility of construction on and around the bridge will also be considered for each option, as well as the impact on traffic during construction.

While conceptualizing the options for the bridge, a preliminary cost estimate for each option will be calculated. Cost estimates will be derived from bridge designs of similar size and materials, in addition to possible construction requirements. Example bridges will be sourced from similar MassDOT projects.

3.4 Develop Schematic Pedestrian Bridge Designs

The team will develop schematic pedestrian bridge designs for the concepts that are deemed feasible. Such designs will entail performing structural calculations for primary member sizes and determining material requirements. From this information, cost estimates will be formed.

The designs will be developed according to the accepted standard practices, following MassDOT Chapter 2 and AASHTO specifications regarding pedestrian bridge requirements.

3.5 Evaluate Alternatives

The team will create and apply the decision matrix to the general designs completed for the selected concepts. The matrix will evaluate each concept design based on the criteria outlined in the Capstone Design Statement. Therefore, the matrix evaluates potential designs based on how well they satisfy each of the following criteria: health and safety, economics, environmental impacts, aesthetics, constructability, and ethicalness. Each design will receive ratings ranging from one to ten for each criterion, with a higher score indicating a greater degree of satisfaction for the specific criterion.

The team will choose the concept with the highest overall score to develop and evaluate further into a final design recommendation.

3.6 Develop Detailed Design for Selected Option

The team will develop the final design for the option selected from the decision matrix. The design option will be detailed through structural calculations. Standard practice guidelines will be followed. Programs such as RISA 2D, RISA 3D, and AutoCad may be used to assist with structural calculations. Overall constructability will be evaluated for the design and construction specifications will be developed. Based on the selected materials for design and construction, a cost estimate will be produced. Stantec’s preferred source of material unit prices will be referenced during this process. The team will also consider traffic management during construction. Considerations for both pedestrian traffic on the trail and automobile traffic on the road will be made during the process of making traffic plans. Finally, the team will create demolition plans for the parts of the current structure that must be removed. The degree of demolition will depend on the design pursued after the application of the decision matrix.

3.8 Deliverables

At the end of the term, the team will present several deliverables. A final pedestrian bridge design to span Route 2 will be presented. This will include a structural design, suggestions for traffic control during construction, a cost estimate, demolition plans, and construction specifications. These will be presented to Stantec at the end of the project. The MQP report will be presented at the completion of the project. The team will also work on the MQP poster, which will be presented at WPI in April.

3.9 Schedule

The proposed tasks and schedule for this project are outlined below. The colors correspond to the person who is in charge of managing the task in the chart. The person in charge of the task will help delegate and organize work among other team members. Several people will be assigned to each task, depending on their interests in the project. The structural calculation task will involve the most people, as everyone on the team is interested in structural design.

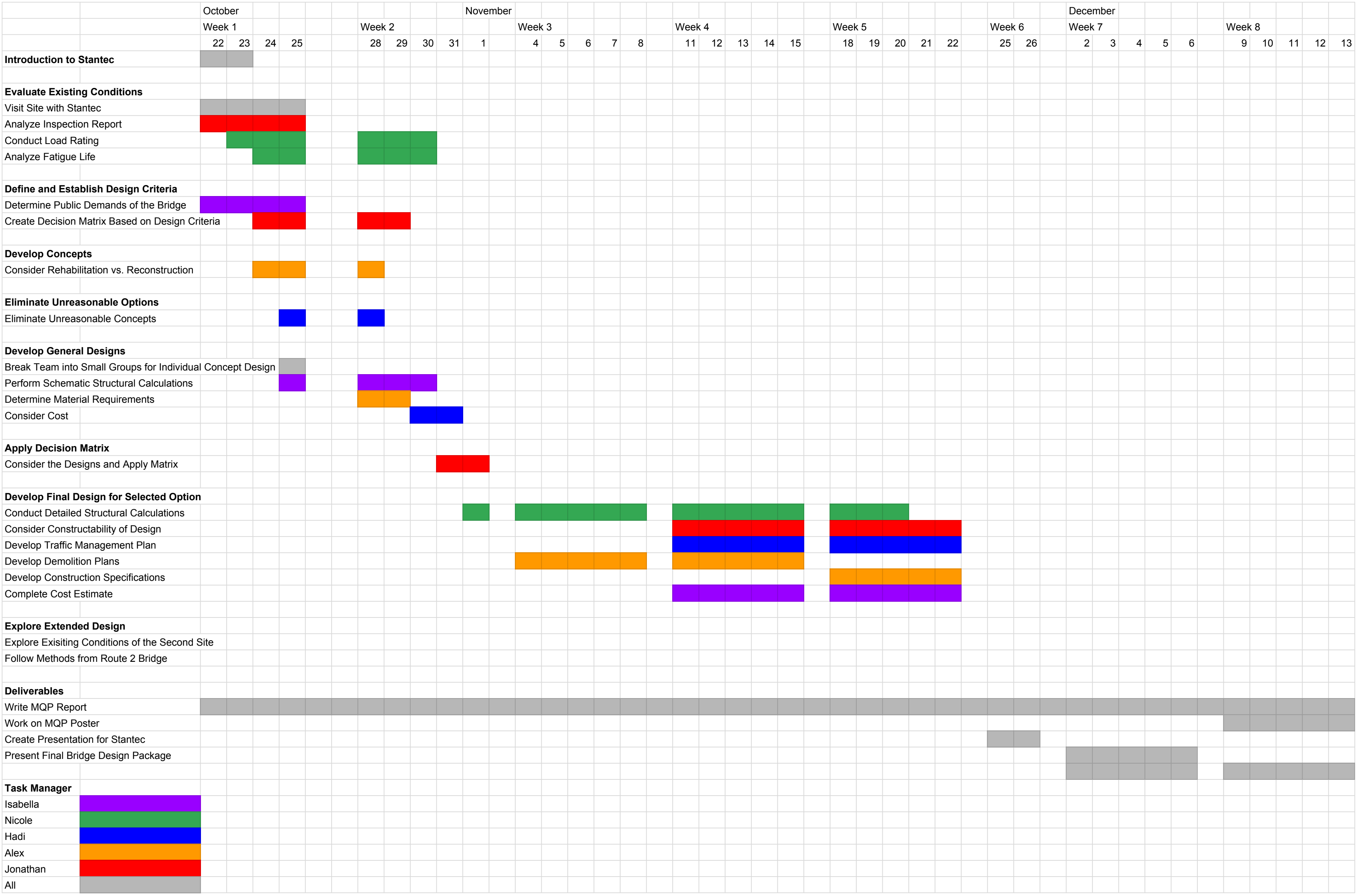

Figure 3. Distribution of Work Over the Term and the Team Members Leading Each Task

References

Figure 1: Pi.1415926535, “Framingham and Worcester Railroad bridge over Route 2 in Leominster, November 2015.JPG https://commons.wikimedia.org/wiki/File:Framingham_and_Worcester_Railroad_bridge_over_Route_2_in_Leominster,_November_2015.JPG

Figure 2: Dore, David. “Fitchburg-Leominster Rail Trail Takes a Step Forward at Public Hearing.” Telegram.com, Telegram.com, 20 Mar. 2019, www.telegram.com/news/20190320/fitchburg-leominster-rail-trail-takes-step-forward-at-public-hearing

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Appendix B: Inspection Reports and As-Built Plans

MASSACHUSETTS DEPARTMENT OF TRANSPORTATION
STRUCTURES INSPECTION FIELD REPORT
FRACTURE CRITICAL INSPECTION

2-DIST	B.I.N.	BR. DEPT. NO.
03	7L0	L-08-023

CITY/TOWN	8.-STRUCTURE NO.	11-Kilo. POINT	90-ROUTINE INSP. DATE	93a - F.C. INSP. DATE
LEOMINSTER	L08023-7L0-DOT-RRO	000.000	Jul 16, 2015	Jul 16, 2015

07-FACILITY CARRIED	MEMORIAL NAME/LOCAL NAME	27-YR BUILT	106-YR REBUILT	*YR REHAB'D (NON 106)
RR ABNDONED CSX		1951	0000	0000

06-FEATURES INTERSECTED	26-FUNCTIONAL CLASS	DIST. BRIDGE INSPECTION ENGINEER
ST 2		M. Azizi

43-STRUCTURE TYPE	22-OWNER	21-MAINTAINER	TEAM LEADER
303 : Steel Girder & Floorbeam	State Highway Agency	State Highway Agency	M. Azizi

107-DECK TYPE	WEATHER	TEMP. (air)	TEAM MEMBERS
1 : Concrete Cast-in-Place	Sunny	23°C	R. ROGERS

WEIGHT POSTING *Not Applicable* X

	H	3	3S2	Single
Actual Posting	N	N	N	N
Recommended Posting	N	N	N	N

Waived Date: 00/00/00 EJDMT Date: 00/00/00

Signs In Place (Y=Yes,N=No, NR=Not Required)
Legibility/ Visibility

	At bridge N	At bridge S	Advance N	Advance S
Signs In Place				
Legibility/ Visibility				

PLANS (Y/N): Y

(V.C.R.) (Y/N): N

TAPE#:

RATING

Rating Report (Y/N): N Date: ----

Request for Rating or Rerating (Y/N): N

If YES please give priority: HIGH ( ) MEDIUM ( ) LOW ( )

REASON:

Inspection data at time of existing rating

I 58: - I 59: - I 60: - I 62: Date : 00/00/00

FRACTURE CRITICAL MEMBER(S):

	MEMBER	CRACK (Y/N):	WELD'S CONDITION (0-9)	LOCATION OF CORROSION, SECTION LOSS (%), CRACKS, COLLISION DAMAGE, STRESS CONCENTRATION, ETC.	CONDITION PREVIOUS (0-9)	CONDITION PRESENT (0-9)	INV. RATING OF MEMBER FROM RATING ANALYSIS H-20	3	3S2	Deficiencies
A	Item 59.4 - Girders or Beams	N	N	See remarks in comments section.	6	6	Not Rated			S-P
B										
C										
D										
E										

List of field tests performed:
None:

	I-59	I-60
(Overall Previous Condition)	6	6
(Overall Current Condition)	6	6

DEFICIENCY: A defect in a structure that requires corrective action.

CATEGORIES OF DEFICIENCIES:

M= Minor Deficiency - Deficiencies which are minor in nature, generally do not impact the structural integrity of the bridge and could easily be repaired. Examples include but are not limited to: Spalled concrete, Minor pot holes, Minor corrosion of steel, Minor scouring, Clogged drainage, etc.

S= Severe/Major Deficiency - Deficiencies which are more extensive in nature and need more planning and effort to repair. Examples include but are not limited to: Moderate to major deterioration in concrete, Exposed and corroded rebars, Considerable settlement, Considerable scouring or undermining, Moderate to extensive corrosion to structural steel with measurable loss of section, etc.

C-S= Critical Structural Deficiency - A deficiency in a structural element of a bridge that poses an extreme unsafe condition due to the failure or imminent failure of the element which will affect the structural integrity of the bridge.

C-H= Critical Hazard Deficiency - A deficiency in a component or element of a bridge that poses an extreme hazard or unsafe condition to the public, but does not impair the structural integrity of the bridge. Examples include but are not limited to: Loose concrete hanging down over traffic or pedestrians, A hole in a sidewalk that may cause injuries to pedestrians, Missing section of bridge railing, etc.

URGENCY OF REPAIR:

I = Immediate- [Inspector(s) immediately contact District Bridge Inspection Engineer (DBIE) to report the Deficiency and to receive further instruction from him/her].
A = ASAP- [Action/Repair should be initiated by District Maintenance Engineer or the Responsible Party (if not a State owned bridge) upon receipt of the Inspection Report].
P = Prioritize- [Shall be prioritized by District Maintenance Engineer or the Responsible Party (if not a State owned bridge) and repairs made when funds and/or manpower is available].

X=UNKNOWN N=NOT APPLICABLE H=HIDDEN/INACCESSIBLE R=REMOVED

F.C.(1)7-96

CITY/TOWN	B.I.N.	BR. DEPT. NO.	8.-STRUCTURE NO.	INSPECTION DATE
LEOMINSTER	7L0	L-08-023	L08023-7L0-DOT-RRO	JUL 16, 2015

REMARKS

BRIDGE ORIENTATION

According to the plans the approaches are South to North and the elevations are West to East. This structure is a two span riveted steel plate girder bridge with two girders numbered from West to East. There are eleven floorbeams and twelve bays in each span numbered from South to North. There are two stringers in each span numbered from West to East. There is one solid concrete pier.

ITEM 59 - SUPERSTRUCTURE

Item 59.4 - Girders or Beams

Girder #1 (West fascia):

Girder #1 has heavy paint peeling and surface rusting in many areas throughout, heaviest to the bottom flange and outside face of the girder web. Girder #1 has isolated areas of minor rust flaking, to the outside face of the web, along the top of the bottom riveted web plate. The outside face of girder #1 shows several areas of minor graffiti. **See photo #1.**

Girder #2 (East fascia):

Girder #2 shows moderate to heavy paint peeling and surface rusting in many areas throughout, heaviest to the bottom flange and the outside web. There is minor rust flaking along the entire length of the outside face of girder #2, below the deck line. The outside face of girder #2 shows several areas of minor graffiti. The top half of the inside East fascia shows heavy rust flaking to the interior vertical stiffeners at the concrete interface.**See photos #1- #3.**

Photo Log

Photo 1 : Typical heavy paint peel and surface rusting to the outside face of girder #2 in span #2.

Photo 2 : Typical heavy paint peel and surface rusting and minor graffiti to the outside face of girder #2 in span #1.

Photo 3 : Typical East side interior vertical stiffeners showing heavy rust flaking.

MASSACHUSETTS DEPARTMENT OF TRANSPORTATION
STRUCTURES INSPECTION FIELD REPORT
ROUTINE INSPECTION

2-DIST	B.I.N.	BR. DEPT. NO.
03	7L0	L-08-023

CITY/TOWN	8.-STRUCTURE NO.	11-Kilo. POINT	41-STATUS	90-ROUTINE INSP. DATE
LEOMINSTER	L08023-7L0-DOT-RRO	000.000	K:CLOSED	JUL 18, 2017

07-FACILITY CARRIED	MEMORIAL NAME/LOCAL NAME	27-YR BUILT	106-YR REBUILT	YR REHAB'D (NON 106)
RR ABNDONED CSX		1951	0000	0000

06-FEATURES INTERSECTED	26-FUNCTIONAL CLASS	DIST. BRIDGE INSPECTION ENGINEER
ST 2		M. Azizi

43-STRUCTURE TYPE	22-OWNER	21-MAINTAINER	TEAM LEADER
303 : Steel Girder & Floorbeam	State Highway Agency	State Highway Agency	D. Simkhovich

107-DECK TYPE	WEATHER	TEMP. (air)	TEAM MEMBERS
1 : Concrete Cast-in-Place	Sunny	23°C	R. ORLANDO

ITEM 58 — DECK: 6

DECK		DEF
1.Wearing surface	N	-
2.Deck Condition	6	M-P
3.Stay in place forms	N	-
4.Curbs	N	-
5.Median	N	-
6.Sidewalks	N	-
7.Parapets	5	S-A
8.Railing	N	-
9.Anti Missile Fence	N	-
10.Drainage System	N	-
11.Lighting Standards	N	-
12.Utilities	N	-
13.Deck Joints	N	-
14.	N	-
15.	N	-
16.	N	-

CURB REVEAL (In millimeters): E: N W: N

APPROACHES		DEF
a. Appr. pavement condition	N	-
b. Appr. Roadway Settlement	N	-
c. Appr. Sidewalk Settlement	N	-
d.	N	-

OVERHEAD SIGNS (Attached to bridge) (Y/N): N

		DEF
a. Condition of Welds	N	-
b. Condition of Bolts	N	-
c. Condition of Signs	N	-

ITEM 59 — SUPERSTRUCTURE: 6

SUPERSTRUCTURE		DEF
1.Stringers	6	M-P
2.Floorbeams	6	M-P
3.Floor System Bracing	N	-
4.Girders or Beams	6	M-P
5.Trusses - General	N	-
a. Upper Chords	N	-
b. Lower Chords	N	-
c. Web Members	N	-
d. Lateral Bracing	N	-
e. Sway Bracings	N	-
f. Portals	N	-
g. End Posts	N	-
6.Pin & Hangers	N	-
7.Conn Plt's, Gussets & Angles	6	M-P
8.Cover Plates	N	-
9.Bearing Devices	6	M-P
10.Diaphragms/Cross Frames	N	-
11.Rivets & Bolts	6	M-P
12.Welds	N	-
13.Member Alignment	8	-
14.Paint/Coating	4	S-P
15.	N	-

Year Painted: 1987

COLLISION DAMAGE: *Please explain*
None (X) Minor ( ) Moderate ( ) Severe ( )

LOAD DEFLECTION: *Please explain*
None (X) Minor ( ) Moderate ( ) Severe ( )

LOAD VIBRATION: *Please explain*
None (X) Minor ( ) Moderate ( ) Severe ( )

Any Fracture Critical Member: (Y/N) Y

Any Cracks: (Y/N) N

ITEM 60 — SUBSTRUCTURE: 6

SUBSTRUCTURE	Dive	Cur	DEF
1. Abutments		6	
a. Pedestals	N	N	-
b. Bridge Seats	N	6	M-P
c. Backwalls	N	7	-
d. Breastwalls	N	6	M-P
e. Wingwalls	N	7	-
f. Slope Paving/Rip-Rap	N	N	-
g. Pointing	N	6	M-P
h. Footings	N	H	-
i. Piles	N	N	-
j. Scour	N	N	-
k. Settlement	N	7	-
l.	N	N	-
m.	N	N	-
2. Piers or Bents		6	
a. Pedestals	N	N	-
b. Caps	N	N	-
c. Columns	N	N	-
d. Stems/Webs/Pierwalls	N	6	M-P
e. Pointing	N	7	-
f. Footing	N	H	-
g. Piles	N	N	-
h. Scour	N	N	-
i. Settlement	N	7	-
j.	N	N	-
k.	N	N	-
3. Pile Bents		N	
a. Pile Caps	N	N	-
b. Piles	N	N	-
c. Diagonal Bracing	N	N	-
d. Horizontal Bracing	N	N	-
e. Fasteners	N	N	-

UNDERMINING (Y/N) If YES please explain: N

COLLISION DAMAGE:
None (X) Minor ( ) Moderate ( ) Severe ( )

SCOUR: *Please explain*
None (X) Minor ( ) Moderate ( ) Severe ( )

I-60 (Dive Report): N *I-60 (This Report):* 6

93B-U/W (DIVE) Insp: 00/00/0000

X=UNKNOWN N=NOT APPLICABLE H=HIDDEN/INACCESSIBLE R=REMOVED

CITY/TOWN	B.I.N.	BR. DEPT. NO.	8.-STRUCTURE NO.	INSPECTION DATE
LEOMINSTER	**7L0**	**L-08-023**	**L08023-7L0-DOT-RRO**	**JUL 18, 2017**

ITEM 61 **N**

CHANNEL & CHANNEL PROTECTION

	Dive	Cur	DEF
1.Channel Scour	N	N	-
2.Embankment Erosion	N	N	-
3.Debris	N	N	-
4.Vegetation	N	N	-
5.Utilities	N	N	-
6.Rip-Rap/Slope Protection	N	N	-
7.Aggradation	N	N	-
8.Fender System	N	N	-

STREAM FLOW VELOCITY:
Tidal ( ) High ( ) Moderate ( ) Low ( ) None (X)

ITEM 61 (Dive Report): **N** *ITEM 61 (This Report):* **N**

93b-U/W INSP. DATE: 00/00/0000

ITEM 36 ***TRAFFIC SAFETY***

	36	COND	DEF
A. Bridge Railing	0	5	S-A
B. Transitions	N	N	-
C. Approach Guardrail	N	N	-
D. Approach Guardrail Ends	N	N	-

WEIGHT POSTING ***Not Applicable*** **X**

	H	3	3S2	Single
Actual Posting	N	N	N	N
Recommended Posting	N	N	N	N

Waived Date: 00/00/0000 **EJDMT Date:** 00/00/0000

	At bridge N	At bridge S	Other Advance N	Other Advance S
Signs In Place (Y=Yes,N=No, NR=NotRequired)				
Legibility/ Visibility				

CLEARANCE POSTING

Not **X**

	E ft	E in	W ft	W in	meter
Actual Field Measurement	16	2	15	11	4.85
Posted Clearance		0		0	

	At bridge E	At bridge W	Advance E	Advance W
Signs In Place (Y=Yes,N=No, NR=Not Required)				
Legibility/ Visibility				

ACCESSIBILITY **(Y/N/P)**

	Needed	Used
Lift Bucket	Y	Y
Ladder	P	N
Boat	N	N
Waders	N	N
Inspector 50	N	N
Rigging	N	N
Staging	N	N
Traffic Control	Y	Y
RR Flagger	N	N
Police	Y	Y
Other:		
OVERTIME	Y	Y

TOTAL HOURS 8

PLANS (Y/N): **Y**

(V.C.R.) (Y/N): **N**

TAPE#:

List of field tests performed:
None:

RATING
Rating Report (Y/N): **N**
Date: 00/00/0000
Inspection data at time of existing rating
I 58: **-** I 59: **-** I 60: **-** Date :00/00/0000

(To be filled out by DBIE)
Request for Rating or Rerating (Y/N): **N**
If YES please give priority: HIGH ( ) MEDIUM ( ) LOW ( )
REASON:

CONDITION RATING GUIDE (For Items 58, 59, 60 and 61)

	CODE	*CONDITION*	*DEFECTS*
	N	NOT APPLICABLE	
G	9	EXCELLENT	Excellent condition.
G	8	VERY GOOD	No problem noted.
G	7	GOOD	Some minor problems.
F	6	SATISFACTORY	Structural elements show some minor deterioration.
F	5	FAIR	All primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
P	4	POOR	Advanced section loss, deterioration, spalling or scour.
P	3	SERIOUS	Loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
C	2	CRITICAL	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
C	1	"IMMINENT" FAILURE	Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stablility. Bridge is closed to traffic but corrective action may put it back in light service.
	0	FAILED	Out of service - beyond corrective action.

DEFICIENCY REPORTING GUIDE

DEFICIENCY: A defect in a structure that requires corrective action.

CATEGORIES OF DEFICIENCIES:

M= Minor Deficiency - Deficiencies which are minor in nature, generally do not impact the structural integrity of the bridge and could easily be repaired. Examples include but are not limited to: Spalled concrete, Minor pot holes, Minor corrosion of steel, Minor scouring, Clogged drainage, etc.

S= Severe/Major Deficiency - Deficiencies which are more extensive in nature and need more planning and effort to repair. Examples include but are not limited to: Moderate to major deterioration in concrete, Exposed and corroded rebars, Considerable settlement, Considerable scouring or undermining, Moderate to extensive corrosion to structural steel with measurable loss of section, etc.

C-S= Critical Structural Deficiency - A deficiency in a structural element of a bridge that poses an extreme unsafe condition due to the failure or imminent failure of the element which will affect the structural integrity of the bridge.

C-H= Critical Hazard Deficiency - A deficiency in a component or element of a bridge that poses an extreme hazard or unsafe condition to the public, but does not impair the structural integrity of the bridge. Examples include but are not limited to: Loose concrete hanging down over traffic or pedestrians, A hole in a sidewalk that may cause injuries to pedestrians, Missing section of bridge railing, etc.

URGENCY OF REPAIR:

I = Immediate- [Inspector(s) immediately contact District Bridge Inspection Engineer (DBIE) to report the Deficiency and to receive further instruction from him/her].
A = ASAP- [Action/Repair should be initiated by District Maintenance Engineer or the Responsible Party (if not a State owned bridge) upon receipt of the Inspection Report].
P = Prioritize- [Shall be prioritized by District Maintenance Engineer or the Responsible Party (if not a State owned bridge) and repairs made when funds and/or manpower is available].

CITY/TOWN	B.I.N.	BR. DEPT. NO.	8.-STRUCTURE NO.	INSPECTION DATE
LEOMINSTER	7L0	L-08-023	L08023-7L0-DOT-RRO	JUL 18, 2017

REMARKS

:cording to the plans, the approaches are South and North and the elevations are West and East. This ructure is a two span riveted steel plate girder bridge with two girders numbered from West to East. There 'e eleven floorbeams and twelve bays in each span numbered from South to North. There are two ringers in each span numbered from West to East. There is one solid concrete pier. The spans are ımbered from South to North.

ENERAL REMARKS

ıis inspection is not intended to be an official Federal Railroad Administration mandated inspection. The ırpose of this inspection was to assess the primary structural elements and report on deficiencies that quire maintenance on this MassDOT owned and maintained railroad structure.

ıere is jersey type barriers in place at both ends of the bridge are to keep traffic off. **See photos 1 and 2.** ıe bridge is open to pedestrians. There is chain link fencing on top of the jersey barrier along both sides of e bridge.

'EM 58 - DECK

em 58.2 - Deck Condition

ıere is moderate longitudinal hairline cracking with heavy efflorescence and efflorescence icicles to the eck underside, at the interface with both girders. **See photo 3.** The areas of worse cracking are on the 'est side.

ıe top of the deck, the sections that are up against the girders have longitudinal and transverse cracking,) to 1/16 inch wide in may areas. **See photos 4 and 5.** According to the plans the deck has a 2 inch ortar protective course over a waterproof membrane on top of the concrete deck. Bays #6, #7 and #10 ıver Eastbound) have areas of moisture staining with efflorescence buildup adjacent to both girders. **See ıoto 6.**

em 58.7 - Parapets

ıere is a section of dislodged chain link fence at the North end of the East jersey shaped parapet. There 'e 4 missing posts in this area. **See photo 7.**

'EM 59 - SUPERSTRUCTURE

em 59.1 - Stringers

ee Item #59.14.

em 59.2 - Floorbeams

ee Item #59.14.

em 59.4 - Girders or Beams

irder #1 (West fascia):

CITY/TOWN	B.I.N.	BR. DEPT. NO.	8.-STRUCTURE NO.	INSPECTION DATE
LEOMINSTER	**7L0**	**L-08-023**	**L08023-7L0-DOT-RRO**	**JUL 18, 2017**

REMARKS

irder #2 has heavy paint peeling and surface rusting in many areas throughout, heaviest to the bottom ange and the outside of the web. The outside of the web has isolated areas of heavy rust flaking 4 to 10 ch in diameter with as little as 0.32 inch remaining. (Original web plate thickness 0.50 inch). **See photo).** These areas area located withing the bottom 2/3 of the web. There is minor rust flaking along the entire ngth of the outside face of girder #2, below the deck line. The top half of the inside of the girder has oderate to heavy rust flaking to the interior vertical stiffeners and knee braces at the deck interface. The st flaking encompasses the bottom 2 to 4 inches of the stiffeners.

<u>em 59.7 - Conn Plt's, Gussets & Angles</u>
ee Item #59.14.

<u>em 59.9 - Bearing Devices</u>
ne bearings have minor paint peeling and surface rusting. **See photos 11 and 12.** The girder #2 bearing the North abutment has a raised nut on the East side.

<u>em 59.11 - Rivets & Bolts</u>
ee Item #59.14.

<u>em 59.14 - Paint/Coating</u>
ne paint system has many areas of heavy paint peeling, exposing strcutural steel. **See photos 3, 4, 6, 10, I.** The worse areas are to the bottom flanges of girders, floorbeams, stringers, and the outside face of oth girders. There is minor rust flaking along the entire length of the outside face of girder #2, below the eck line.

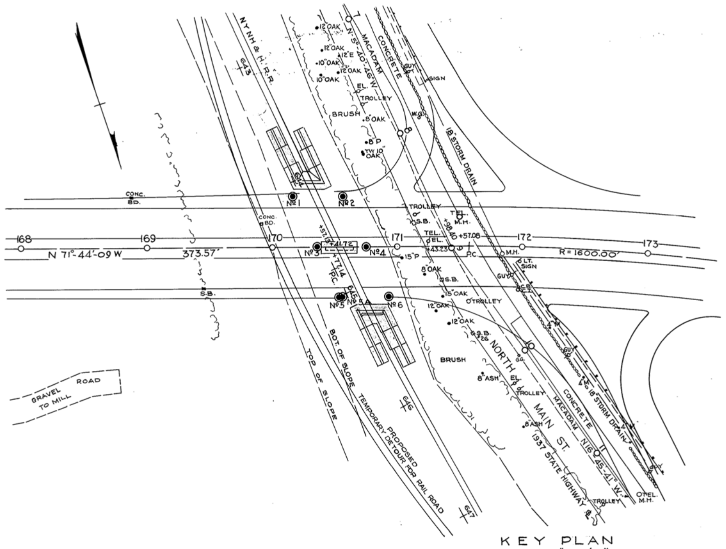
<u>EM 60 - SUBSTRUCTURE</u>

<u>em 60.1 - Abutments</u>
<u>em 60.1.b - Bridge Seats</u>
ee Item 60.1.d.

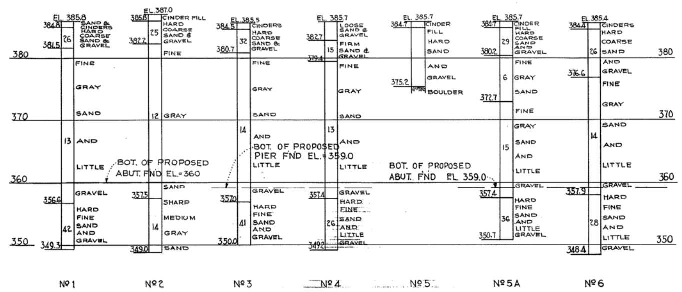
<u>em 60.1.d - Breastwalls</u>
nere is a 5 foot high x up to 3.5 foot wide area of moderate scaling under girder #1 to the North breastwall. nere is an approximately 3 foot diagonal crack, up to 1/8 inch wide, extending down from the bridge seat, the West end of the North breastwall. **See photo 13.** The East end of the South breastwall has a 3 foot x foot area of map cracking with efflorescence.

<u>em 60.1.e - Wingwalls</u>
ne Northwest and the Southeast wingwalls have an approximately 8 foot high x 6 foot wide areas of minor moderate map cracking, minor scaling, and minor efflorescence adjacent to the emblem.

PUB. ROAD DIV. Nº.	STATE	FED. AID PROJ. Nº.	FISCAL YEAR	SHEET Nº	TOTAL SHEETS
1	MASS	FG 273(8)	194	68	272



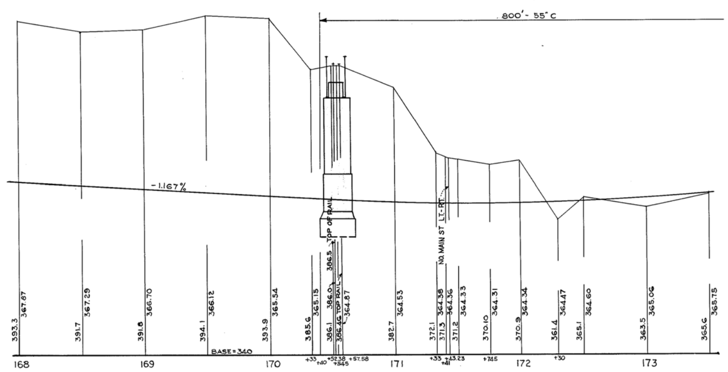
KEY PLAN
SCALE 1" = 40'-0"



BORING DATA
SCALE: 1" = 8'-0"
BORINGS TAKEN MAY, 1936.

BORING NOTES

LOCATION OF BORINGS SHOWN ON KEY PLAN THUS ◉ Nº 1
BORINGS TAKEN FOR PURPOSE OF DESIGN AND SHOW CONDITIONS AT BORING POINTS ONLY BUT DO NOT NECESSARILY SHOW NATURE OF MATERIAL TO BE ENCOUNTERED IN CONNECTION WITH THE CONSTRUCTION OF THE BRIDGE.
FIGURES IN COLUMNS INDICATE BLOWS PER FOOT ON 1" PIPE.
SAMPLES FROM BORINGS MAY BE SEEN AT THE OFFICE OF THE BRIDGE ENGINEER, MR. R.O. SPOFFORD, ROOM 609



PROFILE
HOR. SCALE 1" = 40'-0"
VERT SCALE 1" = 8'-0"

GENERAL NOTES

FOUNDATIONS:
MAY BE ALTERED IF NECESSARY TO SUIT CONDITIONS OF CONSTRUCTION

DESIGN:
IN ACCORDANCE WITH AMERICAN RAILWAY ENGINEERS ASSOCIATION SPECIFICATION FOR COOPERS E-72 LOADING.

BENCH MARK:
STA. 170+50, 118' LT. R.R. SPIKE 15" OAK NEAR POLE #54 ON NO. MAIN ST. EL. 379.86 SEA LEVEL DATUM OF 1929.

ESTIMATED QUANTITIES
(NOT GUARANTEED)

BRIDGE EXCAVATION	1000	CU. YDS.
CLASS B ROCK EXCAVATION	30	CU. YDS.
GRAVEL BORROW	520	CU. YDS.
STRIPPING GRAVEL PITS	50	CU. YDS.
GLASS A CEM. CONC. MASONRY	78	CU. YDS.
CLASS B CEM. CONC. MASONRY	1,090	CU. YDS.
STEEL REINF. FOR STRUCTORES	50,600	POUNDS
STRUCTURAL STEEL	174,000	POUNDS
MEMBRANE WATERPROOFING (3 PLY)	220	SQ. YDS.
BITUMINOUS DAMP-PROOFING	250	SQ. YDS.
WATERPROOFING PROTECTIVE COURSE	220	SQ. YDS.
ORNAMENTS	4	EACH
GRANITE CAP STONE	193	LIN. FT.
GRANITE FACING	675	SQ. FT.

ADVERTISING 22 OCT. 1949	CONSTRUCTION 22 OCT. 1949.

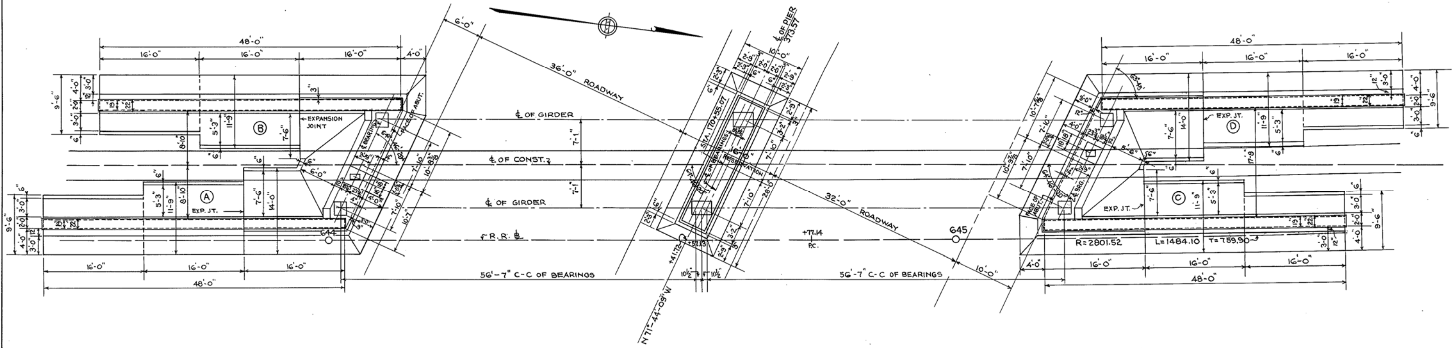
THE COMMONWEALTH OF MASSACHUSETTS
PROPOSED BRIDGE
LEOMINSTER
CONCORD - WESTMINSTER HIGHWAY
STA. 170+55.07 UNDER N.Y. N.H. & H.R.R.
SCALES AS NOTED
OFFICE OF
DEPARTMENT OF PUBLIC WORKS
100 NASHUA ST. – BOSTON, MASS.

R. O. Spofford — BRIDGE ENGINEER
[illegible] — CHIEF ENGINEER

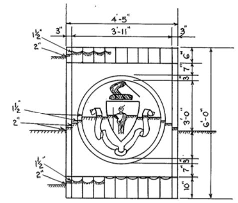
DESIGNED BY G.D.V. TRACED BY N.P.B. R.J.R. C.J.M. R.P.B. CHECKED BY P.C.M.

SHEET 1 OF 5 SHEETS BRIDGE Nº L-8-23

PUB. ROAD DIV. No	STATE	FED AID PROJ. No	FISCAL YEAR	SHEET No	TOTAL SHEETS
1	MASS	F.G.273(8)	194	69	272

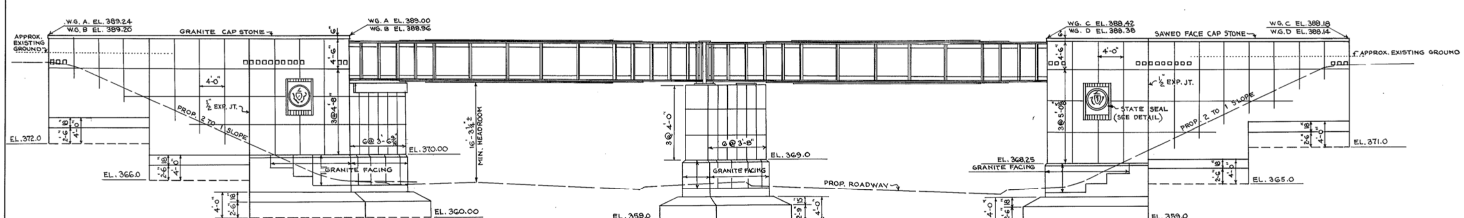


PLAN
SCALE 1/8" = 1'-0"



DETAIL OF PRECAST ORNAMENT
SCALE 1/2" = 1'-0"

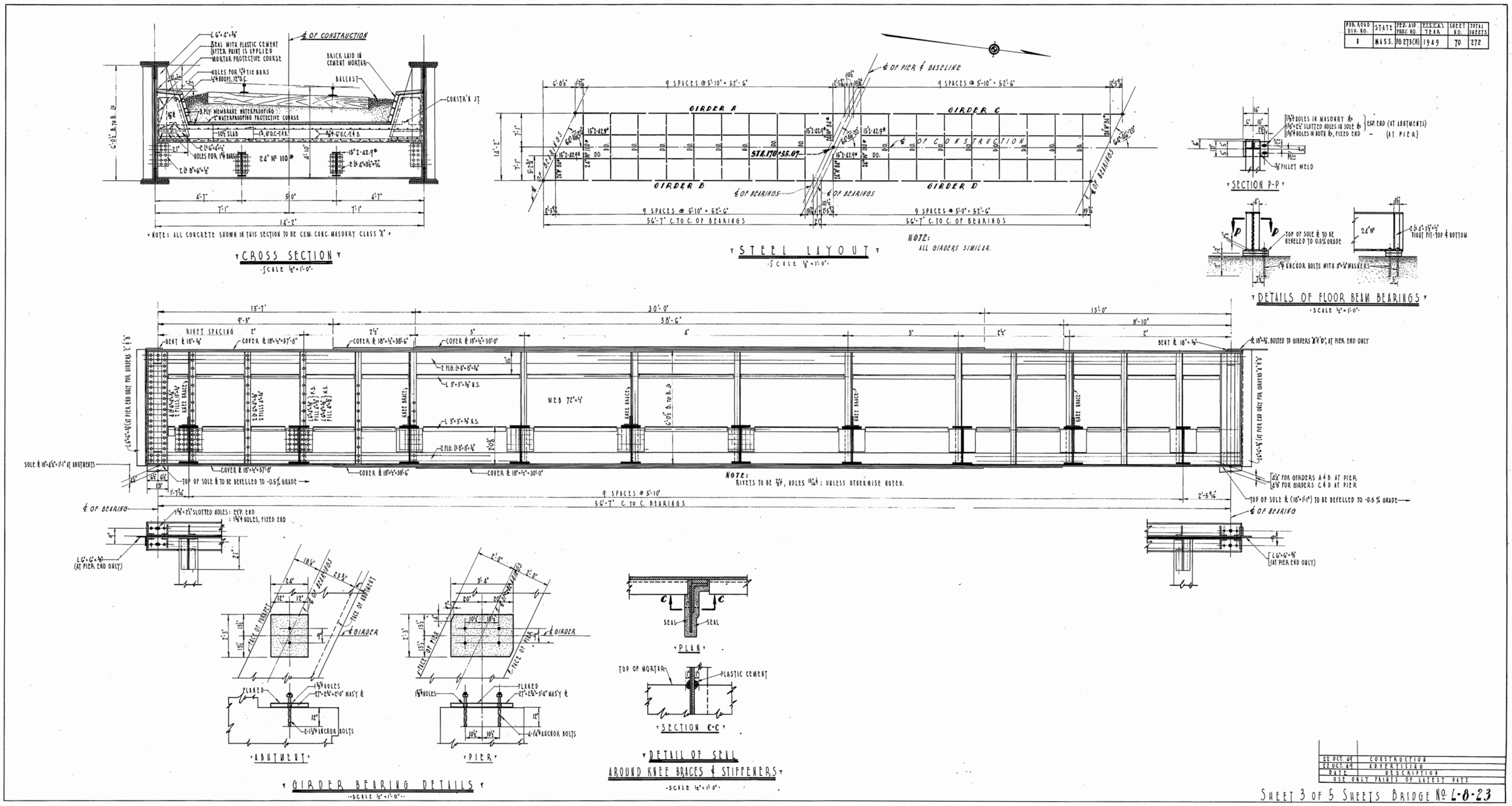
NOTE: ORNAMENT TO BE CONCRETE OF A TEXTURE AND COLOR OF SURROUNDING CONCRETE IN WALL.
MODEL SHALL BE SUBMITTED FOR APPROVAL AFTER WHICH PLASTER WASTE MOULDS SHALL BE MADE.
ORNAMENT MAY BE CAST AND THEN INSERTED IN WALL FORMS AS INDICATED OR MAY BE POURED INTEGRAL WITH THE WALL.



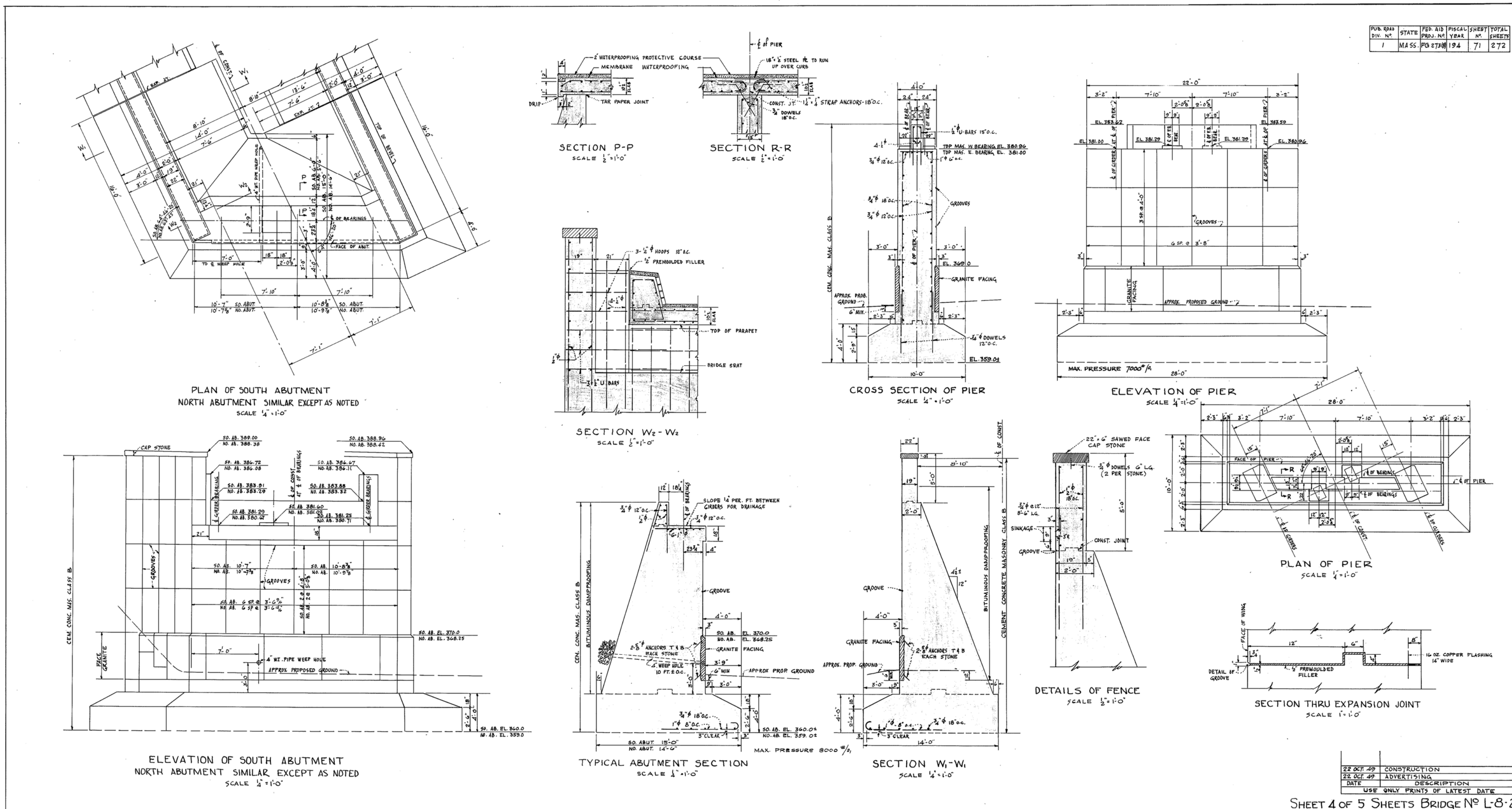
EASTERLY ELEVATION
SCALE 1/8" = 1'-0"

OCT. 22, 49	CONSTRUCTION
OCT. 22 49	ADVERTISING
DATE	DESCRIPTION
USE ONLY PRINTS OF LATEST DATE	

SHEET 2 OF 5 SHEETS BRIDGE No L-8-23

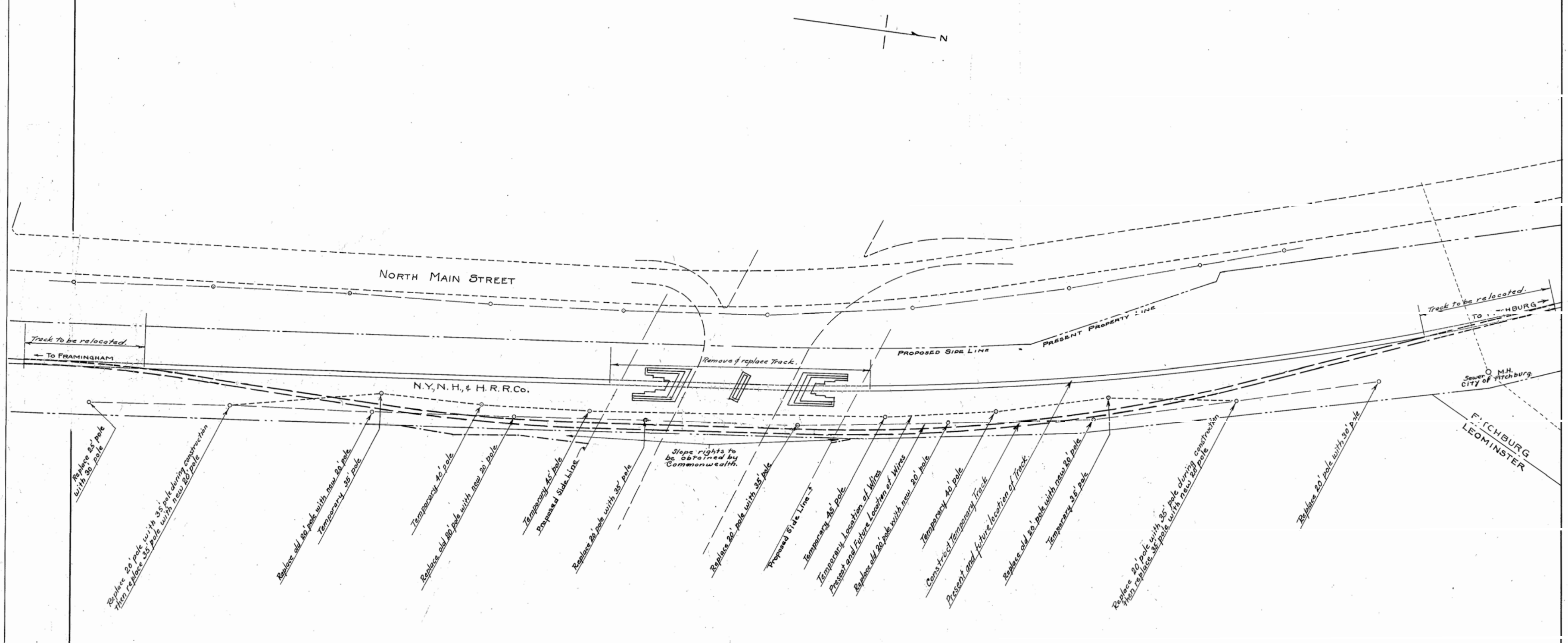


FED. ROAD DIV. NO. | STATE | FED. AID PROJ. NO. | FISCAL YEAR | SHEET NO. | TOTAL SHEETS
1 | MASS. | FG-273C(8) | 1949 | 70 | 272
CROSS SECTION
SCALE ½" = 1'-0"
NOTE: ALL CONCRETE SHOWN IN THIS SECTION TO BE CEM. CONC. MASONRY CLASS "A"
STEEL LAYOUT
SCALE ⅛" = 1'-0"
NOTE: ALL GIRDERS SIMILAR.
GIRDER A
GIRDER B
GIRDER C
GIRDER D
SECTION P-P
DETAILS OF FLOOR BEAM BEARINGS
SCALE ½" = 1'-0"
NOTE: RIVETS TO BE ⅞", HOLES 15/16"; UNLESS OTHERWISE NOTED.
ABUTMENT
PIER
GIRDER BEARING DETAILS
SCALE ½" = 1'-0"
PLAN
SECTION C-C
DETAIL OF SEAL AROUND KNEE BRACES & STIFFENERS
SCALE ½" = 1'-0"
22 OCT. 49 CONSTRUCTION
DATE DESCRIPTION
USE ONLY PRINTS OF LATEST DATE
SHEET 3 OF 5 SHEETS BRIDGE No. L-8-23



SHEET 5 IS ON CLOTH – PART OF ORIG

PUB. RD. DIV. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	MASS	FG 273 (8)	19	72	272



Railroad Work

Care of Wires during construction and rebuilding after completion of bridge.

Building temporary track around site of bridge.

(a) The Commonwealth shall obtain the necessary slope rights for the temporary track.

(b) The General Contract shall include the rough grading for the temporary track.

Work By Railroad:
- Relocate present track to connect with temporary track.
- Remove and rebuild portion of track at site of Bridge.
- Bond temporary track and main line track.
- Surface track during and after construction.
- Provide temporary telephone.
- Engineering and inspection.

Scale 1" = 40'-0"

DATE	DESCRIPTION
22 OCT. '49	CONSTRUCTION
22 OCT. '49	ADVERTISING

USE ONLY PRINTS OF LATEST DATE

SHEET 5 OF 5 SHEETS BRIDGE Nº L-8-23

Appendix C: Site Visit

Site Visit

Location: Existing Railroad Bridge over Route 2 in Leominster, MA (near the Double Tree Hotel)
Date of Site Visit: October 28, 2018
Attendees: Nicole Barrett, Jonathan Benoit, Alex Duffield, Hadi Hamdan, Isabella Morrison Ouellette, Lauren Flanders, PE, and Betsy Kirtland, EIT

Site Visit Notes:

- Traffic is very loud from bridge
- Lots of graffiti on the bridge and temporary barriers
- The paint on the girders is lead - needs to be treated as hazardous waste
- The ballast is contaminated - also needs to be treated as hazardous waste
- Route 2 eventually being widened underneath bridge
- Height of bridge should be at least the same clearance or better as the other bridges in the corridor if it is below the standard
- Structurally, the bridge appears to be in good condition
- Girder height is very low, a good fence will be necessary to replace barriers and fencing
- Space between abutment and girders poses a danger to pedestrians, especially when barriers are removed
- Fence restricts access to Route 2 by the bridge, could not get close to abutments, some map cracking is visible
- Salt from Route 2 is often thrown up against the abutments and increases the wearing of concrete
- The area surrounding the bridge is somewhat limited, but there is some space for staging of construction vehicles
- There are no obstructions behind the existing abutments

Photos: 13 photos on the following pages

Photo 1: Looking north, path on bridge with tree growing through ballast. Barriers with fencing span the length of the bridge.

Photo 2: Looking southwest, view of bridge from shoulder of Route 2.

Photo 3: Underside of bridge from shoulder of Route 2, looking southwest.

Photo 4: View from bridge overlooking Route 2, looking west.

Photo 5: Existing ballast on bridge.

Photo 6: View from bridge looking east onto Route 2 over collapsed fence.

Photo 7: Looking south, collapsed bridge next to low girder, connecting to concrete abutment.

Photo 8: Path leading to bridge, looking north.


Photo 9: Tree overgrowth onto structure, looking west.

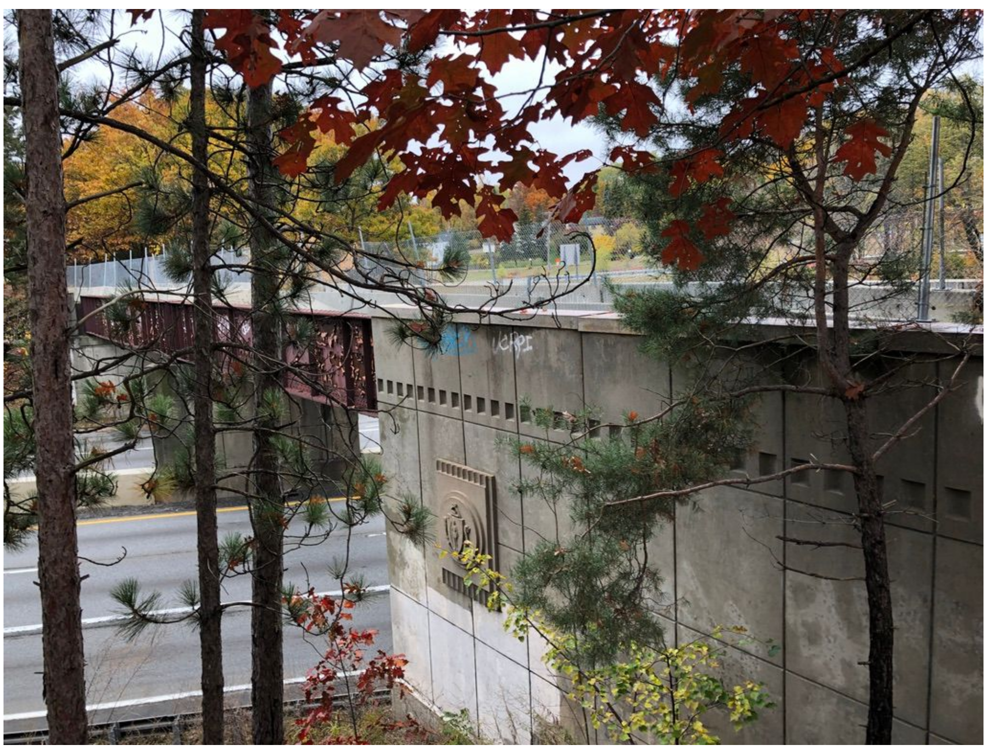


Photo 10: Looking southwest at the abutment on the north side of the bridge.

Photo 11: Looking west at the side of the north abutment of the bridge.

Photo 12: Connection of girders at center of bridge.

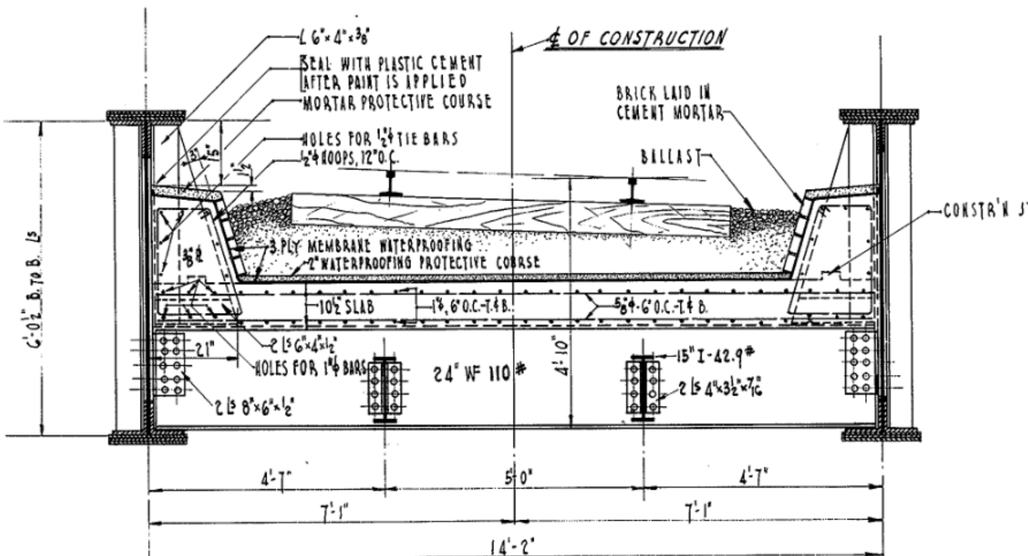
Photo 13: Looking west, map cracking on north bridge abutment.

Appendix D: Existing Bridge Load Rating Factors

Load Development:

Refernces:

- AASHTO LRFD Guide Specifications for the Desing of Pedestrian Bridges, 1st Edition, 2009 w/ 2015 interims
- AASHTO Standard Specidications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, Fifth Edition, 2009
- AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2017
- Manual for Bride Evaluation, Third Edition, 2018



NOTE: ALL CONCRETE SHOWN IN THIS SECTION TO BE CEM. CONC. MASONRY CLASS "A"

CROSS SECTION

SCALE 1/2" = 1'-0"

$L_{span} := 56ft + 7in = 56.583 \cdot ft$ Length of a single span of bridge

$L_{bridge} := 115ft$ Length of entire bridge

$W_{bridge} := 14ft + 2in = 14.167 \cdot ft$ Width of bridge

$No_{floorbeams} := 18$

$Spacing_{floorbeams} := 5ft + 10in$

Dead Load:

Girders:

$$Girder_{web} := 72in\cdot\frac{1}{2}in = 0.25\cdot ft^2$$

$$Girder_{flange} := 18in\cdot\frac{1}{2}in = 0.063\cdot ft^2$$

$$Area_{girder} := Girder_{web} + 2Girder_{flange} = 0.375\cdot ft^2$$

$$Weight_{girder} := Area_{girder}\cdot L_{span}\cdot 0.490\,\frac{kip}{ft^3} = 10.397\cdot kip$$

AASHTO LRFD Table 3.5.1.1

$$DL_{girder} := 1.06\left[\frac{(4\cdot Weight_{girder})}{L_{bridge}}\right] = 0.383\cdot klf$$

Cover Plates:

$$Volume_{cp1} := 18in\cdot 0.5in\cdot 57.67ft = 3.604\cdot ft^3$$

$$Volume_{cp2} := 18in\cdot 0.5in\cdot 38.5ft = 2.406\cdot ft^3$$

$$Volume_{total} := (Volume_{cp1} + Volume_{cp2})\cdot 2\cdot 4 = 48.085\cdot ft^3$$

$$Weight_{cp} := Volume_{total}\cdot 0.49318\,\frac{kip}{ft^3} = 23.715\cdot kip$$

$$DL_{cp} := 1.06\left(\frac{Weight_{cp}}{L_{bridge}}\right) = 0.219\cdot klf$$

Beams:

$No_{beams\_110} := 18$

$No_{beams\_84} := 4$

$L_{beam\_110} := 14ft + 2in = 14.167 \cdot ft$

$L_{beam\_84} := 5ft + 2.75in = 5.229 \cdot ft$

$$Weight_{beam\_110} := No_{beams\_110} \cdot L_{beam\_110} \cdot 0.110 \frac{kip}{ft} = 28.05 \cdot kip$$

$$Weight_{beam\_84} := No_{beams\_84} \cdot L_{beam\_84} \cdot 0.084 \frac{kip}{ft} = 1.757 \cdot kip$$

$$DL_{beam} := 1.06 \left(\frac{Weight_{beam\_110} + Weight_{beam\_84}}{L_{bridge}} \right) = 0.275 \cdot klf$$

$$Weight_{beam\_fb} := \frac{\left(Weight_{beam\_110} + Weight_{beam\_84}\right)}{No_{floorbeams}} = 1.656 \cdot kip$$

$$DL_{beam\_fb} := \frac{Weight_{beam\_fb}}{W_{bridge}} = 0.117 \cdot klf$$

Diaphragms:

$No_{dia} := 36$

$L_{dia} := 5ft + 10in = 5.833 \cdot ft$

$$Weight_{dia} := No_{dia} \cdot L_{dia} \cdot 0.0429 \frac{kip}{ft} = 9.009 \cdot kip$$

$$DL_{dia} := 1.06 \left(\frac{Weight_{dia}}{L_{bridge}} \right) = 0.083 \cdot klf$$

$$DL_{dia\_fb} := 1.06 \left[\frac{\left(\frac{Weight_{dia}}{No_{floorbeams}} \right)}{W_{bridge}} \right] = 0.037 \cdot klf$$

Slab:

$H_{slab} := 10.5in$

$Area_{slab} := H_{slab} \cdot W_{bridge} = 12.396 \cdot ft^2$

$DL_{slab} := Area_{slab} \cdot 0.150 \dfrac{kip}{ft^3} = 1.859 \cdot klf$

$Weight_{slab} := Area_{slab} \cdot L_{bridge} \cdot 0.150 \dfrac{kip}{ft^3} = 213.828 \cdot kip$

$$DL_{slab\_fb} := \frac{\left(\dfrac{Weight_{slab}}{No_{floorbeams}}\right)}{W_{bridge}} = 0.839 \cdot klf$$

Curb:

$H_{curb} := 22.5in$

$W_{curb} := 21in$

$Area_{curb} := H_{curb} \cdot W_{curb} = 3.281 \cdot ft^2$

$Weight_{curb} := 2 \cdot Area_{curb} \cdot L_{bridge} \cdot 0.150 \dfrac{kip}{ft^3} = 113.203 \cdot kip$

$$DL_{curb} := \frac{Weight_{curb}}{L_{bridge}} = 0.984 \cdot klf$$

Ballast:

*Assume weight of ballast is 150 pcf to be conservative

$$H_{ballast} := H_{curb} = 22.5 \cdot in$$

$$W_{ballast} := W_{bridge} - 2 \cdot W_{curb} = 10.667 \cdot ft$$

$$Area_{ballast} := H_{ballast} \cdot W_{ballast} = 20 \cdot ft^2$$

$$DL_{ballast} := Area_{ballast} \cdot 0.150 \frac{kip}{ft^3} = 3 \cdot klf$$

Ballast for Floorbeams:

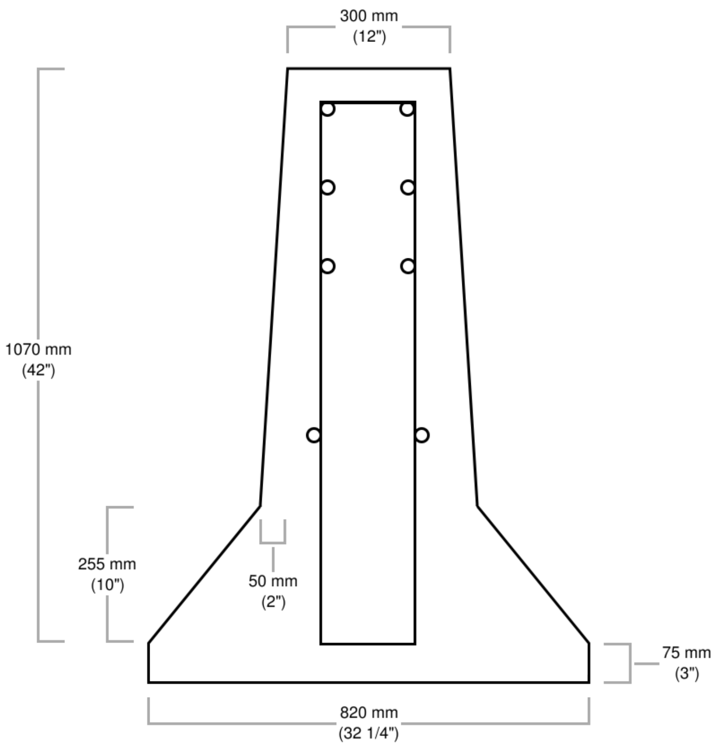
Assume curb is part of ballast for floorbeams to make it a uniform distributed load, as they are the same height and assumed weight

$$Weight_{ballast} := H_{ballast} \cdot W_{bridge} \cdot L_{bridge} \cdot 0.150 \frac{kip}{ft^3} = 458.203 \cdot kip$$

$$DL_{ballast\_fb} := \frac{\left(\frac{Weight_{ballast}}{No_{floorbeams}} \right)}{W_{bridge}} = 1.797 \cdot klf$$

Concrete Barriers:

*Assume area = 702 in^2, and that there are barriers on both sides of entire length of bridge



$$Area_{barrier} := 702\,in^2$$

$$DL_{barrier} := 2 \cdot Area_{barrier} \cdot 0.150 \frac{kip}{ft^3} = 1.462 \cdot klf$$

$$Weight_{barrier} := DL_{barrier} \cdot L_{bridge} = 168.187 \cdot kip$$

$$DL_{barrier\_fb} := \frac{\left(\frac{Weight_{barrier}}{No_{floorbeams}}\right)}{W_{bridge}} = 0.66 \cdot klf$$

Live Load:

Pedestrian Load:

$Area_{bridge} := L_{bridge} \cdot W_{bridge} = 1.629 \times 10^{3} \cdot ft^{2}$

$PL := 0.090 ksf$ AASHTO Pedestrian 3.1

$Weight_{pedestrian} := Area_{bridge} \cdot PL = 146.625 \cdot kip$

$$LL_{pedestrian} := \frac{Weight_{pedestrian}}{L_{bridge}} = 1.275 \cdot klf$$

$$M_{pedestrian} := \frac{\left(LL_{pedestrian} \cdot L_{span}{}^{2}\right)}{8} = 510.267 \cdot kip \cdot ft$$

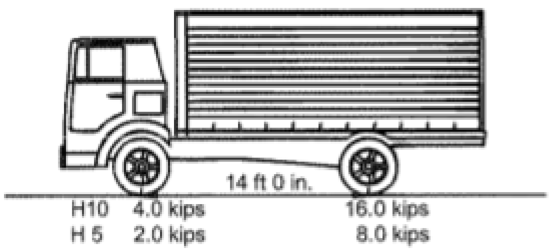
$$LL_{pedestrian\_fb} := \frac{\left(\frac{Weight_{pedestrian}}{No_{floorbeams}}\right)}{W_{bridge}} = 0.575 \cdot klf$$

$$M_{pedestrian\_fb} := \frac{\left(LL_{pedestrian\_fb} \cdot W_{bridge}{}^{2}\right)}{8} = 14.425 \cdot kip \cdot ft$$

Vehicle Load:

AASHTO Pedestrian 3.2

Clear Deck Width	Design Vehicle
7 to 10 ft	H5
Over 10 ft	H10



$P_1 := 4\text{kip}$ $\quad b := \dfrac{L_{span}}{2} = 28.292\cdot\text{ft}$ Distance from end of span to 16 kip wheel

$P_2 := 16\text{kip}$ $\quad a := L_{span} - b - 14\text{ft} = 14.292\cdot\text{ft}$ Distance from end of span to 4 kip wheel

$$R_1 := \frac{\left[P_1\cdot\left(L_{span} - a\right) + P_2\cdot b\right]}{L_{span}} = 10.99\cdot\text{kip}$$

$$R_2 := \frac{\left[P_1\cdot a + P_2\cdot\left(L_{span} - b\right)\right]}{L_{span}} = 9.01\cdot\text{kip}$$

$$M_{vehicle} := \left| \begin{array}{l} \left(R_1\cdot a\right) \ \text{ if } \ R_1 < P_1 \\ \left(R_2\cdot b\right) \ \text{ if } \ R_2 < P_2 \end{array} \right. = 254.917\cdot\text{kip}\cdot\text{ft}$$

Vehicle Load on Floorbeams:

$P_{wheel} := 8kip$

If equally spaced on center of bridge:

$$a := \frac{(W_{bridge} - 6ft)}{2} = 4.083 \cdot ft$$

$M_{vehicle\_fb} := P_{wheel} \cdot a = 32.667 \cdot kip \cdot ft$

Equestrian Load:

$EQ := 1kip$ AASHTO Pedestrian 3.3

Assume 1.0 kip over square 4" x 4" area, or concentrated load, on center of span

$$M_{EQ} := \frac{(EQ \cdot L_{span})}{4} = 14.146 \cdot kip \cdot ft$$

For floorbeams:

$$M_{EQ\_fb} := \frac{(EQ \cdot W_{bridge})}{4} = 3.542 \cdot kip \cdot ft$$

Pedestrian Load governs for girders:

$LL := LL_{pedestrian} = 1.275 \cdot klf$

Vehicle Load governs for floorbeams

Wind Load:

Horizontal Wind Load:

$V := 110mph$ — AASHTO Signs 3.8.1

Not a special wind region
Not an elevated location

$I_r := 1.15$ — AASHTO LRFD Pedestrian 3.4

$K_z := 0.94$ — AASHTO Signs Table 3-5

$G := 1.14$ — AASHTO Signs 3.8.5

$C_d := 2.0$ — AASHTO Signs Table 3.6
For trusses

$$P_z := \left[0.00256 \cdot K_z \cdot G \cdot \left(\frac{V}{mph}\right)^2 \cdot I_r \cdot C_d\right] \cdot psf = 76.346 \cdot psf$$

AASTHO Signs 3.8.1

$H_{horiz} := 72.5in$

$A_{horiz} := H_{horiz} \cdot L_{bridge} = 694.792 \cdot ft^2$

$Weight_{wind\_horiz} := P_z \cdot A_{horiz} = 53.045 \cdot kip$

$$DL_{wind\_horiz} := \frac{2Weight_{wind\_horiz}}{L_{bridge}} = 0.923 \cdot klf$$

Vertical Wind Load:

$P_v := 0.020\text{ksf}$ AASHTO Pedestrian 3.4

$A_{deck} := L_{bridge} \cdot W_{bridge} = 1.629 \times 10^3 \cdot ft^2$

$Weight_{vert} := P_v \cdot A_{deck} = 32.583 \cdot kip$

$WL := \frac{Weight_{vert}}{L_{bridge}} = 0.283 \cdot klf$

$WL_{windward} := \frac{1}{4} \cdot WL = 0.071 \cdot klf$

$WL_{leeward} := \left(\frac{3}{4}\right) \cdot WL = 0.212 \cdot klf$

$WL_{vert} := \max\left(WL_{windward}, WL_{leeward}\right) = 0.212 \cdot klf$

Fatigue Load:

$P_{NW} := \left(5.2 \cdot C_d \cdot I_r\right) \cdot psf = 11.96 \cdot psf$ AASHTO Signs 11.7.3

$P_{TG} := \left(18.8 \cdot C_d \cdot I_r\right) \cdot psf = 43.24 \cdot psf$ AASTHO Signs 11.7.4

$P_{fatigue} := P_{NW} + P_{TG} = 0.055 \cdot ksf$

$LL_{fatigue} := P_z \cdot \left(2 \cdot H_{horiz}\right) = 0.923 \cdot klf$

Load Combinations:

AASHTO LRFD Table 3.4.1-1

$DC := DL_{girder} + DL_{cp} + DL_{beam} + DL_{dia} + DL_{slab} + DL_{curb} + DL_{barrier} = 5.266 \cdot klf$

$DW := DL_{ballast} = 3 \cdot klf$

$LL = 1.275 \cdot klf$

$WS := WL_{vert} = 0.212 \cdot klf$

$\gamma_{DC} := 1.25$ AASHTO LRFD Table 3.4.1-2

$\gamma_{DW} := 1.50$

$\gamma_{EQ} := 0.5$ AASHTO LRFD 3.4.1

<u>Shear on Bridge:</u>

$$V_{DC} := \frac{\left(DC \cdot L_{span}\right)}{2} = 148.983 \cdot kip$$

$$V_{DW} := \frac{\left(DW \cdot L_{span}\right)}{2} = 84.875 \cdot kip$$

$$V_{LL\_pedestrian} := \frac{\left(LL_{pedestrian} \cdot L_{span}\right)}{2} + \frac{EQ}{2} = 36.572 \cdot kip$$

$$V_{LL\_vehicle} := \max\left(R_1, R_2\right) = 10.99 \cdot kip$$

$$V_{LL} := \max\left(V_{LL\_pedestrian}, V_{LL\_vehicle}\right) = 36.572 \cdot kip$$

$$V_{WS} := \frac{\left(WS \cdot L_{span}\right)}{2} = 6.012 \cdot kip$$

$$V_{LL\_fatigue} := \frac{\left(LL_{fatigue} \cdot L_{span}\right)}{2} = 26.099 \cdot kip$$

$Strength_I := \gamma_{DC} \cdot V_{DC} + \gamma_{DW} \cdot V_{DW} + 1.75 \cdot V_{LL} = 377.542 \cdot kip$

$Strength_{III} := \gamma_{DC} \cdot V_{DC} + \gamma_{DW} \cdot V_{DW} + 1.00 \cdot V_{WS} = 319.553 \cdot kip$

$Extreme\_Event_I := 1.00V_{DC} + 1.00V_{DW} + \gamma_{EQ} \cdot V_{LL} = 252.144 \cdot kip$

$Extreme\_Event_{II} := 1.00V_{DC} + 1.00V_{DW} + 0.5V_{LL} = 252.144 \cdot kip$

$Service_I := 1.00V_{DC} + 1.00V_{DW} + 1.00V_{LL} + 1.00V_{WS} = 276.442 \cdot kip$

$Service_{II} := 1.00V_{DC} + 1.00V_{DW} + 1.30V_{LL} = 281.401 \cdot kip$

$Service_{IV} := 1.00V_{DC} + 1.00V_{DW} + 1.00V_{WS} = 239.87 \cdot kip$

$Fatigue_I := 1.75V_{LL\_fatigue} = 45.674 \cdot kip$

Strength I controls

Moments on Bridge:

$$M_{DC} := \frac{\left(\left(\frac{DC}{2} \cdot L_{span}^{2}\right)\right)}{8} = 1.054 \times 10^{3} \cdot kip \cdot ft$$

$$M_{DW} := \frac{\left(\frac{DW}{2} \cdot L_{span}^{2}\right)}{8} = 600.314 \cdot kip \cdot ft$$

$$M_{LL\_pedestrian} := \frac{M_{pedestrian}}{2} + \frac{M_{EQ}}{2} = 262.206 \cdot kip \cdot ft$$

$$M_{LL\_vehicle} := \frac{M_{vehicle}}{2} = 127.458 \cdot kip \cdot ft$$

$$M_{LL} := \max\left(M_{LL\_pedestrian}, M_{LL\_vehicle}\right) = 262.206 \cdot kip \cdot ft$$

$$M_{WS} := \frac{\left(\frac{WS}{2} \cdot L_{span}^{2}\right)}{8} = 42.522 \cdot kip \cdot ft$$

$$M_{LL\_fatigue} := \frac{\left(\frac{LL_{fatigue}}{2} \cdot L_{span}^{2}\right)}{8} = 184.599 \cdot kip \cdot ft$$

$Strength_I := \gamma_{DC} \cdot M_{DC} + \gamma_{DW} \cdot M_{DW} + 1.75 \cdot M_{LL} = 2.677 \times 10^3 \cdot kip \cdot ft$

$Strength_{III} := \gamma_{DC} \cdot M_{DC} + \gamma_{DW} \cdot M_{DW} + 1.00 \cdot M_{WS} = 2.26 \times 10^3 \cdot kip \cdot ft$

$Extreme\_Event_I := 1.00M_{DC} + 1.00M_{DW} + \gamma_{EQ} \cdot M_{LL} = 1.785 \times 10^3 \cdot kip \cdot ft$

$Extreme\_Event_{II} := 1.00M_{DC} + 1.00M_{DW} + 0.5M_{LL} = 1.785 \times 10^3 \cdot kip \cdot ft$

$Service_I := 1.00M_{DC} + 1.00M_{DW} + 1.00M_{LL} + 1.00M_{WS} = 1.959 \times 10^3 \cdot kip \cdot ft$

$Service_{II} := 1.00M_{DC} + 1.00M_{DW} + 1.30M_{LL} = 1.995 \times 10^3 \cdot kip \cdot ft$

$Service_{IV} := 1.00M_{DC} + 1.00M_{DW} + 1.00M_{WS} = 1.697 \times 10^3 \cdot kip \cdot ft$

$Fatigue_I := 1.75M_{LL\_fatigue} = 323.049 \cdot kip \cdot ft$

Strength I controls

$\varphi_c := 1.00$

Condition Factor
MBE 6A.4.2.3-1, Good or Satisfactory

$\varphi_s := 0.9$

System Factor
MBE 6A.4.2.3-1, Riveted Members in Two-Girder

Plastic Moment Capacity of Girders:

AASHTO LRFD Table D6.1-1

$D := 72in$ $t_w := \frac{1}{2}in$

Dimensions of girder

$t_c := \frac{1}{2}in$ $b_c := 18in$

$t_t := \frac{1}{2}in$ $b_t := 18in$

$F_y := 33ksi$

Yield strength of steel, MBE Table 6A.6.2.1-1

$P_c := F_y \cdot b_c \cdot t_c = 297 \cdot kip$

$P_w := F_y \cdot D \cdot t_w = 1.188 \times 10^3 \cdot kip$

$P_t := F_y \cdot b_t \cdot t_t = 297 \cdot kip$

$$Y_{bar} := \left(\frac{D}{2}\right)\left[\frac{(P_t - P_c)}{P_w} + 1\right] = 36 \cdot in$$

$d_c := Y_{bar} + \frac{1}{4}in = 36.25 \cdot in$

Distance from center of compression flange to Y.bar

$d_t := Y_{bar} + \frac{1}{4}in = 36.25 \cdot in$

Distance from center of tension flange to Y.bar

$d_w := 0in$

$$M_p := \left(\frac{P_w}{2 \cdot D}\right)\left[Y_{bar}^{\ 2} + (D - Y_{bar})^2\right] + (P_c \cdot d_c + P_t \cdot d_t) = 3.576 \times 10^3 \cdot kip \cdot ft$$

Moments on Bridge:

$$M_{DC} := \frac{\left(\left(\frac{DC}{2}\cdot L_{span}{}^{2}\right)\right)}{8} = 1.054 \times 10^{3}\cdot kip\cdot ft$$

$$M_{DW} := \frac{\left(\frac{DW}{2}\cdot L_{span}{}^{2}\right)}{8} = 600.314\cdot kip\cdot ft$$

$$M_{LL\_pedestrian} := \frac{M_{pedestrian}}{2} + \frac{M_{EQ}}{2} = 262.206\cdot kip\cdot ft$$

$$M_{LL\_vehicle} := \frac{M_{vehicle}}{2} = 127.458\cdot kip\cdot ft$$

$$RF_{girder\_pedestrian} := \frac{\left(\varphi_c\cdot\varphi_s\cdot M_p - \gamma_{DC}\cdot M_{DC} - \gamma_{DW}\cdot M_{DW}\right)}{1.75\cdot M_{LL\_pedestrian}} = 2.182$$

$$RF_{girder\_vehicle} := \frac{\left(\varphi_c\cdot\varphi_s\cdot M_p - \gamma_{DC}\cdot M_{DC} - \gamma_{DW}\cdot M_{DW}\right)}{1.75\cdot M_{LL\_vehicle}} = 4.488$$

Plastic Moment Capacity Floorbeam

AASHTO LRFD Table D6.1-1

$t_c := 0.855in$

$t_t := 0.855in$

$D := 24.16in - t_c - t_t = 22.45 \cdot in$

$t_w := 0.510in$

$b_c := 12.042in$

$b_t := 12.042in$

Dimensions of beam
24 WF 110
AISC 1948

$F_y := 33ksi$

Yield strength of steel, MBE Table 6A.6.2.1-1

$P_c := F_y \cdot b_c \cdot t_c = 339.765 \cdot kip$

$P_w := F_y \cdot D \cdot t_w = 377.834 \cdot kip$

$P_t := F_y \cdot b_t \cdot t_t = 339.765 \cdot kip$

$$Y_{bar} := \left(\frac{D}{2}\right)\left[\frac{\left(P_t - P_c\right)}{P_w} + 1\right] = 11.225 \cdot in$$

$d_c := Y_{bar} + \frac{1}{4}in = 11.475 \cdot in$ Distance from center of compression flange to Y.bar

$d_t := Y_{bar} + \frac{1}{4}in = 11.475 \cdot in$ Distance from center of tension flange to Y.bar

$d_w := 0in$

$$M_p := \left(\frac{P_w}{2 \cdot D}\right)\left[Y_{bar}^{\ 2} + \left(D - Y_{bar}\right)^2\right] + \left(P_c \cdot d_c + P_t \cdot d_t\right) = 826.516 \cdot kip \cdot ft$$

$DC_{fb} := DL_{beam\_fb} + DL_{dia\_fb} + DL_{slab\_fb} + DL_{barrier\_fb} = 1.652 \cdot klf$

$DW_{fb} := DL_{ballast\_fb} = 1.797 \cdot klf$

$$M_{DC} := \frac{\left(DC_{fb} \cdot W_{bridge}^{\ 2}\right)}{8} = 41.454 \cdot kip \cdot ft$$

$$M_{DW} := \frac{\left(DW_{fb} \cdot W_{bridge}^{\ 2}\right)}{8} = 45.078 \cdot kip \cdot ft$$

$M_{LL\_pedestrian\_fb} := M_{pedestrian\_fb} + M_{EQ\_fb} = 17.967 \cdot kip \cdot ft$

$M_{vehicle\_fb} = 32.667 \cdot kip \cdot ft$

$$RF_{beam\_pedestrian} := \frac{\left(\varphi_c \cdot \varphi_s \cdot M_p - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}\right)}{1.75 \cdot M_{LL\_pedestrian\_fb}} = 19.86$$

$$RF_{beam\_vehicle} := \frac{\left(\varphi_c \cdot \varphi_s \cdot M_p - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}\right)}{1.75 \cdot M_{vehicle\_fb}} = 10.923$$

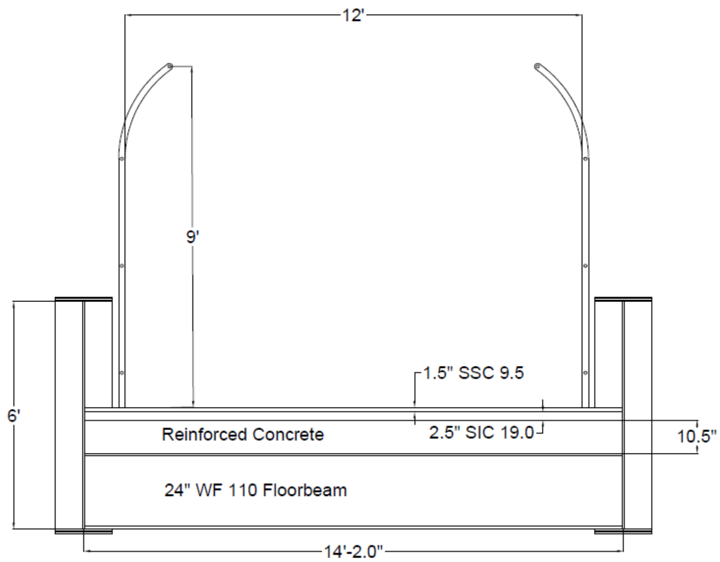
Appendix E. Repair Load Rating Factors

Repair Load

Load Development:

Refernces:

- AASHTO LRFD Guide Specifications for the Desing of Pedestrian Bridges, 1st Edition, 2009 w/ 2015 interims
- AASHTO Standard Specidications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, Fifth Edition, 2009
- AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2017
- Manual for Bride Evaluation, Third Edition, 2018



$L_{span} := 56ft + 7in = 56.583 \cdot ft$ Length of a single span of bridge

$L_{bridge} := 115ft$ Length of entire bridge

$W_{bridge} := 14ft + 2in = 14.167 \cdot ft$ Width of bridge

$No_{floorbeams} := 18$

$Spacing_{floorbeams} := 5ft + 10in$

Repair Load

Dead Load:

Girders:

$$Girder_{web} := 72in \cdot \frac{1}{2} in = 0.25 \cdot ft^2$$

$$Girder_{flange} := 18in \cdot \frac{1}{2} in = 0.063 \cdot ft^2$$

$$Area_{girder} := Girder_{web} + 2Girder_{flange} = 0.375 \cdot ft^2$$

$$Weight_{girder} := Area_{girder} \cdot L_{span} \cdot 0.490 \frac{kip}{ft^3} = 10.397 \cdot kip$$

AASHTO LRFD Table 3.5.1.1

$$DL_{girder} := 1.06 \left[\frac{(4 \cdot Weight_{girder})}{L_{bridge}} \right] = 0.383 \cdot klf$$

Cover Plates:

$$Volume_{cp1} := 18in \cdot 0.5in \cdot 57.67ft = 3.604 \cdot ft^3$$

$$Volume_{cp2} := 18in \cdot 0.5in \cdot 38.5ft = 2.406 \cdot ft^3$$

$$Volume_{total} := (Volume_{cp1} + Volume_{cp2}) \cdot 2 \cdot 4 = 48.085 \cdot ft^3$$

$$Weight_{cp} := Volume_{total} \cdot 0.490 \frac{kip}{ft^3} = 23.562 \cdot kip$$

$$DL_{cp} := 1.06 \left(\frac{Weight_{cp}}{L_{bridge}} \right) = 0.217 \cdot klf$$

Repair Load

Beams:

$No_{beams\_110} := 18$ $No_{beams\_84} := 4$

$L_{beam\_110} := 14ft + 2in = 14.167 \cdot ft$ $L_{beam\_84} := 5ft + 2.75in = 5.229 \cdot ft$

$$Weight_{beam\_110} := No_{beams\_110} \cdot L_{beam\_110} \cdot 0.110 \frac{kip}{ft} = 28.05 \cdot kip$$

$$Weight_{beam\_84} := No_{beams\_84} \cdot L_{beam\_84} \cdot 0.084 \frac{kip}{ft} = 1.757 \cdot kip$$

$$DL_{beam} := 1.06 \left(\frac{Weight_{beam\_110} + Weight_{beam\_84}}{L_{bridge}} \right) = 0.275 \cdot klf$$

$$Weight_{beam\_fb} := \frac{\left(Weight_{beam\_110} + Weight_{beam\_84}\right)}{No_{floorbeams}} = 1.656 \cdot kip$$

$$DL_{beam\_fb} := \frac{Weight_{beam\_fb}}{W_{bridge}} = 0.117 \cdot klf$$

Diaphragms:

$No_{dia} := 36$

$L_{dia} := 5ft + 10in = 5.833 \cdot ft$

$$Weight_{dia} := No_{dia} \cdot L_{dia} \cdot 0.0429 \frac{kip}{ft} = 9.009 \cdot kip$$

$$DL_{dia} := 1.06 \left(\frac{Weight_{dia}}{L_{bridge}} \right) = 0.083 \cdot klf$$

$$DL_{dia\_fb} := 1.06 \left[\frac{\left(\frac{Weight_{dia}}{No_{floorbeams}} \right)}{W_{bridge}} \right] = 0.037 \cdot klf$$

Repair Load

Slab:

$H_{slab} := 10.5in$

$Area_{slab} := H_{slab} \cdot W_{bridge} = 12.396 \cdot ft^2$

$DL_{slab} := Area_{slab} \cdot 0.150 \dfrac{kip}{ft^3} = 1.859 \cdot klf$

$Weight_{slab} := Area_{slab} \cdot L_{bridge} \cdot 0.150 \dfrac{kip}{ft^3} = 213.828 \cdot kip$

$$DL_{slab\_fb} := \frac{\left(\dfrac{Weight_{slab}}{No_{floorbeams}}\right)}{W_{bridge}} = 0.839 \cdot klf$$

Wearing Surface:

$h_{ws} := 4in$ $\qquad$ $w_{ws} := 140pcf$

$DL_{ws} := h_{ws} \cdot W_{bridge} \cdot w_{ws} = 0.661 \cdot klf$

$Weight_{ws} := h_{ws} \cdot W_{bridge} \cdot L_{bridge} \cdot w_{ws} = 76.028 \cdot kip$

$$DL_{ws\_fb} := \frac{\left(\dfrac{Weight_{ws}}{No_{floorbeams}}\right)}{W_{bridge}} = 0.298 \cdot klf$$

$Weight_{fence} := 4.1plf$

$DL_{fence} := 2 \cdot Weight_{fence} = 8.2 \cdot plf$

$$DL_{fence\_fb} := \frac{\left(2 \cdot \dfrac{Weight_{fence} \cdot L_{bridge}}{No_{floorbeams}}\right)}{W_{bridge}} = 3.698 \cdot plf$$

Repair Load

Live Load:

Pedestrian Load:

$$Area_{bridge} := L_{bridge} \cdot W_{bridge} = 1.629 \times 10^3 \cdot ft^2$$

$$PL := 0.090ksf$$ AASHTO Pedestrian 3.1

$$Weight_{pedestrian} := Area_{bridge} \cdot PL = 146.625 \cdot kip$$

$$LL_{pedestrian} := \frac{Weight_{pedestrian}}{L_{bridge}} = 1.275 \cdot klf$$

$$M_{pedestrian} := \frac{\left(LL_{pedestrian} \cdot L_{span}{}^2\right)}{8} = 510.267 \cdot kip \cdot ft$$

$$LL_{pedestrian\_fb} := \frac{\left(\frac{Weight_{pedestrian}}{No_{floorbeams}}\right)}{W_{bridge}} = 0.575 \cdot klf$$

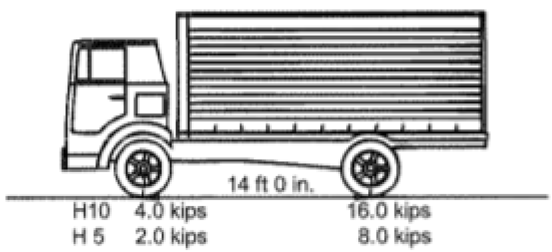
$$M_{pedestrian\_fb} := \frac{\left(LL_{pedestrian\_fb} \cdot W_{bridge}{}^2\right)}{8} = 14.425 \cdot kip \cdot ft$$

Repair Load

Vehicle Load:

AASHTO Pedestrian 3.2

Clear Deck Width	Design Vehicle
7 to 10 ft	H5
Over 10 ft	H10



$P_1 := 4\text{kip}$ $\qquad b := \dfrac{L_{span}}{2} = 28.292 \cdot \text{ft}$ $\qquad$ Distance from end of span to 16 kip wheel

$P_2 := 16\text{kip}$ $\qquad a := L_{span} - b - 14\text{ft} = 14.292 \cdot \text{ft}$ $\qquad$ Distance from end of span to 4 kip wheel

$$R_1 := \frac{\left[P_1 \cdot \left(L_{span} - a\right) + P_2 \cdot b\right]}{L_{span}} = 10.99 \cdot \text{kip}$$

$$R_2 := \frac{\left[P_1 \cdot a + P_2 \cdot \left(L_{span} - b\right)\right]}{L_{span}} = 9.01 \cdot \text{kip}$$

$$M_{vehicle} := \left| \begin{array}{ll} \left(R_1 \cdot a\right) & \text{if } R_1 < P_1 \\ \left(R_2 \cdot b\right) & \text{if } R_2 < P_2 \end{array} \right. = 254.917 \cdot \text{kip} \cdot \text{ft}$$

Repair Load

Vehicle Load on Floorbeams:

$P_{wheel} := 8kip$

If equally spaced on center of bridge:

$$a := \frac{(W_{bridge} - 6ft)}{2} = 4.083 \cdot ft$$

$M_{vehicle\_fb} := P_{wheel} \cdot a = 32.667 \cdot kip \cdot ft$

Equestrian Load:

$EQ := 1kip$ AASHTO Pedestrian 3.3

Assume 1.0 kip over square 4" x 4" area, or concentrated load, on center of span

$$M_{EQ} := \frac{(EQ \cdot L_{span})}{4} = 14.146 \cdot kip \cdot ft$$

For floorbeams:

$$M_{EQ\_fb} := \frac{(EQ \cdot W_{bridge})}{4} = 3.542 \cdot kip \cdot ft$$

Pedestrian Load governs for girders:

$LL := LL_{pedestrian} = 1.275 \cdot klf$

Vehicle Load governs for floorbeams

Repair Load

Wind Load:

Horizontal Wind Load:

$V := 110mph$ AASHTO Signs 3.8.1

Not a special wind region
Not an elevated location

$I_r := 1.15$ AASHTO LRFD Pedestrian 3.4

$K_z := 0.94$ AASHTO Signs Table 3-5

$G := 1.14$ AASHTO Signs 3.8.5

$C_d := 2.0$ AASHTO Signs Table 3.6
For trusses

$$P_z := \left[0.00256 \cdot K_z \cdot G \cdot \left(\frac{V}{mph}\right)^2 \cdot I_r \cdot C_d\right] \cdot psf = 76.346 \cdot psf$$ AASTHO Signs 3.8.1

$H_{horiz} := 72.5in$

$A_{horiz} := H_{horiz} \cdot L_{bridge} = 694.792 \cdot ft^2$

$Weight_{wind\_horiz} := P_z \cdot A_{horiz} = 53.045 \cdot kip$

$$DL_{wind\_horiz} := \frac{2Weight_{wind\_horiz}}{L_{bridge}} = 0.923 \cdot klf$$

Repair Load

Vertical Wind Load:

$P_v := 0.020ksf$ AASHTO Pedestrian 3.4

$A_{deck} := L_{bridge} \cdot W_{bridge} = 1.629 \times 10^3 \cdot ft^2$

$Weight_{vert} := P_v \cdot A_{deck} = 32.583 \cdot kip$

$$WL := \frac{Weight_{vert}}{L_{bridge}} = 0.283 \cdot klf$$

$$WL_{windward} := \frac{1}{4} \cdot WL = 0.071 \cdot klf$$

$$WL_{leeward} := \left(\frac{3}{4}\right) \cdot WL = 0.212 \cdot klf$$

$$WL_{vert} := \max\left(WL_{windward}, WL_{leeward}\right) = 0.212 \cdot klf$$

Fatigue Load:

$P_{NW} := \left(5.2 \cdot C_d \cdot I_r\right) \cdot psf = 11.96 \cdot psf$ AASHTO Signs 11.7.3

$P_{TG} := \left(18.8 \cdot C_d \cdot I_r\right) \cdot psf = 43.24 \cdot psf$ AASTHO Signs 11.7.4

$P_{fatigue} := P_{NW} + P_{TG} = 0.055 \cdot ksf$

$LL_{fatigue} := P_z \cdot \left(2 \cdot H_{horiz}\right) = 0.923 \cdot klf$

Repair Load

Load Combinations:

AASHTO LRFD Table 3.4.1-1

$$DC := DL_{girder} + DL_{cp} + DL_{beam} + DL_{dia} + DL_{slab} + DL_{fence} = 2.826 \cdot klf$$

$$DW := DL_{ws} = 0.661 \cdot klf$$

$$LL = 1.275 \cdot klf$$

$$WS := WL_{vert} = 0.212 \cdot klf$$

$\gamma_{DC} := 1.25$ AASHTO LRFD Table 3.4.1-2

$\gamma_{DW} := 1.50$

$\gamma_{EQ} := 0.5$ AASHTO LRFD 3.4.1

Shear on Bridge:

$$V_{DC} := \frac{(DC \cdot L_{span})}{2} = 79.949 \cdot kip$$

$$V_{DW} := \frac{(DW \cdot L_{span})}{2} = 18.704 \cdot kip$$

$$V_{LL\_pedestrian} := \frac{(LL_{pedestrian} \cdot L_{span})}{2} + \frac{EQ}{2} = 36.572 \cdot kip$$

$$V_{LL\_vehicle} := \max(R_1, R_2) = 10.99 \cdot kip$$

$$V_{LL} := \max(V_{LL\_pedestrian}, V_{LL\_vehicle}) = 36.572 \cdot kip$$

$$V_{WS} := \frac{(WS \cdot L_{span})}{2} = 6.012 \cdot kip$$

$$V_{LL\_fatigue} := \frac{(LL_{fatigue} \cdot L_{span})}{2} = 26.099 \cdot kip$$

Repair Load

$$Strength_{I} := \gamma_{DC} \cdot V_{DC} + \gamma_{DW} \cdot V_{DW} + 1.75 \cdot V_{LL} = 191.993 \cdot kip$$

$$Strength_{III} := \gamma_{DC} \cdot V_{DC} + \gamma_{DW} \cdot V_{DW} + 1.00 \cdot V_{WS} = 134.004 \cdot kip$$

$$Extreme\_Event_{I} := 1.00V_{DC} + 1.00V_{DW} + \gamma_{EQ} \cdot V_{LL} = 116.939 \cdot kip$$

$$Extreme\_Event_{II} := 1.00V_{DC} + 1.00V_{DW} + 0.5V_{LL} = 116.939 \cdot kip$$

$$Service_{I} := 1.00V_{DC} + 1.00V_{DW} + 1.00V_{LL} + 1.00V_{WS} = 141.236 \cdot kip$$

$$Service_{II} := 1.00V_{DC} + 1.00V_{DW} + 1.30V_{LL} = 146.196 \cdot kip$$

$$Service_{IV} := 1.00V_{DC} + 1.00V_{DW} + 1.00V_{WS} = 104.665 \cdot kip$$

$$Fatigue_{I} := 1.75V_{LL\_fatigue} = 45.674 \cdot kip$$

Strength I controls

Repair Load

Moments on Bridge:

$$M_{DC} := \frac{\left(\left(\frac{DC}{2}\cdot L_{span}{}^{2}\right)\right)}{8} = 565.47\cdot kip\cdot ft$$

$$M_{DW} := \frac{\left(\frac{DW}{2}\cdot L_{span}{}^{2}\right)}{8} = 132.291\cdot kip\cdot ft$$

$$M_{LL\_pedestrian} := \frac{M_{pedestrian}}{2} + \frac{M_{EQ}}{2} = 262.206\cdot kip\cdot ft$$

$$M_{LL\_vehicle} := \frac{M_{vehicle}}{2} = 127.458\cdot kip\cdot ft$$

$$M_{LL} := \max\left(M_{LL\_pedestrian}, M_{LL\_vehicle}\right) = 262.206\cdot kip\cdot ft$$

$$M_{WS} := \frac{\left(\frac{WS}{2}\cdot L_{span}{}^{2}\right)}{8} = 42.522\cdot kip\cdot ft$$

$$M_{LL\_fatigue} := \frac{\left(\frac{LL_{fatigue}}{2}\cdot L_{span}{}^{2}\right)}{8} = 184.599\cdot kip\cdot ft$$

Repair Load

$Strength_{I} := \gamma_{DC} \cdot M_{DC} + \gamma_{DW} \cdot M_{DW} + 1.75 \cdot M_{LL} = 1.364 \times 10^{3} \cdot kip \cdot ft$

$Strength_{III} := \gamma_{DC} \cdot M_{DC} + \gamma_{DW} \cdot M_{DW} + 1.00 \cdot M_{WS} = 947.797 \cdot kip \cdot ft$

$Extreme\_Event_{I} := 1.00M_{DC} + 1.00M_{DW} + \gamma_{EQ} \cdot M_{LL} = 828.865 \cdot kip \cdot ft$

$Extreme\_Event_{II} := 1.00M_{DC} + 1.00M_{DW} + 0.5M_{LL} = 828.865 \cdot kip \cdot ft$

$Service_{I} := 1.00M_{DC} + 1.00M_{DW} + 1.00M_{LL} + 1.00M_{WS} = 1.002 \times 10^{3} \cdot kip \cdot ft$

$Service_{II} := 1.00M_{DC} + 1.00M_{DW} + 1.30M_{LL} = 1.039 \times 10^{3} \cdot kip \cdot ft$

$Service_{IV} := 1.00M_{DC} + 1.00M_{DW} + 1.00M_{WS} = 740.284 \cdot kip \cdot ft$

$Fatigue_{I} := 1.75M_{LL\_fatigue} = 323.049 \cdot kip \cdot ft$

Strength I controls

Repair Load

$\varphi_c := 1.00$ — Condition Factor
MBE 6A.4.2.3-1, Good or Satisfactory

$\varphi_s := 0.9$ — System Factor
MBE 6A.4.2.3-1, Riveted Members in Two-Girder

Plastic Moment Capacity of Girders: — AASHTO LRFD Table D6.1-1

$D := 72\text{in}$ $\quad t_w := \frac{1}{2}\text{in}$ — Dimensions of girder

$t_c := \frac{1}{2}\text{in}$ $\quad b_c := 18\text{in}$

$t_t := \frac{1}{2}\text{in}$ $\quad b_t := 18\text{in}$

$F_y := 33\text{ksi}$ — Yield strength of steel, MBE Table 6A.6.2.1-1

$$P_c := F_y \cdot b_c \cdot t_c = 297 \cdot \text{kip}$$

$$P_w := F_y \cdot D \cdot t_w = 1.188 \times 10^3 \cdot \text{kip}$$

$$P_t := F_y \cdot b_t \cdot t_t = 297 \cdot \text{kip}$$

$$Y_{bar} := \left(\frac{D}{2}\right)\left[\frac{\left(P_t - P_c\right)}{P_w} + 1\right] = 36 \cdot \text{in}$$

$d_c := Y_{bar} + \frac{1}{4}\text{in} = 36.25 \cdot \text{in}$ — Distance from center of compression flange to Y.bar

$d_t := Y_{bar} + \frac{1}{4}\text{in} = 36.25 \cdot \text{in}$ — Distance from center of tension flange to Y.bar

$$d_w := 0\text{in}$$

$$M_p := \left(\frac{P_w}{2 \cdot D}\right)\left[Y_{bar}^{\;2} + \left(D - Y_{bar}\right)^2\right] + \left(P_c \cdot d_c + P_t \cdot d_t\right) = 3.576 \times 10^3 \cdot \text{kip} \cdot \text{ft}$$

$$RF_{girder\_pedestrian} := \frac{\left(\varphi_c \cdot \varphi_s \cdot M_p - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}\right)}{1.75 \cdot M_{LL\_pedestrian}} = 5.042$$

$$RF_{girder\_vehicle} := \frac{\left(\varphi_c \cdot \varphi_s \cdot M_p - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}\right)}{1.75 \cdot M_{LL\_vehicle}} = 10.372$$

Repair Load

Plastic Moment Capacity Floorbeam

AASHTO LRFD Table D6.1-1

$t_c := 0.855in$

$t_w := 0.510in$

Dimensions of beam
24 WF 110
AISC 1948

$t_t := 0.855in$

$b_c := 12.042in$

$D := 24.16in - t_c - t_t = 22.45 \cdot in$

$b_t := 12.042in$

$F_y := 33ksi$

Yield strength of steel, MBE Table 6A.6.2.1-1

$P_c := F_y \cdot b_c \cdot t_c = 339.765 \cdot kip$

$P_w := F_y \cdot D \cdot t_w = 377.834 \cdot kip$

$P_t := F_y \cdot b_t \cdot t_t = 339.765 \cdot kip$

$$Y_{bar} := \left(\frac{D}{2}\right)\left[\frac{(P_t - P_c)}{P_w} + 1\right] = 11.225 \cdot in$$

$d_c := Y_{bar} + \frac{1}{4}in = 11.475 \cdot in$

Distance from center of compression flange to Y.bar

$d_t := Y_{bar} + \frac{1}{4}in = 11.475 \cdot in$

Distance from center of tension flange to Y.bar

$d_w := 0in$

$$M_p := \left(\frac{P_w}{2 \cdot D}\right)\left[{Y_{bar}}^2 + (D - Y_{bar})^2\right] + (P_c \cdot d_c + P_t \cdot d_t) = 826.516 \cdot kip \cdot ft$$

Repair Load

$$DC_{fb} := DL_{beam\_fb} + DL_{dia\_fb} + DL_{slab\_fb} + DL_{fence\_fb} = 0.997 \cdot klf$$

$$DW_{fb} := DL_{ws\_fb} = 0.298 \cdot klf$$

$$M_{DC} := \frac{\left(DC_{fb} \cdot W_{bridge}^{\ 2}\right)}{8} = 25.001 \cdot kip \cdot ft$$

$$M_{DW} := \frac{\left(DW_{fb} \cdot W_{bridge}^{\ 2}\right)}{8} = 7.48 \cdot kip \cdot ft$$

$$M_{LL\_pedestrian\_fb} := M_{pedestrian\_fb} + M_{EQ\_fb} = 17.967 \cdot kip \cdot ft$$

$$M_{vehicle\_fb} = 32.667 \cdot kip \cdot ft$$

$$RF_{beam\_pedestrian} := \frac{\left(\varphi_c \cdot \varphi_s \cdot M_p - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}\right)}{1.75 \cdot M_{LL\_pedestrian\_fb}} = 22.308$$

$$RF_{beam\_vehicle} := \frac{\left(\varphi_c \cdot \varphi_s \cdot M_p - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}\right)}{1.75 \cdot M_{vehicle\_fb}} = 12.269$$

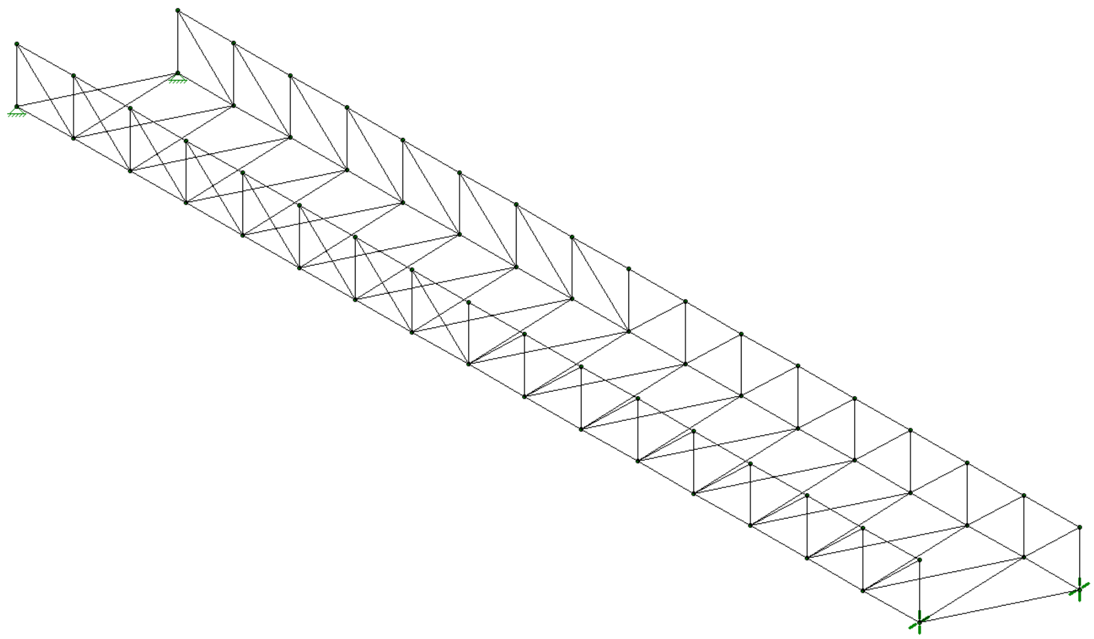
Appendix F: Superstructure Replacement

Superstructure

Superstructure Replacement Design

Refernces:

- AASHTO LRFD Guide Specifications for the Desing of Pedestrian Bridges, 1st Edition, 2009 w/ 2015 interims
- AASHTO Standard Specidications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, Fifth Edition, 2009
- AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2017
- AISC Steel Construction Manual, 15th Edition



$L_{span} := 116ft$ Length of superstructure

$W_{bridge} := 14ft$ Width of superstructure

$NO_{panels} := 16$ Number of truss panels

$F_y := 50ksi$ Strength of steel

$E := 29000ksi$ Modulus of Elasticity of steel

Superstructure

Beam Dimensions and Properties:

Top and bottom chords: HSS 10x10x5/8

$W_{chord} := 76.33plf$ Weight of chord

$Width_{chord} := 10in$ $r_x := 3.8in$

$Height_{chord} := 10$ $r_y := 3.8in$

$Thickness_{chord} := 0.581in$ $t_{w\_chord} := 0.698in$

$Length_{chord} := 7.25ft$ $h_{chord} := t_{w\_chord} \cdot 14.2 = 9.912 \cdot in$

$A_{s\_chord} := 21in^2$

$$\frac{h_{chord}}{t_{w\_chord}} = 14.2$$

$a_{w\_chord} := 2 \cdot h_{chord} \cdot t_{w\_chord} = 13.837 \cdot in^2$ Area for shear check

End posts: HSS 6x6x1/2

$W_{end\_post} := 35.24plf$ Weight of end post

$Width_{end\_post} := 6in$ $t_{w\_epost} := 0.465in$

$Height_{end\_post} := 6in$ $h_{epost} := t_{w\_epost} \cdot 12.1 = 5.627 \cdot in$

$Thickness_{end\_post} := 0.465in$

$$\frac{h_{epost}}{t_{w\_epost}} = 12.1$$

$a_{w\_epost} := 2 \cdot h_{epost} \cdot t_{w\_epost} = 5.233 \cdot in^2$ Area for shear check

$Length_{end\_post} := 7ft$ Height of truss

Superstructure

Vertical posts: HSS 8x6x3/8

$W_{vpost} := 32.58plf$ Weight of vertical posts

$NO_{vpost} := 15$

$I_c := 79.1in^4$

$Width_{vpost} := 6in$

$t_{w\_vpost} := 0.349in$

$Height_{vpost} := 8in$

$h_{vpost} := t_{w\_vpost} \cdot 19.9 = 6.945 \cdot in$

$Thickness_{vpost} := 0.349in$

$$\frac{h_{vpost}}{t_{w\_vpost}} = 19.9$$

$a_{w\_vpost} := 2 \cdot h_{vpost} \cdot t_{w\_vpost} = 4.848 \cdot in^2$ Area for shear check

$Length_{vpost} := 7ft$ Height of truss

Diagonal posts: HSS 4x4x1/4

$W_{dpost} := 25.03plf$ Weight of diagonal posts

$NO_{dpost} := NO_{panels} = 16$

$t_{w\_dpost} := 0.233in$

$Width_{dpost} := 4in$

$h_{dpost} := t_{w\_dpost} \cdot 14.2 = 3.309 \cdot in$

$Height_{dpost} := 4in$

$Thickness_{dpost} := 0.233in$

$$\frac{h_{dpost}}{t_{w\_dpost}} = 14.2$$

$a_{w\_dpost} := 2 \cdot h_{dpost} \cdot t_{w\_dpost} = 1.542 \cdot in^2$

$Length_{dpost} := 10.08ft$

Superstructure

Floorbeams: W 10x22

$W_{floorbeam} := 22plf$ Weight of floorbeam

$Spacing_{floorbeams} := 7.25ft$ $d_{fb} := 10.2in$ $I_b := 118in^4$

$Fbeam_{depth} := 10.2in$ $t_{w\_fb} := 0.240in$

$NO_{fbeams} := 17$ $h_{fb} := t_{w\_fb} \cdot 36.9 = 8.856 \cdot in$

$$\frac{h_{fb}}{t_{w\_fb}} = 36.9$$

$a_{w\_fb} := d_{fb} \cdot t_{w\_fb} = 2.448 \cdot in^2$

$L_{floorbeam} := 15.476ft$

Floor Diagonals: W 8x31

$W_{fdiagonals} := 31plf$ Weight of diagonal floorbeams

$NO_{fdiagonals} := 16$ $t_{w\_dfb} := 0.285in$

$d_f := 8in$ $h_{dfb} := t_{w\_dfb} \cdot 22.3 = 6.355 \cdot in$

$$\frac{h_{dfb}}{t_{w\_dfb}} = 22.3$$

$a_{w\_dfb} := 2 \cdot h_{dfb} \cdot t_{w\_dfb} = 3.623 \cdot in^2$

$L_{fdiagonals} := 14.051ft$

Superstructure

Loads:

Dead Loads:

$DL_{chord} := 2 \cdot W_{chord} \cdot L_{span} = 17.709 \cdot kip$

$DL_{end\_post} := 2 \cdot W_{end\_post} \cdot Length_{end\_post} = 0.493 \cdot kip$

$DL_{vpost} := NO_{vpost} \cdot W_{vpost} \cdot Length_{vpost} = 3.421 \cdot kip$

$DL_{dpost} := NO_{dpost} \cdot W_{dpost} \cdot Length_{dpost} = 4.037 \cdot kip$

$Weight_{truss} := DL_{chord} + DL_{end\_post} + DL_{vpost} + DL_{dpost} = 25.66 \cdot kip$

$DL_{truss} := 1.06 \cdot \left(\frac{Weight_{truss}}{L_{span}} \right) = 234.476 \cdot plf$ Dead load of one truss
Multiplied by 1.06 to account for misc steel

$h_{curb} := 6in$ $w_{curb} := 6in$

$concrete_{depth} := 2in$

$weight_{concrete} := 115pcf$ Lightweight concrete

$W_{concrete\_deck} := weight_{concrete} \cdot concrete_{depth} = 19.167 \cdot psf$

$DL_{concrete} := W_{concrete\_deck} \cdot \left(\frac{W_{bridge}}{2} \right) = 134.167 \cdot plf$ Dead load of concrete on one truss

$DL_{curb} := h_{curb} \cdot w_{curb} \cdot weight_{concrete} = 28.75 \cdot plf$

$DL_{floorbeam} := NO_{fbeams} \cdot W_{floorbeam} \cdot L_{floorbeam} = 5.788 \cdot kip$

$DL_{fdiagonals} := NO_{fdiagonals} \cdot W_{fdiagonals} \cdot L_{fdiagonals} = 6.969 \cdot kip$

$W_{fbeams} := \frac{\left(DL_{floorbeam} + DL_{fdiagonals} \right)}{2} = 6.379 \cdot kip$ Dead load of floorbeams on one truss

$DL_{fbeams} := \frac{W_{fbeams}}{L_{span}} = 54.988 \cdot plf$

Superstructure

$$CLtoCL_{trusses} := W_{bridge} + Height_{end\_post} = 14.5 \cdot ft$$

$$CLtoCL_{chords} := Length_{vpost} + Width_{chord} = 7.833 \cdot ft$$

$$DL_{metal\_decking} := 10.5psf \cdot \left(\frac{W_{bridge}}{2}\right) = 73.5 \cdot plf$$

Dead load of decking on one truss

Pedestrian loading per truss:

$$DL_{Total} := DL_{truss} + DL_{concrete} + DL_{curb} + DL_{fbeams} + DL_{metal\_decking} = 0.526 \cdot klf$$

$$LL_{Pedestrian} := 90psf \cdot \left(\frac{W_{bridge}}{2}\right) = 630 \cdot plf$$

$$M_{Pedestrian} := \frac{\left(LL_{Pedestrian} \cdot L_{span}{}^{2}\right)}{8} = 1.06 \times 10^{3} \cdot kip \cdot ft$$

$$W_{pedestrian} := 90psf \cdot W_{bridge} \cdot L_{span} = 146.16 \cdot kip$$

$$LL_{pedestrian\_fb} := \frac{\left(\frac{W_{pedestrian}}{NO_{fbeams}}\right)}{W_{bridge}} = 0.614 \cdot klf$$

$$M_{pedestrian\_fb} := \frac{\left(LL_{pedestrian\_fb} \cdot W_{bridge}{}^{2}\right)}{8} = 15.046 \cdot kip \cdot ft$$

Moment requirement of each floorbeam

Superstructure

Vehicle Load: H10 Vehicle

$P_1 := 4kip$ $\qquad b := \frac{L_{span}}{2} = 58 \cdot ft$

$P_2 := 16kip$ $\qquad a := L_{span} - b - 14ft = 44 \cdot ft$

$$R_1 := \frac{\left[P_1 \cdot \left(L_{span} - a\right) + P_2 \cdot b\right]}{L_{span}} = 10.483 \cdot kip$$

$$R_2 := \frac{\left[P_1 \cdot a + P_2 \cdot \left(L_{span} - b\right)\right]}{L_{span}} = 9.517 \cdot kip$$

$$M_{vehicle} := \left| \begin{array}{l} \left(\left(R_1 \cdot a\right)\right) \text{ if } R_1 < P_1 \\ \left(\left(R_2 \cdot b\right)\right) \text{ if } R_2 < P_2 \end{array} \right. = 552 \cdot kip \cdot ft$$

$$M_{Vehicle\_truss} := \frac{M_{vehicle}}{2} = 276 \cdot kip \cdot ft$$

Moment of vehicle on one truss

Vehicle Load on Floor Beams:

$P_{wheel} := 8kip$

If equally spaced on center of bridge:

$$a := \frac{\left(L_{floorbeam} - 6ft\right)}{2} = 4.738 \cdot ft$$

$$M_{vehicle\_fb} := P_{wheel} \cdot a = 37.904 \cdot kip \cdot ft$$

Equestrian Load:

$EQ := 1kip$ $\qquad$ AASHTO Pedestrian 3.3

Assume 1.0 kip over square 4" x 4" area, or concentrated load, on center of span

$$M_{EQ} := \frac{\left(EQ \cdot L_{span}\right)}{4} = 29 \cdot kip \cdot ft$$

For floorbeams:

$$M_{EQ\_fb} := \frac{\left(EQ \cdot W_{bridge}\right)}{4} = 3.5 \cdot kip \cdot ft$$

Superstructure

$$Controlling\_LL := \begin{cases} \text{"Pedestrian"} & \text{if } \left(M_{Pedestrian} + M_{EQ}\right) > M_{Vehicle\_truss} \\ \text{"Vehicle"} & \text{otherwise} \end{cases} = \text{"Pedestrian"}$$

$$Controlling\_LL\_fb := \begin{cases} \text{"Pedestrian"} & \text{if } \left(M_{pedestrian\_fb} + M_{EQ\_fb}\right) > M_{vehicle\_fb} \\ \text{"Vehicle"} & \text{otherwise} \end{cases} = \text{"Vehicle"}$$

Pedestrian Load governs for trusses:

$$LL := LL_{Pedestrian} = 0.63 \cdot klf$$

Vehicle Load governs for floorbeams

Wind Load:

Horizontal Wind Load:

$V := 110mph$ AASHTO Signs 3.8.1

Not a special wind region
Not an elevated location

$I_r := 1.15$ AASHTO LRFD Pedestrian 3.4

$K_z := 1.0$ (conservative) AASHTO Signs Table 3-5

$G := 1.14$ AASHTO Signs 3.8.5

$C_d := 2.0$ AASHTO Signs Table 3.6
For trusses

$$P_z := \left[0.00256 \cdot K_z \cdot G \cdot \left(\frac{V}{mph}\right)^2 \cdot I_r \cdot C_d\right] \cdot psf = 81.219 \cdot psf$$ AASTHO Signs 3.8.1

Superstructure

Projected Vertical area per linear foot:

$$A_{deck\_and\_stringers} := \left(Fbeam_{depth} + concrete_{depth}\right)\cdot\left(\frac{Length_{chord}}{Length_{chord}}\right) = 1.017\cdot\frac{ft^2}{ft}$$

$$A_{chords} := 2\cdot\left(Width_{chord}\right)\cdot\left(\frac{Length_{chord}}{Length_{chord}}\right) = 1.667\cdot\frac{ft^2}{ft}$$

$$A_{verticals} := \left(Width_{vpost}\right)\cdot\left(\frac{Length_{vpost}}{Length_{chord}}\right) = 0.483\cdot\frac{ft^2}{ft}$$

$$A_{diagonals} := \left(Width_{dpost}\right)\cdot\left(\frac{Length_{dpost}}{Length_{chord}}\right) = 0.463\cdot\frac{ft^2}{ft}$$

$$Total_{per\_truss} := A_{chords} + A_{verticals} + A_{diagonals} = 2.613\cdot\frac{ft^2}{ft}$$

$$W_{SH} := \left[\left(2\cdot Total_{per\_truss}\right) + A_{deck\_and\_stringers}\right]\cdot P_z = 0.507\cdot klf$$

Vertical Wind Load:

$$P_v := 0.020ksf$$

AASHTO Pedestrian 3.4

$$A_{deck} := L_{span}\cdot W_{bridge} = 1.624\times 10^3\cdot ft^2$$

$$Wind\_Pressure_{vert} := P_v\cdot A_{deck} = 32.48\cdot kip$$

$$WL := \frac{Wind\_Pressure_{vert}}{L_{span}} = 280\cdot plf$$

$$WL_{leeward} := WL\cdot\left[\frac{\left[\left(0.75\cdot W_{bridge}\right) + \frac{CLtoCL_{trusses} - W_{bridge}}{2}\right]}{CLtoCL_{trusses}}\right] = 207.586\cdot plf$$

$$WL_{windward} := WL\cdot\left[\frac{\left[\left(0.25\cdot W_{bridge}\right) + \frac{CLtoCL_{trusses} - W_{bridge}}{2}\right]}{CLtoCL_{trusses}}\right] = 72.414\cdot plf$$

$$WL_{vert} := \max\left(WL_{windward}, WL_{leeward}\right) = 207.586\cdot plf$$

Superstructure

Fatigue:

$P_{NW} := 5.2 C_d \cdot I_r \cdot psf = 11.96 \cdot psf$ AASHTO Signs 11.7.3

$P_{TG} := \left(18.8 C_d \cdot I_r\right) \cdot psf = 43.24 \cdot psf$ AASHTO Signs 11.7.4

$P_{fatigue} := P_{NW} + P_{TG} = 55.2 \cdot psf$

$WS_{Hfat} := \left[\left(2 \cdot Total_{per\_truss}\right) + A_{deck\_and\_stringers}\right] \cdot P_{fatigue} = 344.581 \cdot plf$

Maximum Member Force From Wind:

M10, Bottom Chord = 39.896 kips (from RISA 3D analysis)

$$\Delta f := \frac{(39.896 kip)}{A_{s\_chord}} = 1.9 \cdot ksi$$

$\Delta F_{\cdot n}$ is equal to $\Delta F_{\cdot TH}$ for infinite life

$\Delta F_n := 16 ksi$ AASHTO Signs, Table 11.9.3.1-1

$\gamma := 1.00$ AASHTO LRFD Pedestrian 3.7

$$fatigue_{check} := \begin{vmatrix} \text{"OK"} & \text{if } \gamma \cdot \Delta f < \Delta F_n \\ \text{"Does Not Pass"} & \text{otherwise} \end{vmatrix} = \text{"OK"}$$ AASHTO LRFD, Eq. 6.6.1.2.2-1

Superstructure

Load Combinations:

AASHTO LRFD Table 3.4.1-1

$DC := DL_{Total} = 0.526 \cdot klf$

$LL = 0.63 \cdot klf$

$WS := WL_{vert} = 0.208 \cdot klf$

$LL_{fatigue} := W_{SH} = 0.507 \cdot klf$

$\gamma_{DC} := 1.25$ AASHTO LRFD Table 3.4.1-2

$\gamma_{EQ} := 0.5$ AASHTO LRFD Section 3.4.1

Shear on Bridge:

$$V_{DC} := \frac{(DC \cdot L_{span})}{2} = 30.501 \cdot kip$$

$$V_{LL\_pedestrian} := \frac{(LL_{Pedestrian} \cdot L_{span})}{2} + \frac{EQ}{2} = 37.04 \cdot kip$$

$$V_{LL\_vehicle} := \max(R_1, R_2) = 10.483 \cdot kip$$

$$V_{LL} := \max(V_{LL\_pedestrian}, V_{LL\_vehicle}) = 37.04 \cdot kip$$

$$V_{WS} := \frac{(WS \cdot L_{span})}{2} = 12.04 \cdot kip$$

$$V_{LL\_fatigue} := \frac{(LL_{fatigue} \cdot L_{span})}{2} = 29.406 \cdot kip$$

Superstructure

Shear Load Combinations:

$$Strength_{I} := \gamma_{DC} \cdot V_{DC} + 1.75 \cdot V_{LL} = 102.946 \cdot kip$$

$$Strength_{III} := \gamma_{DC} \cdot V_{DC} + 1.00 \cdot V_{WS} = 50.166 \cdot kip$$

$$Extreme\_Event_{I} := 1.00 V_{DC} + \gamma_{EQ} \cdot V_{LL} = 49.021 \cdot kip$$

$$Extreme\_Event_{II} := 1.00 V_{DC} + 0.5 V_{LL} = 49.021 \cdot kip$$

$$Service_{I} := 1.00 V_{DC} + 1.00 V_{LL} + 1.00 V_{WS} = 79.581 \cdot kip$$

$$Service_{II} := 1.00 V_{DC} + 1.30 V_{LL} = 78.653 \cdot kip$$

$$Service_{IV} := 1.00 V_{DC} + 1.00 V_{WS} = 42.541 \cdot kip$$

$$Fatigue_{I} := 1.75 V_{LL\_fatigue} = 51.461 \cdot kip$$

Moments on Bridge:

$$M_{DC} := \frac{\left(\left(DC \cdot L_{span}{}^{2}\right)\right)}{8} = 884.532 \cdot kip \cdot ft$$

$$M_{LL\_pedestrian} := M_{Pedestrian} + M_{EQ} = 1.089 \times 10^{3} \cdot kip \cdot ft$$

$$M_{LL\_vehicle} := M_{vehicle} = 552 \cdot kip \cdot ft$$

$$M_{LL} := \max\left(M_{LL\_pedestrian}, M_{LL\_vehicle}\right) = 1.089 \times 10^{3} \cdot kip \cdot ft$$

$$M_{WS} := \frac{\left(\frac{WS}{2} \cdot L_{span}{}^{2}\right)}{8} = 174.58 \cdot kip \cdot ft$$

$$M_{LL\_fatigue} := \frac{\left(\frac{LL_{fatigue}}{2} \cdot L_{span}{}^{2}\right)}{8} = 426.39 \cdot kip \cdot ft$$

Superstructure

Moment Load Combinations:

$Strength_I := \gamma_{DC} \cdot M_{DC} + 1.75 \cdot M_{LL} = 3.011 \times 10^3 \cdot kip \cdot ft$

$Strength_{III} := \gamma_{DC} \cdot M_{DC} + 1.00 \cdot M_{WS} = 1.28 \times 10^3 \cdot kip \cdot ft$

$Extreme\_Event_I := 1.00 M_{DC} + \gamma_{EQ} \cdot M_{LL} = 1.429 \times 10^3 \cdot kip \cdot ft$

$Extreme\_Event_{II} := 1.00 M_{DC} + 0.5 M_{LL} = 1.429 \times 10^3 \cdot kip \cdot ft$

$Service_I := 1.00 M_{DC} + 1.00 M_{LL} + 1.00 M_{WS} = 2.148 \times 10^3 \cdot kip \cdot ft$

$Service_{II} := 1.00 M_{DC} + 1.30 M_{LL} = 2.3 \times 10^3 \cdot kip \cdot ft$

$Service_{IV} := 1.00 M_{DC} + 1.00 M_{WS} = 1.059 \times 10^3 \cdot kip \cdot ft$

$Fatigue_I := 1.75 M_{LL\_fatigue} = 746.182 \cdot kip \cdot ft$

Strength I controls

Service III is not applicable

Superstructure

Truss Member Design Loads:

From RISA 3D analysis of governing load combination

$Chord := 406.4kip$ (compression)

$End\_post := 88.746kip$ (compression)

$Diagonal := 119.31kip$ (tension)

$Vertical := 70.472kip$ (compression)

Truss Top Chord Lateral Support

$b := CLtoCL_{trusses} = 174\cdot in$

$h := CLtoCL_{chords} = 94\cdot in$

$$C := \frac{E}{h^2\left[\left(\frac{h}{3I_c}\right)+\left(\frac{b}{2\cdot I_b}\right)\right]} = 2.896\cdot\frac{kip}{in}$$

$L := Spacing_{floorbeams} = 87\cdot in$

$P_c := Chord\cdot 1.33 = 540.512\cdot kip$ AASHTO Pedestrian 7.1.2

$n := 14$

$$\frac{(C\cdot L)}{P_c} = 0.466$$

Use (C*L/Pc) and n to fine 1/K in AASHTO Pedestrian Table 7.1.2-1

$\frac{1}{K} = 0.435$ $K := 2.30$

Superstructure

Top Chord Compressive Resistance:

$$\frac{(K \cdot L)}{r_x} = 52.658$$

KL/r must be less than 120

$$\frac{(K \cdot L)}{r_y} = 52.658$$

Determine Pn (nominal compressive resistance):

$$r_s := \min(r_x, r_y) = 3.8 \cdot in$$

$$\frac{(K \cdot L)}{r_s} = 52.658$$

$$F_y := 50ksi$$

$$\lambda := \left[\left[\frac{(K \cdot L)}{r_s}\right] \cdot \left(\frac{1}{\pi}\right)\right]^2 \cdot \left(\frac{F_y}{E}\right) = 0.484$$

$$P_n := \begin{vmatrix} \left[\frac{(0.88 \cdot F_y \cdot A_{s\_chord})}{\lambda}\right] & \text{if } \lambda > 2.25 \\ \left(0.66^{\lambda} \cdot F_y \cdot A_{s\_chord}\right) & \text{if } \lambda \le 2.25 \end{vmatrix} = 858.573 \cdot kip$$

$$P_r := 0.9 \cdot P_n = 772.716 \cdot kip$$

$$\text{Capacity} := \begin{vmatrix} \text{"OK"} & \text{if } P_r > \text{Chord} \\ \text{"Does Not Pass"} & \text{otherwise} \end{vmatrix} = \text{"OK"}$$

$$\text{limit\_check} := \begin{vmatrix} \text{"OK"} & \text{if } \left(\frac{0.01}{K} \ge 0.0030\right) \\ \text{"Does Not Pass"} & \text{otherwise} \end{vmatrix} = \text{"OK"}$$

Superstructure

Truss Vertical Posts Resistance to Lateral Force:

$H_f := \left(\frac{0.01}{K}\right) \cdot Chord = 1.767 \cdot kip$

AASHTO Pedestrian 7.1.1

$Lateral\_moment_{vpost} := H_f \cdot Length_{vpost} = 12.369 \cdot kip \cdot ft$

HSS 8x6x6 Available Flexural Strength

$Lateral\_capacity := 73.1 kip \cdot ft$

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$$Lateral\_capacity\_check := \begin{vmatrix} \text{"OK"} \quad \text{if } Lateral\_moment_{vpost} < Lateral\_capacity \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{vmatrix} = \text{"OK"}$$

End Posts

$C_{end\_post} := 0.01 \cdot End\_post = 0.887 \cdot kip$

Apply Cend_post at the top of the end post and check capacity

$Lateral\_moment_{epost} := C_{end\_post} \cdot Length_{end\_post} = 6.212 \cdot kip \cdot ft$

HSS 6x6x8 Available Flexural Strength: 59.3 kip-ft

$Lateral\_capacity := 59.33 kip \cdot ft$

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$$Lateral\_capacity\_check := \begin{vmatrix} \text{"OK"} \quad \text{if } Lateral\_moment_{epost} < Lateral\_capacity \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{vmatrix} = \text{"OK"}$$

Diagonals:

Loading on Diagonals (Tension):

$Diagonal = 119.31 \cdot kip$

HSS 4x4x4 Available Strength in Axial Tension

$Axial\_capacity\_tension := 152 kip$

AISC Steel Construction Manual

$$Capacity\_check := \begin{vmatrix} \text{"OK"} \quad \text{if } Axial\_capacity\_tension > Diagonal \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{vmatrix} = \text{"OK"}$$

Superstructure

Deflection:

Check LL deflection from RISA 3D truss analysis: Use Unfactored Live Load

$$LL_{deflection\_max} := \left(\frac{1}{360}\right) \cdot L_{span} = 3.867 \cdot in$$

From seperate RISA 3D analysis:

$$\Delta_{LL} := 1.954in$$

$$Deflection\_check := \begin{vmatrix} \text{"OK"} \quad \text{if } LL_{deflection\_max} > \Delta_{LL} \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{vmatrix} = \text{"OK"}$$

Wind Load Deflection:

Apply horizontal wind load to the side of the truss in RISA 3D

$$WL\_max := \frac{L_{span}}{500} = 2.784 \cdot in$$

$$\Delta_{WL} := 0.44in$$

$$WL\_Deflection\_check := \begin{vmatrix} \text{"OK"} \quad \text{if } WL\_max > \Delta_{WL} \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{vmatrix} = \text{"OK"}$$

Superstructure

Vibrations:

Check maximum vertical deflection of the truss due to dead load (ft)

3.0 Hz minimum desirable (f>3.0)

$f_{min} := 3.0\text{Hz}$

$f_{superstructure} := 3.183\text{Hz}$ From RISA Analysis

$$F_{check} := \begin{vmatrix} \text{"OK"} \quad \text{if } f_{superstructure} > f_{min} \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{vmatrix} = \text{"OK"}$$

$f_{horizontal} := 2.596\text{Hz}$ Horizontal Frequency>1.3Hz

Floorbeams:

$$W_{concrete\_deck} \cdot Spacing_{floorbeams} = 0.139 \cdot \frac{kip}{ft}$$

From Separate RISA analysis:

$Moment_{concrete\_fbeam} := 2.27kip \cdot ft$

Moment from vehicle load controls:

$M_{vehicle\_fb} = 37.904 \cdot kip \cdot ft$ Required Capacity

W 10x22 Available Moment Capacity: 60 kip-ft

$Moment\_capacity := 60 \cdot kip \cdot ft$

$$Moment\_capacity\_check := \begin{vmatrix} \text{"OK"} \quad \text{if } Moment\_capacity > M_{vehicle\_fb} \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{vmatrix} = \text{"OK"}$$

Shear Check:

Chord Shear Check

$k_v := 5$

$$C_{v2\_chord} := \begin{cases} 1.0 & \text{if } \dfrac{h_{chord}}{t_{w\_chord}} \le 1.10\cdot\sqrt{k_v\cdot\dfrac{E}{F_y}} \\ 1.10\dfrac{\sqrt{k_v\cdot\dfrac{E}{F_y}}}{\dfrac{h_{chord}}{t_{w\_chord}}} & \text{if } \left(1.10\sqrt{k_v\cdot\dfrac{E}{F_y}} < \dfrac{h_{chord}}{t_{w\_chord}} \le 1.37\sqrt{k_v\cdot\dfrac{E}{F_y}}\right) \\ 1.51\cdot k_v\dfrac{E}{\left(\dfrac{h_{chord}}{t_{w\_chord}}\right)^2\cdot F_y} & \text{if } \dfrac{h_{chord}}{t_{w\_chord}} > 1.37\sqrt{k_v\cdot\dfrac{E}{F_y}} \end{cases} = 1$$

$V_{n\_chord} := 0.6\cdot F_y\cdot a_{w\_chord}\cdot C_{v2\_chord} = 415.098\cdot kip$

From Risa 3D analysis, find max shear on the member from governing load equation

$V_{chord} := 5.99kip$

$$V_{check\_chord} := \begin{cases} \text{"OK"} & \text{if } V_{n\_chord} > V_{chord} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Superstructure

End Post Shear Check

$$C_{v2\_epost} := \begin{cases} 1.0 & \text{if } \dfrac{h_{epost}}{t_{w\_epost}} \le 1.10\cdot\sqrt{k_v\cdot\dfrac{E}{F_y}} \\ 1.10\dfrac{\sqrt{k_v\cdot\dfrac{E}{F_y}}}{\dfrac{h_{epost}}{t_{w\_epost}}} & \text{if } \left(1.10\sqrt{k_v\cdot\dfrac{E}{F_y}} < \dfrac{h_{epost}}{t_{w\_epost}} \le 1.37\sqrt{k_v\cdot\dfrac{E}{F_y}}\right) \\ 1.51\cdot k_v\dfrac{E}{\left(\dfrac{h_{epost}}{t_{w\_epost}}\right)^2\cdot F_y} & \text{if } \dfrac{h_{epost}}{t_{w\_epost}} > 1.37\sqrt{k_v\cdot\dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_epost} := 0.6\cdot F_y\cdot a_{w\_epost}\cdot C_{v2\_epost} = 156.979\cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{epost} := 5.57kip$$

$$V_{check\_epost} := \begin{cases} \text{"OK"} & \text{if } V_{n\_epost} > V_{epost} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Superstructure

Vertical Post Shear Check

$$C_{v2\_vpost} := \begin{cases} 1.0 & \text{if } \dfrac{h_{vpost}}{t_{w\_vpost}} \le 1.10 \cdot \sqrt{k_v \cdot \dfrac{E}{F_y}} \\[2ex] 1.10 \dfrac{\sqrt{k_v \cdot \dfrac{E}{F_y}}}{\dfrac{h_{vpost}}{t_{w\_vpost}}} & \text{if } \left(1.10 \sqrt{k_v \cdot \dfrac{E}{F_y}} < \dfrac{h_{vpost}}{t_{w\_vpost}} \le 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}}\right) \\[2ex] 1.51 \cdot k_v \dfrac{E}{\left(\dfrac{h_{vpost}}{t_{w\_vpost}}\right)^2 \cdot F_y} & \text{if } \dfrac{h_{vpost}}{t_{w\_vpost}} > 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_vpost} := 0.6 \cdot F_y \cdot a_{w\_vpost} \cdot C_{v2\_vpost} = 145.43 \cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{vpost} := 8.2kip$$

$$V_{check\_vpost} := \begin{cases} \text{"OK"} & \text{if } V_{n\_vpost} > V_{vpost} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Superstructure

Diagonal Post Shear Check

$$C_{v2\_dpost} := \begin{cases} 1.0 & \text{if } \dfrac{h_{dpost}}{t_{w\_dpost}} \le 1.10 \cdot \sqrt{k_v \cdot \dfrac{E}{F_y}} \\ 1.10 \dfrac{\sqrt{k_v \cdot \dfrac{E}{F_y}}}{\dfrac{h_{dpost}}{t_{w\_dpost}}} & \text{if } \left(1.10 \sqrt{k_v \cdot \dfrac{E}{F_y}} < \dfrac{h_{dpost}}{t_{w\_dpost}} \le 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}}\right) \\ 1.51 \cdot k_v \dfrac{E}{\left(\dfrac{h_{dpost}}{t_{w\_dpost}}\right)^2 \cdot F_y} & \text{if } \dfrac{h_{dpost}}{t_{w\_dpost}} > 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_dpost} := 0.6 \cdot F_y \cdot a_{w\_dpost} \cdot C_{v2\_dpost} = 46.254 \cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{dpost} := 0.1kip$$

$$V_{check\_dpost} := \begin{cases} \text{"OK"} & \text{if } V_{n\_dpost} > V_{dpost} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Superstructure

Floorbeam Shear Check

$$C_{v2\_fb} := \begin{cases} 1.0 & \text{if } \dfrac{h_{fb}}{t_{w\_fb}} \le 1.10 \cdot \sqrt{k_v \cdot \dfrac{E}{F_y}} \\ 1.10 \dfrac{\sqrt{k_v \cdot \dfrac{E}{F_y}}}{\dfrac{h_{fb}}{t_{w\_fb}}} & \text{if } \left(1.10 \sqrt{k_v \cdot \dfrac{E}{F_y}} < \dfrac{h_{fb}}{t_{w\_fb}} \le 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}}\right) \\ 1.51 \cdot k_v \dfrac{E}{\left(\dfrac{h_{fb}}{t_{w\_fb}}\right)^2 \cdot F_y} & \text{if } \dfrac{h_{fb}}{t_{w\_fb}} > 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_fb} := 0.6 \cdot F_y \cdot a_{w\_fb} \cdot C_{v2\_fb} = 73.44 \cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{fb} := P_{wheel} = 8 \cdot kip$$

$$V_{check\_fb} := \begin{cases} \text{"OK"} & \text{if } V_{n\_fb} > V_{fb} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Diagonal Floorbeam Shear Check

$$C_{v2\_dfb} := \begin{cases} 1.0 & \text{if } \dfrac{h_{dfb}}{t_{w\_dfb}} < 1.10 \cdot \sqrt{k_v \cdot \dfrac{E}{F_y}} \\ 1.10 \dfrac{\sqrt{k_v \cdot \dfrac{E}{F_y}}}{\dfrac{h_{dfb}}{t_{w\_dfb}}} & \text{if } \left(1.10 \sqrt{k_v \cdot \dfrac{E}{F_y}} < \dfrac{h_{dfb}}{t_{w\_dfb}} < 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}}\right) \\ 1.51 \cdot k_v \dfrac{E}{\left(\dfrac{h_{dfb}}{t_{w\_dfb}}\right)^2 \cdot F_y} & \text{if } \dfrac{h_{dfb}}{t_{w\_dfb}} > 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_dfb} := 0.6 \cdot F_y \cdot a_{w\_dfb} \cdot C_{v2\_dfb} = 108.679 \cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{dfb} := 8kip$$

$$V_{check\_dfb} := \begin{cases} \text{"OK"} & \text{if } V_{n\_dfb} > V_{dfb} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

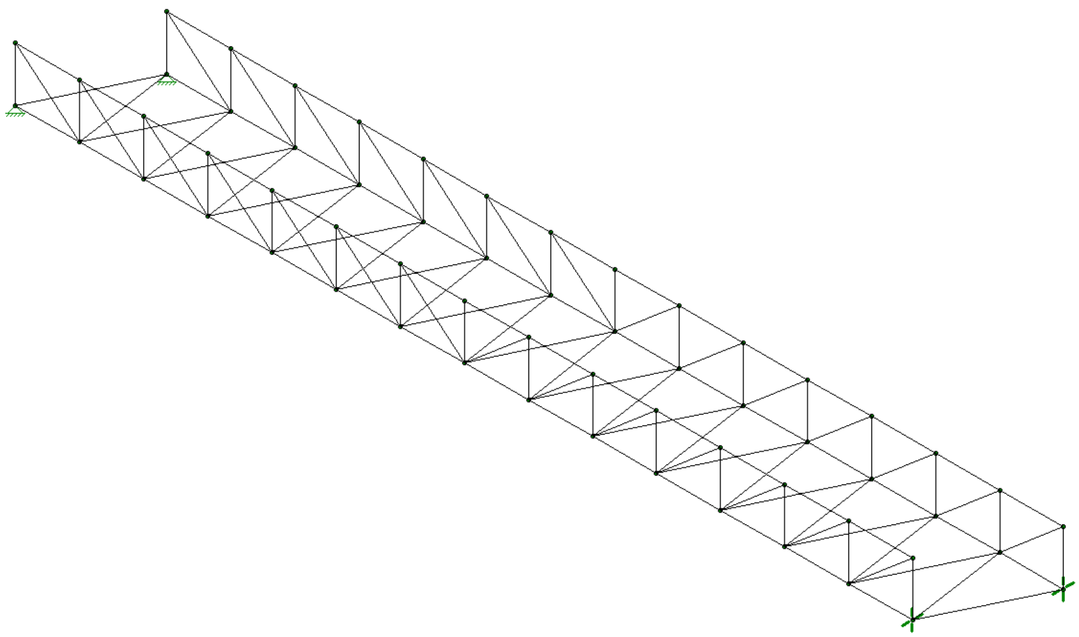
Appendix G: Superstructure Full Replacement

Full Replacement

Full Replacement Superstructure Design

Refernces:

- AASHTO LRFD Guide Specifications for the Desing of Pedestrian Bridges, 1st Edition, 2009 w/ 2015 interims
- AASHTO Standard Specidications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, Fifth Edition, 2009
- AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2017
- AISC Steel Construction Manual, 15th Edition



$L_{span} := 140ft$	Length of superstructure
$W_{bridge} := 16ft$	Width of superstructure
$NO_{panels} := 14$	Number of truss panels
$F_y := 50ksi$	Strength of steel
$E := 29000ksi$	Modulus of Elasticity of steel

Full Replacement

Beam Dimensions and Properties:

Top and bottom chords: HSS 14x14x7/8

$W_{chord} := 149.61 plf$ Weight of chord

$Width_{chord} := 14 in$

$Height_{chord} := 14 in$

$Thickness_{chord} := .814 in$

$Length_{chord} := 10 ft$

$r_x := 5.33 in$

$r_y := 5.33 in$

$t_{w\_chord} := .814 in$

$h_{chord} := t_{w\_chord} \cdot 14.3 = 11.64 \cdot in$

$A_{s\_chord} := 41.2 in^2$

$$\frac{h_{chord}}{t_{w\_chord}} = 14.3$$

$a_{w\_chord} := 2 \cdot h_{chord} \cdot t_{w\_chord} = 18.95 \cdot in^2$ Area for shear check

End posts: HSS 6x6x1/2

$W_{end\_post} := 35.24 plf$ Weight of end post

$Width_{end\_post} := 6 in$

$Height_{end\_post} := 6 in$

$Thickness_{end\_post} := 0.465 in$

$t_{w\_epost} := .465 in$

$h_{epost} := t_{w\_epost} \cdot 9.90 = 4.604 \cdot in$

$$\frac{h_{epost}}{t_{w\_epost}} = 9.9$$

$a_{w\_epost} := 2 \cdot h_{epost} \cdot t_{w\_epost} = 4.281 \cdot in^2$ Area for shear check

$Length_{end\_post} := 8.5 ft$ Height of truss

Full Replacement

Vertical posts: HSS 8x6x3/8

$W_{vpost} := 32.58plf$ Weight of vertical posts

$NO_{vpost} := 13$

$I_c := 79.1in^4$

$Width_{vpost} := 6in$

$t_{w\_vpost} := .349in$

$Height_{vpost} := 8in$

$h_{vpost} := t_{w\_vpost} \cdot 19.9 = 6.945 \cdot in$

$Thickness_{vpost} := 0.349in$

$$\frac{h_{vpost}}{t_{w\_vpost}} = 19.9$$

$a_{w\_vpost} := 2 \cdot h_{vpost} \cdot t_{w\_vpost} = 4.848 \cdot in^2$ Area for shear check

$Length_{vpost} := 8.5ft$ Height of truss

Diagonal posts: HSS 4x4x1/2

$W_{dpost} := 25.03plf$ Weight of diagonal posts

$NO_{dpost} := 14$

$t_{w\_dpost} := .465in$

$Width_{dpost} := 4in$

$h_{dpost} := t_{w\_dpost} \cdot 5.60 = 2.604 \cdot in$

$Height_{dpost} := 4in$

$Thickness_{dpost} := .465in$

$$\frac{h_{dpost}}{t_{w\_dpost}} = 5.6$$

$a_{w\_dpost} := 2 \cdot h_{dpost} \cdot t_{w\_dpost} = 2.422 \cdot in^2$

$Length_{dpost} := 13.12ft$

Full Replacement

Floorbeams: W 12x50

$W_{floorbeam} := 50plf$ Weight of floorbeam

$Spacing_{floorbeams} := 10ft$ $d_{fb} := 12.2in$ $I_b := 391in^4$

$Fbeam_{depth} := 12.2in$ $t_{w\_fb} := .370in$

$NO_{fbeams} := 15$ $h_{fb} := t_{w\_fb} \cdot 26.8 = 9.916 \cdot in$

$$\frac{h_{fb}}{t_{w\_fb}} = 26.8$$

$a_{w\_fb} := d_{fb} \cdot t_{w\_fb} = 4.514 \cdot in^2$

$L_{floorbeam} := 17.687ft$

Floor Diagonals: W 10x45

$W_{fdiagonals} := 45plf$ Weight of diagonal floorbeams

$NO_{fdiagonals} := 14$ $t_{w\_dfb} := .350in$

$d_f := 10.1in$ $h_{dfb} := t_{w\_dfb} \cdot 22.5 = 7.875 \cdot in$

$$\frac{h_{dfb}}{t_{w\_dfb}} = 22.5$$

$a_{w\_dfb} := 2 \cdot h_{dfb} \cdot t_{w\_dfb} = 5.512 \cdot in^2$

$L_{fdiagonals} := 16.188ft$

Full Replacement

Loads:

Dead Loads:

$DL_{chord} := 2 \cdot W_{chord} \cdot L_{span} = 41.891 \cdot kip$

$DL_{end\_post} := 2 \cdot W_{end\_post} \cdot Length_{end\_post} = 0.599 \cdot kip$

$DL_{vpost} := NO_{vpost} \cdot W_{vpost} \cdot Length_{vpost} = 3.6 \cdot kip$

$DL_{dpost} := NO_{dpost} \cdot W_{dpost} \cdot Length_{dpost} = 4.598 \cdot kip$

$Weight_{truss} := DL_{chord} + DL_{end\_post} + DL_{vpost} + DL_{dpost} = 50.687 \cdot kip$

$DL_{truss} := 1.06 \cdot \left(\frac{Weight_{truss}}{L_{span}} \right) = 383.777 \cdot plf$ Dead load of one truss
Multiplied by 1.06 to account for misc steel

$h_{curb} := 6.in$ $w_{curb} := 6in$

$concrete_{depth} := 2in$

$weight_{concrete} := 115pcf$ Lightweight concrete

$W_{concrete\_deck} := weight_{concrete} \cdot concrete_{depth} = 19.167 \cdot psf$

$DL_{concrete} := W_{concrete\_deck} \cdot \left(\frac{W_{bridge}}{2} \right) = 153.333 \cdot plf$ Dead load of concrete on one truss

$DL_{curb} := weight_{concrete} \cdot h_{curb} \cdot w_{curb} = 28.75 \cdot plf$

$DL_{floorbeam} := NO_{fbeams} \cdot W_{floorbeam} \cdot L_{floorbeam} = 13.265 \cdot kip$

$DL_{fdiagonals} := NO_{fdiagonals} \cdot W_{fdiagonals} \cdot L_{fdiagonals} = 10.198 \cdot kip$

$W_{fbeams} := \frac{\left(DL_{floorbeam} + DL_{fdiagonals}\right)}{2} = 11.732 \cdot kip$ Dead load of floorbeams on one truss

$DL_{fbeams} := \frac{W_{fbeams}}{L_{span}} = 83.799 \cdot plf$

Full Replacement

$$CLtoCL_{trusses} := W_{bridge} + Height_{end\_post} = 16.5 \cdot ft$$

$$CLtoCL_{chords} := Length_{vpost} + Width_{chord} = 9.667 \cdot ft$$

$$DL_{metal\_decking} := 10.5psf \cdot \left(\frac{W_{bridge}}{2}\right) = 84 \cdot plf$$

Dead load of decking on one truss

Pedestrian loading per truss:

$$DL_{Total} := DL_{truss} + DL_{concrete} + DL_{curb} + DL_{fbeams} + DL_{metal\_decking} = 0.734 \cdot klf$$

$$LL_{Pedestrian} := 90psf \cdot \left(\frac{W_{bridge}}{2}\right) = 720 \cdot plf$$

$$M_{Pedestrian} := \frac{\left(LL_{Pedestrian} \cdot L_{span}{}^{2}\right)}{8} = 1.764 \times 10^{3} \cdot kip \cdot ft$$

$$W_{pedestrian} := 90psf \cdot W_{bridge} \cdot L_{span} = 201.6 \cdot kip$$

$$LL_{pedestrian\_fb} := \frac{\left(\frac{W_{pedestrian}}{NO_{fbeams}}\right)}{W_{bridge}} = 0.84 \cdot klf$$

$$M_{pedestrian\_fb} := \frac{\left(LL_{pedestrian\_fb} \cdot W_{bridge}{}^{2}\right)}{8} = 26.88 \cdot kip \cdot ft$$

Moment requirement of each floorbeam

Full Replacement

Vehicle Load: H10 Vehicle

$P_1 := 4kip$

$b := \frac{L_{span}}{2} = 70 \cdot ft$

$P_2 := 16kip$

$a := L_{span} - b - 14ft = 56 \cdot ft$

$$R_1 := \frac{\left[P_1 \cdot \left(L_{span} - a\right) + P_2 \cdot b\right]}{L_{span}} = 10.4 \cdot kip$$

$$R_2 := \frac{\left[P_1 \cdot a + P_2 \cdot \left(L_{span} - b\right)\right]}{L_{span}} = 9.6 \cdot kip$$

$$M_{vehicle} := \begin{vmatrix} \left(\left(R_1 \cdot a\right)\right) \text{ if } R_1 < P_1 \\ \left(\left(R_2 \cdot b\right)\right) \text{ if } R_2 < P_2 \end{vmatrix} = 672 \cdot kip \cdot ft$$

$$M_{Vehicle\_truss} := \frac{M_{vehicle}}{2} = 336 \cdot kip \cdot ft$$

Moment of vehicle on one truss

Vehicle Load on Floor Beams:

$P_{wheel} := 8kip$

If equally spaced on center of bridge:

$$a := \frac{\left(L_{floorbeam} - 6ft\right)}{2} = 5.843 \cdot ft$$

$$M_{vehicle\_fb} := P_{wheel} \cdot a = 46.748 \cdot kip \cdot ft$$

Equestrian Load:

$EQ := 1kip$ AASHTO Pedestrian 3.3

Assume 1.0 kip over square 4" x 4" area, or concentrated load, on center of span

$$M_{EQ} := \frac{\left(EQ \cdot L_{span}\right)}{4} = 35 \cdot kip \cdot ft$$

For floorbeams:

$$M_{EQ\_fb} := \frac{\left(EQ \cdot W_{bridge}\right)}{4} = 4 \cdot kip \cdot ft$$

Full Replacement

$$Controlling\_LL := \begin{cases} \text{"Pedestrian"} & \text{if } (M_{Pedestrian} + M_{EQ}) > M_{Vehicle\_truss} \\ \text{"Vehicle"} & \text{otherwise} \end{cases} = \text{"Pedestrian"}$$

$$Controlling\_LL\_fb := \begin{cases} \text{"Pedestrian"} & \text{if } (M_{pedestrian\_fb} + M_{EQ\_fb}) > M_{vehicle\_fb} \\ \text{"Vehicle"} & \text{otherwise} \end{cases} = \text{"Vehicle"}$$

Pedestrian Load governs for trusses:

$$LL := LL_{Pedestrian} = 0.72 \cdot klf$$

Vehicle Load governs for floorbeams

Wind Load:

Horizontal Wind Load:

Table numbers may differ slightly as some reference the hard-copy edition we used vs. the online version

$V := 110mph$ AASHTO Signs 3.8.1

Not a special wind region
Not an elevated location

$I_r := 1.15$ AASHTO LRFD Pedestrian 3.4

$K_z := 1.0$ (conservative) AASHTO Signs Table 3-5

$G := 1.14$ AASHTO Signs 3.8.5

$C_d := 2.0$ AASHTO Signs Table 3.6
For trusses

$$P_z := \left[0.00256 \cdot K_z \cdot G \cdot \left(\frac{V}{mph}\right)^2 \cdot I_r \cdot C_d\right] \cdot psf = 81.219 \cdot psf$$ AASTHO Signs 3.8.1

Full Replacement

Projected Vertical area per linear foot:

$$A_{deck\_and\_stringers} := \left(Fbeam_{depth} + concrete_{depth}\right)\cdot\left(\frac{Length_{chord}}{Length_{chord}}\right) = 1.183\cdot\frac{ft^2}{ft}$$

$$A_{chords} := 2\cdot\left(Width_{chord}\right)\cdot\left(\frac{Length_{chord}}{Length_{chord}}\right) = 2.333\cdot\frac{ft^2}{ft}$$

$$A_{verticals} := \left(Width_{vpost}\right)\cdot\left(\frac{Length_{vpost}}{Length_{chord}}\right) = 0.425\cdot\frac{ft^2}{ft}$$

$$A_{diagonals} := \left(Width_{dpost}\right)\cdot\left(\frac{Length_{dpost}}{Length_{chord}}\right) = 0.437\cdot\frac{ft^2}{ft}$$

$$Total_{per\_truss} := A_{chords} + A_{verticals} + A_{diagonals} = 3.196\cdot\frac{ft^2}{ft}$$

$$W_{SH} := \left[\left(2\cdot Total_{per\_truss}\right) + A_{deck\_and\_stringers}\right]\cdot P_z = 0.615\cdot klf$$

Vertical Wind Load:

$$P_v := 0.020ksf$$

AASHTO Pedestrian 3.4

$$A_{deck} := L_{span}\cdot W_{bridge} = 2.24\times 10^3\cdot ft^2$$

$$Wind\_Pressure_{vert} := P_v\cdot A_{deck} = 44.8\cdot kip$$

$$WL := \frac{Wind\_Pressure_{vert}}{L_{span}} = 320\cdot plf$$

$$WL_{leeward} := WL\cdot\left[\frac{\left[\left(0.75\cdot W_{bridge}\right) + \frac{CLtoCL_{trusses} - W_{bridge}}{2}\right]}{CLtoCL_{trusses}}\right] = 237.576\cdot plf$$

$$WL_{windward} := WL\cdot\left[\frac{\left[\left(0.25\cdot W_{bridge}\right) + \frac{CLtoCL_{trusses} - W_{bridge}}{2}\right]}{CLtoCL_{trusses}}\right] = 82.424\cdot plf$$

$$WL_{vert} := \max\left(WL_{windward}, WL_{leeward}\right) = 237.576\cdot plf$$

Full Replacement

Fatigue:

$P_{NW} := 5.2C_d \cdot I_r \cdot psf = 11.96 \cdot psf$ AASHTO Signs 11.7.3

$P_{TG} := \left(18.8C_d \cdot I_r\right) \cdot psf = 43.24 \cdot psf$ AASHTO Signs 11.7.4

$P_{fatigue} := P_{NW} + P_{TG} = 55.2 \cdot psf$

$WS_{Hfat} := \left[\left(2 \cdot Total_{per\_truss}\right) + A_{deck\_and\_stringers}\right] \cdot P_{fatigue} = 418.122 \cdot plf$

Maximum Member Force From Wind:

M10, Bottom Chord = 39.896 kips (from RISA 3D analysis)

$\Delta f := \frac{(39.896kip)}{A_{s\_chord}} = 0.968 \cdot ksi$

$\Delta F_{\cdot n}$ is equal to $\Delta F_{\cdot TH}$ for infinite life

$\Delta F_n := 16ksi$ AASHTO Signs, Table 11.9.3.1-1

$\gamma := 1.00$ AASHTO LRFD Pedestrian 3.7

$fatigue_{check} := \begin{cases} \text{"OK"} & \text{if } \gamma \cdot \Delta f < \Delta F_n \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$ AASHTO LRFD, Eq. 6.6.1.2.2-1

Full Replacement

Load Combinations:

AASHTO LRFD Table 3.4.1-1

$DC := DL_{Total} = 0.734 \cdot klf$

$LL = 0.72 \cdot klf$

$WS := WL_{vert} = 0.238 \cdot klf$

$LL_{fatigue} := W_{SH} = 0.615 \cdot klf$

$\gamma_{DC} := 1.25$ AASHTO LRFD Table 3.4.1-2

$\gamma_{EQ} := 0.5$ AASHTO LRFD Section 3.4.1

Shear on Bridge:

$$V_{DC} := \frac{\left(DC \cdot L_{span}\right)}{2} = 51.356 \cdot kip$$

$$V_{LL\_pedestrian} := \frac{\left(LL_{Pedestrian} \cdot L_{span}\right)}{2} + \frac{EQ}{2} = 50.9 \cdot kip$$

$$V_{LL\_vehicle} := \max\left(R_1, R_2\right) = 10.4 \cdot kip$$

$$V_{LL} := \max\left(V_{LL\_pedestrian}, V_{LL\_vehicle}\right) = 50.9 \cdot kip$$

$$V_{WS} := \frac{\left(WS \cdot L_{span}\right)}{2} = 16.63 \cdot kip$$

$$V_{LL\_fatigue} := \frac{\left(LL_{fatigue} \cdot L_{span}\right)}{2} = 43.065 \cdot kip$$

Full Replacement

Shear Load Combinations:

$$Strength_{I} := \gamma_{DC} \cdot V_{DC} + 1.75 \cdot V_{LL} = 153.27 \cdot kip$$

$$Strength_{III} := \gamma_{DC} \cdot V_{DC} + 1.00 \cdot V_{WS} = 80.825 \cdot kip$$

$$Extreme\_Event_{I} := 1.00V_{DC} + \gamma_{EQ} \cdot V_{LL} = 76.806 \cdot kip$$

$$Extreme\_Event_{II} := 1.00V_{DC} + 0.5V_{LL} = 76.806 \cdot kip$$

$$Service_{I} := 1.00V_{DC} + 1.00V_{LL} + 1.00V_{WS} = 118.886 \cdot kip$$

$$Service_{II} := 1.00V_{DC} + 1.30V_{LL} = 117.526 \cdot kip$$

$$Service_{IV} := 1.00V_{DC} + 1.00V_{WS} = 67.986 \cdot kip$$

$$Fatigue_{I} := 1.75V_{LL\_fatigue} = 75.363 \cdot kip$$

Moments on Bridge:

$$M_{DC} := \frac{\left(\left(DC \cdot L_{span}^{\ 2}\right)\right)}{8} = 1.797 \times 10^{3} \cdot kip \cdot ft$$

$$M_{LL\_pedestrian} := M_{Pedestrian} + M_{EQ} = 1.799 \times 10^{3} \cdot kip \cdot ft$$

$$M_{LL\_vehicle} := M_{vehicle} = 672 \cdot kip \cdot ft$$

$$M_{LL} := \max\left(M_{LL\_pedestrian}, M_{LL\_vehicle}\right) = 1.799 \times 10^{3} \cdot kip \cdot ft$$

$$M_{WS} := \frac{\left(\frac{WS}{2} \cdot L_{span}^{\ 2}\right)}{8} = 291.03 \cdot kip \cdot ft$$

$$M_{LL\_fatigue} := \frac{\left(\frac{LL_{fatigue}}{2} \cdot L_{span}^{\ 2}\right)}{8} = 753.629 \cdot kip \cdot ft$$

Full Replacement

Moment Load Combinations:

$Strength_{I} := \gamma_{DC} \cdot M_{DC} + 1.75 \cdot M_{LL} = 5.395 \times 10^{3} \cdot kip \cdot ft$

$Strength_{III} := \gamma_{DC} \cdot M_{DC} + 1.00 \cdot M_{WS} = 2.538 \times 10^{3} \cdot kip \cdot ft$

$Extreme\_Event_{I} := 1.00M_{DC} + \gamma_{EQ} \cdot M_{LL} = 2.697 \times 10^{3} \cdot kip \cdot ft$

$Extreme\_Event_{II} := 1.00M_{DC} + 0.5M_{LL} = 2.697 \times 10^{3} \cdot kip \cdot ft$

$Service_{I} := 1.00M_{DC} + 1.00M_{LL} + 1.00M_{WS} = 3.887 \times 10^{3} \cdot kip \cdot ft$

$Service_{II} := 1.00M_{DC} + 1.30M_{LL} = 4.136 \times 10^{3} \cdot kip \cdot ft$

$Service_{IV} := 1.00M_{DC} + 1.00M_{WS} = 2.088 \times 10^{3} \cdot kip \cdot ft$

$Fatigue_{I} := 1.75M_{LL\_fatigue} = 1.319 \times 10^{3} \cdot kip \cdot ft$

Strength I controls

Service III is not applicable

Full Replacement

Truss Member Design Loads:

From RISA 3D analysis of governing load combination

$Chord := 751.7kip$ (compression)

$End\_post := 169.8kip$ (compression)

$Diagonal := 250.3kip$ (tension)

$Vertical := 137.2kip$ (compression)

Truss Top Chord Lateral Support

$b := CLtoCL_{trusses} = 198 \cdot in$

$h := CLtoCL_{chords} = 116 \cdot in$

$$C := \frac{E}{h^2 \left[\left(\frac{h}{3I_c} \right) + \left(\frac{b}{2 \cdot I_b} \right) \right]} = 2.904 \cdot \frac{kip}{in}$$

$L := Spacing_{floorbeams} = 120 \cdot in$

$P_c := Chord \cdot 1.33 = 999.761 \cdot kip$ AASHTO Pedestrian 7.1.2

$n := 16$

$$\frac{(C \cdot L)}{P_c} = 0.349$$

$\frac{1}{K} = 0.368$ $K := 2.72$

Full Replacement

Top Chord Compressive Resistance:

$\frac{(K \cdot L)}{r_x} = 61.238$ KL/r must be less than 120

$\frac{(K \cdot L)}{r_y} = 61.238$

Determine Pn (nominal compressive resistance):

$r_s := \min(r_x, r_y) = 5.33 \cdot in$

$\frac{(K \cdot L)}{r_s} = 61.238$

$F_y := 50ksi$

$$\lambda := \left[\left[\frac{(K \cdot L)}{r_s}\right] \cdot \left(\frac{1}{\pi}\right)\right]^2 \cdot \left(\frac{F_y}{E}\right) = 0.655$$

$$P_n := \begin{vmatrix} \left[\frac{(0.88 \cdot F_y \cdot A_{s\_chord})}{\lambda}\right] \text{ if } \lambda > 2.25 \\ \left(0.66^{\lambda} \cdot F_y \cdot A_{s\_chord}\right) \text{ if } \lambda \le 2.25 \end{vmatrix} = 1.569 \times 10^3 \cdot kip$$

$P_r := 0.9 \cdot P_n = 1.412 \times 10^3 \cdot kip$

$$\text{Capacity} := \begin{vmatrix} \text{"OK" if } P_r > \text{Chord} \\ \text{"Does Not Pass" otherwise} \end{vmatrix} = \text{"OK"}$$

$$\text{limit\_check} := \begin{vmatrix} \text{"OK" if } \left(\frac{0.01}{K} \ge 0.0030\right) \\ \text{"Does Not Pass" otherwise} \end{vmatrix} = \text{"OK"}$$

Full Replacement

Truss Vertical Posts Resistance to Lateral Force:

$H_f := \left(\frac{0.01}{K}\right) \cdot Chord = 2.764 \cdot kip$ AASHTO Pedestrian 7.1.1

$Lateral\_moment_{vpost} := H_f \cdot Length_{vpost} = 23.491 \cdot kip \cdot ft$

HSS 8x6x6 Available Flexural Strength

$Lateral\_capacity := 73.1 kip \cdot ft$ AISC Steel Construction Manual

$$Lateral\_capacity\_check := \begin{cases} \text{"OK"} & \text{if } Lateral\_moment_{vpost} < Lateral\_capacity \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

End Posts

$C_{end\_post} := 0.01 \cdot End\_post = 1.698 \cdot kip$

Apply Cend_post at the top of the end post and check capacity

$Lateral\_moment_{epost} := C_{end\_post} \cdot Length_{end\_post} = 14.433 \cdot kip \cdot ft$

HSS 6x6x8 Available Flexural Strength: 59.3 kip-ft

$Lateral\_capacity := 59.33 kip \cdot ft$ AISC Steel Construction Manual

$$Lateral\_capacity\_check := \begin{cases} \text{"OK"} & \text{if } Lateral\_moment_{epost} < Lateral\_capacity \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Diagonals:

Loading on Diagonals (Tension):

$Diagonal = 250.3 \cdot kip$

HSS 4x4x8 Available Strength in Axial Tension

$Axial\_capacity\_tension := 271 kip$ AISC Steel Construction Manual

$$Capacity\_check := \begin{cases} \text{"OK"} & \text{if } Axial\_capacity\_tension > Diagonal \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Full Replacement

Deflection:

Check LL deflection from RISA 3D truss analysis: Use Unfactored Live Load

$$LL_{deflection\_max} := \left(\frac{1}{360}\right) \cdot L_{span} = 4.667 \cdot in$$

From seperate RISA 3D analysis:

$$\Delta_{LL} := 1.784in$$

$$Deflection\_check := \begin{vmatrix} \text{"OK"} \quad \text{if } LL_{deflection\_max} > \Delta_{LL} \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{matrix} = \text{"OK"}$$

Wind Load Deflection:

Apply horizontal wind load to the side of the truss in RISA 3D

$$WL\_max := \frac{L_{span}}{500} = 3.36 \cdot in$$

$$\Delta_{WL} := 0.37in$$

$$WL\_Deflection\_check := \begin{vmatrix} \text{"OK"} \quad \text{if } WL\_max > \Delta_{WL} \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{matrix} = \text{"OK"}$$

Full Replacement

Vibrations:

Check maximum vertical deflection of the truss due to dead load (ft)

$$g := 32.2 \frac{ft}{s^2}$$

$$\Delta_{DL} := 1.4in = 0.117 \cdot ft$$

$$f := 0.18 \sqrt{\frac{g}{\Delta_{DL}}} = 2.99 \cdot Hz$$

$$W := 2 \cdot DC \cdot L_{span} = 205.424 \cdot kip$$

$$f_{alt} := 2.86 \cdot \ln\left(\frac{180 \cdot kip}{W}\right) \cdot Hz = -0.378 \cdot Hz$$

3.0 Hz minimum desirable (f>3.0)

$$f_{min} := 3.0Hz$$

$f_{superstructure} := 3.013Hz$ From RISA Analysis

$$F_{check} := \begin{array}{|l} \text{"OK"} \quad \text{if } f_{superstructure} > f_{min} \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{array} = \text{"OK"}$$

Floorbeams:

$$W_{concrete\_deck} \cdot Spacing_{floorbeams} = 0.192 \cdot \frac{kip}{ft}$$

From Separate RISA analysis:

$$Moment_{concrete\_fbeam} := 2.5kip \cdot ft$$

Moment from vehicle load controls:

$M_{vehicle\_fb} = 46.748 \cdot kip \cdot ft$ Required Capacity

W 12x50 Available Moment Capacity: 192 kip-ft

$$Moment\_capacity := 192kip \cdot ft$$

$$Moment\_capacity\_check := \begin{array}{|l} \text{"OK"} \quad \text{if } Moment\_capacity > M_{vehicle\_fb} \\ \text{"Does Not Pass"} \quad \text{otherwise} \end{array} = \text{"OK"}$$

Full Replacement

Shear Check:

Chord Shear Check

$k_v := 5$

$$C_{v2\_chord} := \begin{cases} 1.0 & \text{if } \dfrac{h_{chord}}{t_{w\_chord}} \le 1.10 \cdot \sqrt{k_v \cdot \dfrac{E}{F_y}} \\ 1.10 \dfrac{\sqrt{k_v \cdot \dfrac{E}{F_y}}}{\dfrac{h_{chord}}{t_{w\_chord}}} & \text{if } \left(1.10 \sqrt{k_v \cdot \dfrac{E}{F_y}} < \dfrac{h_{chord}}{t_{w\_chord}} \le 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}}\right) \\ 1.51 \cdot k_v \dfrac{E}{\left(\dfrac{h_{chord}}{t_{w\_chord}}\right)^2 \cdot F_y} & \text{if } \dfrac{h_{chord}}{t_{w\_chord}} > 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \end{cases} = 1$$

$V_{n\_chord} := 0.6 \cdot F_y \cdot a_{w\_chord} \cdot C_{v2\_chord} = 568.507 \cdot kip$

From Risa 3D analysis, find max shear on the member from governing load equation

$V_{chord} := 16.032kip$

$$V_{check\_chord} := \begin{cases} \text{"OK"} & \text{if } V_{n\_chord} > V_{chord} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Full Replacement

End Post Shear Check

$$C_{v2\_epost} := \begin{cases} 1.0 & \text{if } \dfrac{h_{epost}}{t_{w\_epost}} \le 1.10 \cdot \sqrt{k_v \cdot \dfrac{E}{F_y}} \\ 1.10 \dfrac{\sqrt{k_v \cdot \dfrac{E}{F_y}}}{\dfrac{h_{epost}}{t_{w\_epost}}} & \text{if } \left(1.10 \sqrt{k_v \cdot \dfrac{E}{F_y}} < \dfrac{h_{epost}}{t_{w\_epost}} \le 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}}\right) \\ 1.51 \cdot k_v \dfrac{E}{\left(\dfrac{h_{epost}}{t_{w\_epost}}\right)^2 \cdot F_y} & \text{if } \dfrac{h_{epost}}{t_{w\_epost}} > 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_epost} := 0.6 \cdot F_y \cdot a_{w\_epost} \cdot C_{v2\_epost} = 128.438 \cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{epost} := 5.363kip$$

$$V_{check\_epost} := \begin{cases} \text{"OK"} & \text{if } V_{n\_epost} > V_{epost} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Full Replacement

Vertical Post Shear Check

$$C_{v2\_vpost} := \begin{cases} 1.0 & \text{if } \dfrac{h_{vpost}}{t_{w\_vpost}} \le 1.10 \cdot \sqrt{k_v \cdot \dfrac{E}{F_y}} \\ 1.10 \dfrac{\sqrt{k_v \cdot \dfrac{E}{F_y}}}{\dfrac{h_{vpost}}{t_{w\_vpost}}} & \text{if } \left(1.10 \sqrt{k_v \cdot \dfrac{E}{F_y}} < \dfrac{h_{vpost}}{t_{w\_vpost}} \le 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \right) \\ 1.51 \cdot k_v \dfrac{E}{\left(\dfrac{h_{vpost}}{t_{w\_vpost}} \right)^2 \cdot F_y} & \text{if } \dfrac{h_{vpost}}{t_{w\_vpost}} > 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_vpost} := 0.6 \cdot F_y \cdot a_{w\_vpost} \cdot C_{v2\_vpost} = 145.43 \cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{vpost} := 7.829 kip$$

$$V_{check\_vpost} := \begin{cases} \text{"OK"} & \text{if } V_{n\_vpost} > V_{vpost} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Full Replacement

Diagonal Post Shear Check

$$C_{v2\_dpost} := \begin{cases} 1.0 & \text{if } \dfrac{h_{dpost}}{t_{w\_dpost}} \le 1.10\cdot\sqrt{k_v\cdot\dfrac{E}{F_y}} \\ 1.10\dfrac{\sqrt{k_v\cdot\dfrac{E}{F_y}}}{\dfrac{h_{dpost}}{t_{w\_dpost}}} & \text{if } \left(1.10\sqrt{k_v\cdot\dfrac{E}{F_y}} < \dfrac{h_{dpost}}{t_{w\_dpost}} \le 1.37\sqrt{k_v\cdot\dfrac{E}{F_y}}\right) \\ 1.51\cdot k_v\dfrac{E}{\left(\dfrac{h_{dpost}}{t_{w\_dpost}}\right)^2\cdot F_y} & \text{if } \dfrac{h_{dpost}}{t_{w\_dpost}} > 1.37\sqrt{k_v\cdot\dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_dpost} := 0.6\cdot F_y\cdot a_{w\_dpost}\cdot C_{v2\_dpost} = 72.652\cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{dpost} := 0.36kip$$

$$V_{check\_dpost} := \begin{cases} \text{"OK"} & \text{if } V_{n\_dpost} > V_{dpost} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Full Replacement

Floorbeam Shear Check

$$C_{v2\_fb} := \begin{cases} 1.0 & \text{if } \dfrac{h_{fb}}{t_{w\_fb}} \le 1.10 \cdot \sqrt{k_v \cdot \dfrac{E}{F_y}} \\ 1.10 \dfrac{\sqrt{k_v \cdot \dfrac{E}{F_y}}}{\dfrac{h_{fb}}{t_{w\_fb}}} & \text{if } \left(1.10 \sqrt{k_v \cdot \dfrac{E}{F_y}} < \dfrac{h_{fb}}{t_{w\_fb}} \le 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \right) \\ 1.51 \cdot k_v \dfrac{E}{\left(\dfrac{h_{fb}}{t_{w\_fb}} \right)^2 \cdot F_y} & \text{if } \dfrac{h_{fb}}{t_{w\_fb}} > 1.37 \sqrt{k_v \cdot \dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_fb} := 0.6 \cdot F_y \cdot a_{w\_fb} \cdot C_{v2\_fb} = 135.42 \cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{fb} := P_{wheel} = 8 \cdot kip$$

$$V_{check\_fb} := \begin{cases} \text{"OK"} & \text{if } V_{n\_fb} > V_{fb} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Full Replacement

Diagonal Floorbeam Shear Check

$$C_{v2\_dfb} := \begin{cases} 1.0 & \text{if } \dfrac{h_{dfb}}{t_{w\_dfb}} < 1.10\cdot\sqrt{k_v\cdot\dfrac{E}{F_y}} \\ 1.10\,\dfrac{\sqrt{k_v\cdot\dfrac{E}{F_y}}}{\dfrac{h_{dfb}}{t_{w\_dfb}}} & \text{if } \left(1.10\sqrt{k_v\cdot\dfrac{E}{F_y}} < \dfrac{h_{dfb}}{t_{w\_dfb}} < 1.37\sqrt{k_v\cdot\dfrac{E}{F_y}}\right) \\ 1.51\cdot k_v\,\dfrac{E}{\left(\dfrac{h_{dfb}}{t_{w\_dfb}}\right)^2\cdot F_y} & \text{if } \dfrac{h_{dfb}}{t_{w\_dfb}} > 1.37\sqrt{k_v\cdot\dfrac{E}{F_y}} \end{cases} = 1$$

$$V_{n\_dfb} := 0.6\cdot F_y\cdot a_{w\_dfb}\cdot C_{v2\_dfb} = 165.375\cdot kip$$

From Risa 3D analysis, find max shear on the member from governing load equation

$$V_{dfb} := 8kip$$

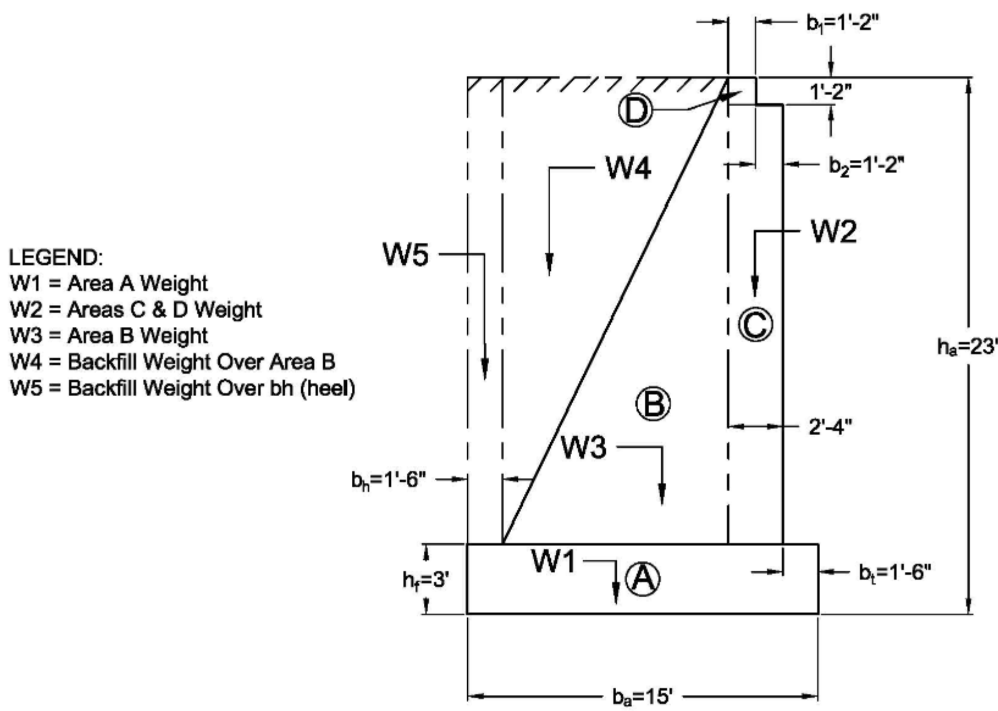
$$V_{check\_dfb} := \begin{cases} \text{"OK"} & \text{if } V_{n\_dfb} > V_{dfb} \\ \text{"Does Not Pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Appendix H: Substructure Full Replacement

Full Replacement Substructure Design

Refernces:

- Design of Reinforced Concrete Structures
- Army Field Manual 3-34-343



$W_{bridge} := 16ft$ — Width of bridge

$LL_{pedestrian} := 90psf$ — Live load on superstructure

$\gamma_{soil} := 120\frac{lb}{ft^3}$ — Assumed unit weight of sand and gravel, from Design of Concrete Structures, soil type from existing bridge plans

$\gamma_{concrete} := 150\frac{lb}{ft^3}$ — Assume unit weight of reinforced concrete

$Length := W_{bridge} + 4ft = 20\cdot ft$ — Add 2 ft to either side of superstructure on abutment

$clearance_{height} := 17ft$ — From required bridge clearance

$Skew := 25deg$

Vertical load from superstructure (per truss)

Assume cohesionless soil and active condition

$\phi_{prime} := 30deg$

$K_a := \tan\left[45deg - \left(\frac{\phi_{prime}}{2}\right)\right]^2 = 0.333$ Active earth pressure

$H_{prime} := \frac{LL_{pedestrian}}{\gamma_{soil} \cdot g} = 0.75\,ft$ Equivalent height of surcharge live load

$fill := 3ft$ Required fill over footing

$h_f := 3ft$ Height of footing

$h_a := clearance_{height} + fill + h_f = 23\,ft$

$\mu := 0.4$ Coefficient of friction Design of Concrete Strucutes

$b_1 := 14in$

$b_2 := 14in$

$b_t := 1.5ft$

$b_h := 1.5ft$

$b_a := (15ft) - 3ft + b_t + b_h = 15\,ft$

$Area_A := h_f \cdot b_a = 45\,ft^2$

$B_{width} := b_a - b_t - b_h - b_1 - b_2 = 9.667\,ft$

$Area_B := (h_a - h_f) \cdot B_{width} \cdot 0.5 = 96.667\,ft^2$

$C_{width} := b_a - b_t - b_h - B_{width} = 2.333\,ft$

$Area_C := C_{width} \cdot (h_a - h_f - b_2) = 43.944\,ft^2$

$Area_D := b_1{}^2 = 1.361\,ft^2$

Centroids:

X bar:

$$Area_{A\_x} := \frac{b_a}{2} = 7.5\,ft$$

$$Area_{B\_x} := \left[B_{width}\cdot\left(\frac{2}{3}\right)\right] + b_t = 7.944\,ft$$

$$Area_{C\_x} := b_t + B_{width} + \left(0.5\cdot C_{width}\right) = 12.333\,ft$$

$$Area_{D\_x} := b_t + B_{width} + \left(0.5\cdot b_1\right) = 11.75\,ft$$

$$Volume := \left(Area_A + Area_B + Area_C + Area_D\right)\cdot 20ft = 3.739\times 10^3\cdot ft^3$$

y bar:

$$Area_{A\_y} := \frac{h_f}{2} = 1.5\,ft$$

$$Area_{B\_y} := \left[\left(h_a - h_f\right)\cdot\left(\frac{1}{3}\right)\right] + h_f = 9.667\,ft$$

$$Area_{C\_y} := \frac{\left(h_a - h_f - b_2\right)}{2} + h_f = 12.417\,ft$$

$$Area_{D\_y} := b_1\cdot 0.5 + h_a - b_1 = 22.417\,ft$$

$$Centroid_x := \left(Area_A\cdot Area_{A\_x}\right) + \left(Area_B\cdot Area_{B\_x}\right) + \left(Area_C\cdot Area_{C\_x}\right) + \left(Area_D\cdot Area_{D\_x}\right) = 1.663\times 10^3\cdot ft^3$$

$$Centroid_y := \left(Area_A\cdot Area_{A\_y}\right) + \left(Area_B\cdot Area_{B\_y}\right) + \left(Area_C\cdot Area_{C\_y}\right) + \left(Area_D\cdot Area_{D\_y}\right) = 1.578\times 10^3\cdot ft^3$$

$$Sum_{areas} := Area_A + Area_B + Area_C + Area_D = 186.972\,ft^2$$

Abutment Centroid:

$$X_{bar} := \frac{Centroid_x}{Sum_{areas}} = 8.897\ \text{ft}$$

$$Y_{bar} := \frac{Centroid_y}{Sum_{areas}} = 8.44\ \text{ft}$$

Centroid of Stem:

$$Centroid_{x\_CandD} := \left(Area_D \cdot Area_{D\_x}\right) + \left(Area_C \cdot Area_{C\_x}\right) = 557.975 \cdot \text{ft}^3$$

$$Sum_{areaC\_and\_D} := Area_C + Area_D = 45.306\ \text{ft}^2$$

$$X_{bar\_stem} := \frac{Centroid_{x\_CandD}}{Sum_{areaC\_and\_D}} = 12.316\ \text{ft}$$

$$W_{4\_x} := b_t + \left[B_{width} \cdot \left(\frac{1}{3}\right)\right] = 4.722\ \text{ft}$$

Vertical Weights:

$$W_1 := Area_A \cdot \gamma_{concrete} \cdot 1\,\text{ft} = 6.75 \times 10^3\ \text{lb}$$

$$W_2 := \left(Area_C + Area_D\right) \cdot \gamma_{concrete} \cdot 1\,\text{ft} = 6.796 \times 10^3\ \text{lb}$$

$$W_3 := Area_B \cdot \gamma_{concrete} \cdot 1\,\text{ft} = 1.45 \times 10^4\ \text{lb}$$

$$W_4 := Area_B \cdot \gamma_{soil} \cdot 1\,\text{ft} = 1.16 \times 10^4\ \text{lb}$$

$$W_5 := b_t \cdot \left(h_a - h_f\right) \cdot \gamma_{soil} \cdot 1\,\text{ft} = 3.6 \times 10^3\ \text{lb}$$

Moments From Vertical Loads:

$Moment_1 := W_1 \cdot Area_{A\_x} = 5.063 \times 10^4 \, lb \cdot ft$

$Moment_2 := W_2 \cdot X_{bar\_stem} = 8.37 \times 10^4 \, lb \cdot ft$ W2 Includes areas 3 and 4 (stem weight)

$Moment_3 := W_3 \cdot Area_{B\_x} = 1.152 \times 10^5 \, lb \cdot ft$

$Moment_4 := W_4 \cdot W_{4\_x} = 5.478 \times 10^4 \, lb \cdot ft$

$Moment_5 := W_5 \cdot b_t \cdot 0.5 = 2.7 \times 10^3 \, lb \cdot ft$

Table X. Vertical Loads and Moments Developed over Abutment

	Vertical Force (kip)	Moment Arm (ft)	Bending Moment (kip*ft)
W1	4.95	5.5	27.225
W2	6.6405	8.31	55.182555
W3	8.07975	5.28	42.66108
W4	6.4638	3.39	21.912282
W5	3.42	0.75	2.565
Total	**29.55405**		**149.545917**

$Sum_W := W_1 + W_2 + W_3 + W_4 + W_5 = 4.325 \times 10^4 \, lb$

$Sum_M := Moment_1 + Moment_2 + Moment_3 + Moment_4 + Moment_5 = 3.07 \times 10^5 \, lb \cdot ft$

$P_a := 0.5 \cdot K_a \cdot \gamma_{soil} \cdot h_a \cdot \left[h_a + \left(2 \cdot H_{prime}\right)\right] \cdot 1ft = 1.127 \times 10^4 \, lb$ Horizontal Force on Abutment

$$y := \frac{\left(h_a^{\,2} + 3 \cdot h_a \cdot H_{prime}\right)}{3 \cdot \left[h_a + \left(2 \cdot H_{prime}\right)\right]} = 7.901 \, ft$$ Height of Horizontal Force

$M_o := P_a \cdot y = 8.905 \times 10^4 \, lb \cdot ft$

$e := \dfrac{(Sum_M - M_o)}{(Sum_W)} = 5.04 \, ft$

Distance of the Resultant of Vertical Forces from Origin

$SF := \dfrac{Sum_M}{M_o} = 3.447$

Safety Factor for Overturning Moment

$$Safety_{Factor} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } SF > 1.5 \\ \text{"Does Not Pass"} \quad \text{if } SF < 1.5 \end{array} \right. = \text{"OK"}$$

$q := \dfrac{Sum_W}{b_a \cdot 1ft} \cdot \left[1 + \dfrac{(6 \cdot e)}{b_a} \right] = 8.695 \times 10^3 \dfrac{lb}{ft^2}$

Pressure on Soil

$q := \dfrac{Sum_W}{b_a \cdot 1ft} + \left[\dfrac{(6 \cdot M_o)}{(1ft)(b_a)^2} \right] = 5.258 \times 10^3 \dfrac{lb}{ft^2}$

$$Bearing_{Pressure} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } q < 8000 \dfrac{lb}{ft^2} \\ \text{"Does Not Pass"} \quad \text{if } q > 8000 \dfrac{lb}{ft^2} \end{array} \right. = \text{"OK"}$$

$F := \mu \cdot Sum_W = 1.73 \times 10^4 \, lb$

Frictional Resistance

$FS := \dfrac{F}{P_a} = 1.535$

$$Friction_{Resistance} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } FS > 1.5 \\ \text{"Does Not Pass"} \quad \text{if } FS < 1.5 \end{array} \right. = \text{"OK"}$$

Appendix I: Material Cost Estimate

MATERIAL COST ESTIMATE REPAIR					
Item No.	**Description**	**Quantity**	**Unit**	**Unit Cost**	**Cost**
106.302	Clean and Paint Structural Steel	7250	SF	$100.00	$725,000.00
127.100	Reinforced Concrete Excavation	30	CY	$3,000.00	$90,000.00
181.140	Disposal of Hazardous Waste	230	TON	$400.00	$92,000.00
450.321	Superpave Intermediate Course 19.0	21	TON	$134.00	$2,814.00
450.600	Superpave Bridge Surface Course 9.5	13	TON	$163.00	$2,119.00
645.097	120-in Chain Link Fence	270	FT	$52.59	$14,199.30
				TOTAL COST:	**$926,132.30**

MATERIAL COST ESTIMATE REPAIR

Caculated by: NB	Checked by: IMO	Date: 11/13/2019
Item No. 106.302	Description Clean and Paint Structural Steel	Quantity: 7250 SF

Say 2 Girders/Bridge

Girders:			**24 WF 110**		
Bridge Length	113.167	ft	Bridge Width	14.1666667	ft
Girder Height	6	ft	Beam Height	2	ft
Flange width	1.5	ft	Flange width	1	ft
Flange height	0.04167	ft	Flange Height	0.07125	ft
Number Girders	2		Number Beams	18	
Girder Surface Area:	2055.87	ft^2	Beam SA:	117.370833	ft^2
Total Girder SA:	4111.73	ft^2	Total Beam SA:	2112.675	ft^2

24 WF 84			**15 I 42.9**		
Beam Length	4.58333	ft	Beam Length	5.83333333	ft
Beam Height	2	ft	Beam Height	1.25	ft
Flange width	0.75	ft	Flange width	0.45833333	ft
Flange Height	0.06433	ft	Flange Height	0.05729167	ft
Number Beams	2		Number Beams	36	
Beam SA:	33.2628	ft^2	Beam SA:	26.6145833	ft^2
Total Beam SA:	66.5256	ft^2	Total Beam SA:	958.125	ft^2

*Beam SAs are overconservative to include paint removal and repair on the top flange, it may be covered by concrete

Total Area:	7249.05989	SF
SAY	**7250**	SF

MATERIAL COST ESTIMATE REPAIR

Caculated by: NB		Checked by: IMO	Date: 11/13/2019
Item No. 127.10	Description	Reinforced Concrete Excavation	Quantity: 30 CY

Bridge Length	115	ft
Curb Height	1.875	ft
Curb Width	1.75	ft
Number Curbs	2	
Volume of Curbs	754.688	ft^3
Volume of Curbs	27.9514	CY
SAY	30	CY

MATERIAL COST ESTIMATE REPAIR

Caculated by: NB		Checked by: IMO	Date: 11/13/2019	
Item No.	181.14	Description Disposal of Hazardous Waste	Quantity:	230 TON

Contaminated Ballast

Height Ballast	1.875	ft
Length of Bridge	115	ft
Width of Bridge	14.1667	ft
Volume Ballast	3054.69	CF
Weight Ballast	150	pcf
Weight of Disposal	458203	lbs
	229.102	TON
SAY	230	TON

*conservatively assume 150 pcf

MATERIAL COST ESTIMATE REPAIR

Caculated by: NB		Checked by: IMO	Date: 11/13/2019
Item No. 450.321	Description	Superpave Intermediate Course 19.0	Quantity: 21 TON

Bridge Length:	115	ft
Width Paved	12	ft
Depth Pavement	0.20833	ft
Unit Weight	140	pcf
Weight of pavement	40250	lbs
	20.125	TON
SAY	21	TON

MATERIAL COST ESTIMATE REPAIR

Caculated by: NB		Checked by: IMO		Date: 11/13/2019	
Item No.	450.60	Description	Superpave Bridge Surface Course 9.5	Quantity:	13 TON

Bridge Length:	115	ft
Width Paved	12	ft
Depth Pavement	0.125	ft
Unit Weight	140	pcf
Weight of pavement	24150	lbs
	12.075	TON
SAY	13	TON

MATERIAL COST ESTIMATE REPAIR

Caculated by: NB		Checked by: IMO		Date: 11/17/2019	
Item No.	644.097	Description	120-in Chain Link Fence	Quantity:	270 FT

Length of Bridge	115	ft
Extra Length on ends	20	ft
Number of fences	2	
Length of fence	270	ft
SAY	270	FT

MATERIAL COST ESTIMATE SUPERSTRUCTURE REPLACEMENT					
Item No.	**Description**	**Quantity**	**Unit**	**Unit Cost**	**Cost**
645.097	120-in Chain Link Fence	280	FT	$52.59	$14,725.20
904.000	4000 psi, 3/4 inch, 565 cement concrete	4	CY	$1,200.00	$4,800.00
909.000	Lightweight Cement Concrete	11	CY	$1,300.00	$14,300.00
960.000	Structural Steel	81200	LB	$12.33	$1,001,196.00
				TOTAL:	$1,035,021.20

MATERIAL COST ESTIMATE SUPERSTRUCTURE REPLACEMENT

Caculated by: NB	Checked by: IMO	Date: 11/14/2019

Item No.	645.097	Description	120-in Chain Link Fence	Quantity:	280 FT

Length of Bridge	116	ft
Extra Length on ends	20	ft
Number of fences	2	
Length of fence	272	ft
SAY	280	FT

MATERIAL COST ESTIMATE SUPERSTRUCTURE REPLACEMENT

Caculated by: NB		Checked by: AD		Date: 11/22/2019	
Item No.	904	Description	4000 psi, 3/4 inch, 565 cement concrete	Quantity:	4 CY

Abutment Width	21.281	ft
Cap height	0.5	ft
Width	3.83	ft
Number of Abutments	2	
Volume	81.506	ft^3
	3.0187	CY
SAY	4	CY

MATERIAL COST ESTIMATE SUPERSTRUCTURE REPLACEMENT

Caculated by: NB	Checked by: IMO	Date: 11/14/2019
Item No. 909	Description Lightweight Cement Concrete	Quantity: 11 CY

Bridge Length:	116	ft
Brigdge Width:	14	ft
Concrete Thickness	0.166667	ft
Volume	270.6667	CF
Total Volume	10.02469	CY
SAY	11	CY

MATERIAL COST ESTIMATE SUPERSTRUCTURE REPLACEMENT

Caculated by: NB	Checked by: IMO	Date: 11/14/2019
Item No. 960	Description Structural Steel	Quantity: 81200 LB

		Length:	Number:	Weight:
Chords:				
Top	76.33 plf	116 ft	2	17708.56 lb
Bottom	76.33 plf	116 ft	2	17708.56 lb
End Post	35.24 plf	7 ft	4	986.72 lb
Vertical Post	32.58 plf	7 ft	30	6841.8 lb
Diagonal Post	25.03 plf	10.08 ft	32	8073.677 lb
Floorbeams	22 plf	15.476 ft	17	5788.024 lb
Diagnal Floorbeams	31 plf	14.051 ft	16	6969.296 lb

Metal Decking:

Weight	10.5 psf
Bridge Length	116 ft
Bridge Width	14 ft
Metal Decking Total:	17052 lb

TOTAL: 81128.6 LB
SAY: 81200 LB

MATERIAL COST ESTIMATE FULL REPLACEMENT					
Item No.	**Description**	**Quantity**	**Unit**	**Unit Cost**	**Cost**
120.000	Earth Excavation	2400	CY	$33.64	$80,736.00
156.100	Crushed Stone for Bridge Foundation	0.5	TON	$55.16	$27.58
645.097	120-in Chain Link Fence	320	FT	$52.59	$16,828.80
904.000	4000 psi, 3/4 inch, 565 cement concrete	140	CY	$1,200.00	$168,000.00
909.000	Lightweight Cement Concrete	14	CY	$1,300.00	$18,200.00
910.000	Steel Reinforcement for Structures	6000	LB	$2.79	$16,740.00
960.000	Structural Steel	148360	LB	$12.33	$1,829,278.80
				Total	$2,129,811.18

MATERIAL COST ESTIMATE FULL REPLACEMENT

Caculated by: AD	Checked by: IMO	Date: 11/22/2019
Item No. 120	Description Earth Excavation	Quantity: 2400 CY

Per abutment

over wingwall footings	1248	CF
2' to the sides of footings	448	CF
1:1 slopes	3456	CF
between wingwalls	19931.8	CF
2' behind wingwall approach	1008.62	CF
1:1 behind approach	5925.64	CF
Total	1185.85	CY
Two Abutment Total	**2371.71**	**CY**

Total Area:	2371.7	CY
SAY	**2380**	CY

MATERIAL COST ESTIMATE FULL REPLACEMENT

Caculated by: IMO		Checked by: NB	Date: 11/17/2019
Item No. 645.097	Description	120-in Chain Link Fence	Quantity: 320 FT

Length of Bridge	140	ft
Extra Length on ends	20	ft
Number of fences	2	
Length of fence	320	ft
SAY	320	FT

MATERIAL COST ESTIMATE FULL REPLACEMENT

Caculated by: JB		Checked by: IMO	Date: 11/18/2019	
Item No. 156.10	Description	Crushed Stone for Bridge foundation	Quantity:	0.5 TON

At least 1 ft^3 crushed stone is required per weeping hole

Weeping holes	6	
Stone/hole	1	CF
Total	6	CF
Density of Crushed Stone	100	pcf
Weight	600	lbs
	0.3	TON

Total Weight:	0.3	TON
SAY	**0.5**	TON

MATERIAL COST ESTIMATE FULL REPLACEMENT

Caculated by: JB		Checked by: IMO		Date: 11/25/2019	
Item No.	904.000	Description	4000 psi, 3/4 inch, 565 cement concrete	Quantity:	140 CY

Assume footing only extends 20' wide. Calculations can be found in Appendix of paper and were completed in Mathcad.

Total Volume	3739	CF
Total Area	138.481	CY
SAY	**140**	CY

MATERIAL COST ESTIMATE FULL REPLACEMENT

Caculated by: IMO	Checked by: NB	Date: 11/20/2019
Item No. 909.000	Description Lightweight Cement Concrete	Quantity: 14 CY

Bridge Length	140	ft
Bridge Width	16	ft
Concrete Thickness	0.16667	ft

Total Volume:	373.33333	CF
Total Volume:	13.82716	CY
SAY	**14**	CY

MATERIAL COST ESTIMATE FULL REPLACEMENT

Caculated by: AD	Checked by: NB	Date: 11/25/2019
Item No. 910.000	Description Steel Reinforcement for Structures	Quantity: 6000 LB

Number of Abutments	2	
Length of Abutment	20	ft

Transverse #5 Bars			**Longitudinal #5 Bars**		
Diameter	0.625	in	Diameter	0.625	in
Area	0.31	in^2	Area	0.31	in^2
Weight	1.043	plf	Weight	1.043	plf
Length of Bar	50	ft	Length of Bar	19	ft
Number of bars	18		Number of bars	47	
Weight per abut	938.7	lbs	Weight per abut	931.399	lbs
Total Weight	1877.4	lbs	Total Weight	1862.8	lbs

#7 Bars Transverse			**Longitudinal #7 Bars**		
Diameter	0.875	in	Diameter	0.875	in
Area	0.6	in^2	Area	0.6	in^2
Weight	2.044	plf	Weight	2.044	plf
Length of Bars	14.5	ft	Length of bars	19	ft
Number of bars	18		Number of bars	15	
Weight per abut	533.484	lbs	Weight per abut	582.54	
Total Weight	1066.968	lbs	Total Weight	1165.08	

Total Area:	5972.25	LB
SAY	**6000**	LB

MATERIAL COST ESTIMATE FULL REPLACEMENT

Caculated by: IMO	Checked by: NB	Date: 11/20/2019
Item No. 960	Description Structural Steel	Quantity: 148360 LB

Members:		Length:	Number:	Weight:
Top Chord	149.61 plf	140 ft	2	41890.8 lb
Bottom Chord	149.61 plf	140 ft	2	41890.8 lb
End Post	35.24 plf	8.5 ft	4	1198.16 lb
Vertical Post	32.58 plf	8.5 ft	26	7200.18 lb
Diagonal Post	25.03 plf	13.12 ft	28	9195.02 lb
Floorbeams	50 plf	17.687 ft	15	13265.3 lb
Diagonal Floorbeams	45 plf	16.188 ft	14	10198.4 lb

Metal Decking:	
Weight	10.5 psf
Bridge Length	140 ft
Bridge Width	16 ft
Metal Decking Total Weight	23520 lb

Total Weight:	148359 LB
SAY	**148360** LB

Appendix J: Typical Connection Design

Welded Connections:

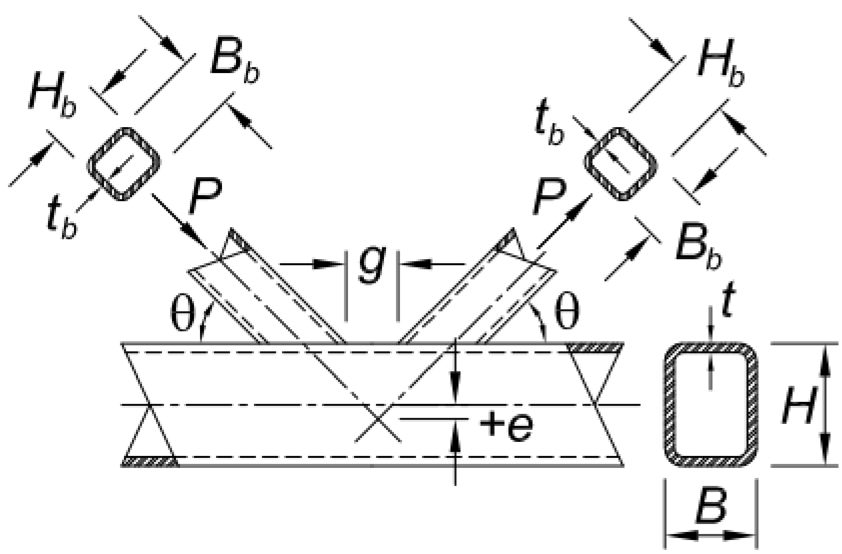
Refernces:

- AISC Steel Construction Manual, 15th Edition
- AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2017

Check weld connections at center connection, K-connection with vertical post in center, make sure K-shape can hold capacity of all three branches

Bottom Chord: HSS 14x14x7/8
Diagonals: HSS 4x4x1/2
Vertical Posts: HSS 8x6x3/8

Gapped K-Connections



$e := 0in$ Eccentricity

$H := 14in$ Height of chord

$B := 14in$ Width of chord

$t := 0.814in$ Thickness of chord

$H_b := 4in$ Height of branch

$B_b := 4in$ Width of branch

$t_b := 0.465in$ Thickness of branch

$x := 10ft$ Spacing

$y := 8.5ft$ Height of truss

$$\theta := \operatorname{atan}\left(\frac{y}{x}\right) = 40.365 \cdot deg$$

$g := 10.29in$ Gap, calculated in AutoCAD

$$\zeta := \frac{g}{B} = 0.735$$ Gap ratio

$$Perimeter_{diagonal} := 2 \cdot (H_b + B_b) = 16 \cdot in$$

$$Perimeter_{vertical} := 2 \cdot (8in + 6in) = 28 \cdot in$$

$$\beta_{eff} := \frac{(Perimeter_{diagonal} + Perimeter_{vertical})}{(8B)} = 0.393$$

$\gamma := \frac{B}{2 \cdot t} = 8.6$ Chord slenderness ratio

$F_y := 50ksi$ Yield strength of steel

$E := 29000ksi$ Modulus of elasticity of steel

$$F_c := F_y = 50 \cdot ksi$$

$$A_g := 6.02in^2$$

$$S := 5.47in^3$$

Based on M123 from RISA, lower compressive forces:

$M_{ro} := 0.431\text{kip}\cdot\text{ft}$

$P_{ro} := 18.306\text{kip}$

$$U := \left|\left(\frac{P_{ro}}{F_c\cdot A_g}\right) + \left(\frac{M_{ro}}{F_c\cdot S}\right)\right| = 0.08$$

$$Q_f := 1.3 - 0.4\left(\frac{U}{\beta_{eff}}\right) = 1.219$$

$$P_n := \frac{\left[F_y\cdot t^2\left(9.8\cdot\beta_{eff}\cdot\gamma^{0.5}\right)\cdot Q_f\right]}{\sin(\theta)} = 703.911\cdot\text{kip}$$

$\phi := 0.90$

$P_{diagonal\_1} := P_{ro} = 18.306\,\text{kip}$ M123

$P_{diagonal\_2} := 21.386\text{kip}$ M122

$P_{vertical} := 1.55\text{kip}$ M109

$P := P_{diagonal\_1} + P_{diagonal\_2} + P_{vertical} = 41.242\,\text{kip}$

$$\text{Chord\_wall\_plastification\_check} := \begin{vmatrix} \text{"OK"} \quad \text{if } \phi\cdot P_n \geq P \\ \text{"Does not pass"} \quad \text{otherwise} \end{vmatrix} = \text{"OK"}$$

Table K3.2A: Limits of Applicability of Table K3.2

$$\text{Joint\_eccentricity} := \begin{cases} \text{"OK"} & \text{if } -0.55 \le \dfrac{e}{H} \le 0.25 \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{Chord\_slenderness} := \begin{cases} \text{"OK"} & \text{if } \left(\dfrac{B}{t} \le 35\right) \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{Branch\_wall\_slenderness}_B := \begin{cases} \text{"OK"} & \text{if } \dfrac{B_b}{t_b} \le \min\left(35, 1.25\sqrt{\dfrac{E}{F_y}}\right) \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{Branch\_wall\_slenderness}_H := \begin{cases} \text{"OK"} & \text{if } \dfrac{H_b}{t_b} \le \min\left(35, 1.25\sqrt{\dfrac{E}{F_y}}\right) \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{Aspect\_ratio\_chord} := \begin{cases} \text{"OK"} & \text{if } 0.5 \le \dfrac{H}{B} \le 2.0 \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{Aspect\_ratio\_branch} := \begin{cases} \text{"OK"} & \text{if } 0.5 \le \dfrac{H_b}{B_b} \le 2.0 \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{Material\_strength} := \begin{cases} \text{"OK"} & \text{if } F_y \le 52\text{ksi} \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{Width\_ratio}_B := \begin{cases} \text{"OK"} & \text{if } \dfrac{B_b}{B} \geq 0.1 + \dfrac{\gamma}{50} \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{Width\_ratio}_H := \begin{cases} \text{"OK"} & \text{if } \dfrac{H_b}{B} \geq 0.1 + \dfrac{\gamma}{50} \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\beta_{eff\_check} := \begin{cases} \text{"OK"} & \text{if } \beta_{eff} \geq 0.35 \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{gap\_ratio} := \begin{cases} \text{"OK"} & \text{if } \zeta \geq 0.5 \cdot (1 - \beta_{eff}) \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\text{gap} := \begin{cases} \text{"OK"} & \text{if } g \geq 2 \cdot t_b \\ \text{"Does not pass"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$t_{weld} := \begin{cases} \left(\dfrac{1}{4}\text{in}\right) & \text{if } \max(t, t_b) \leq 0.75\text{in} \\ \left(\dfrac{5}{16}\text{in}\right) & \text{otherwise} \end{cases} = 0.313\ \text{in}$$

AASHTO Table 6.13.3.4-1

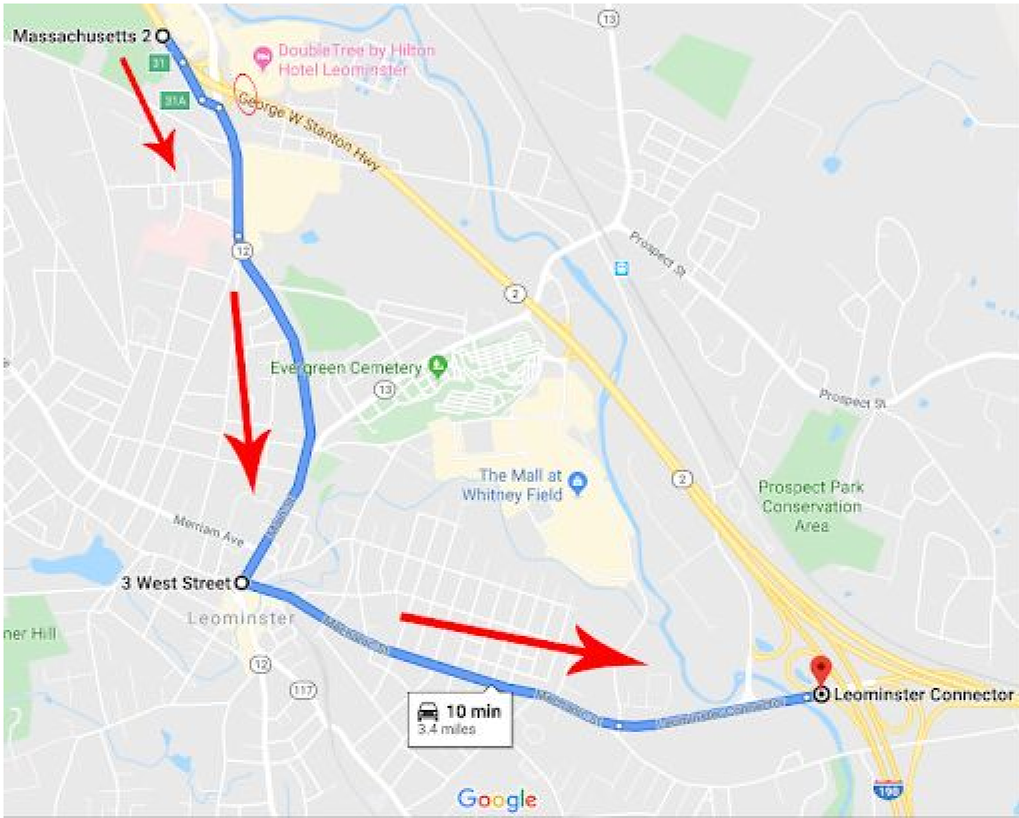
$$l_{weld} := \text{Perimeter}_{diagonal} = 16\ \text{in}$$

$$D := \left(\frac{t_{weld}}{\text{in}}\right) \cdot 16 = 5$$

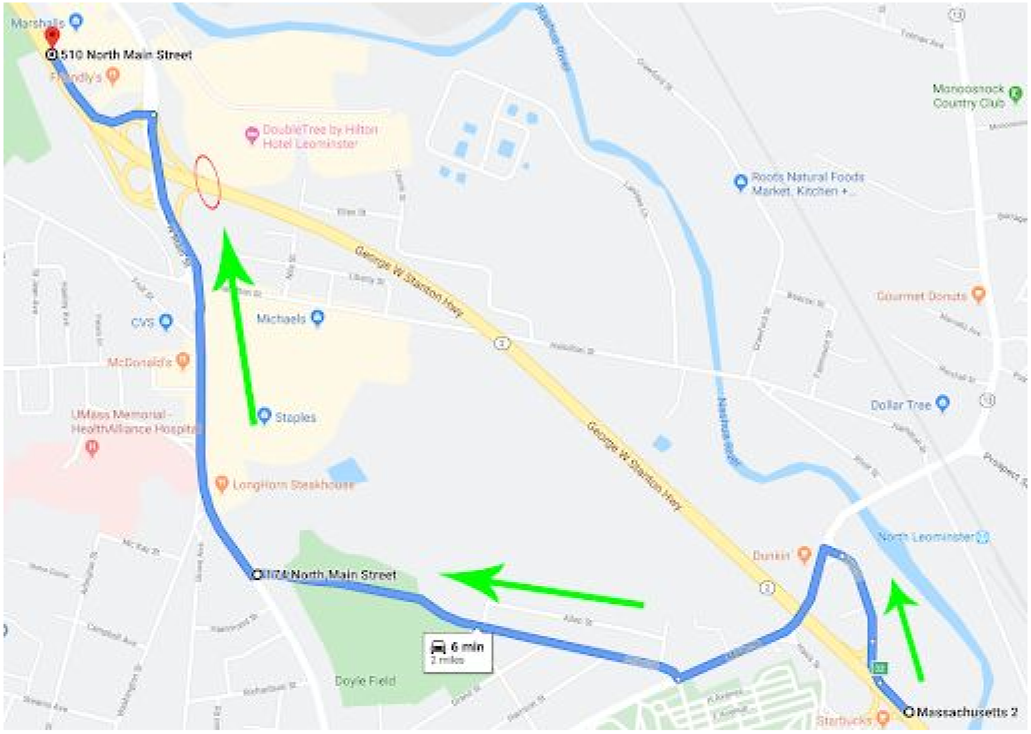
$$\phi R_n := \left(1.392\ \frac{\text{kip}}{\text{in}}\right) \cdot D \cdot l_{weld} = 111.36\ \text{kip}$$

Appendix K: Alternative Traffic Detour Routes

Alternative Traffic Detour Routes



Eastbound


Marshalls
510 North Main Street
Friendly's
DoubleTree by Hilton Hotel Leominster
Roots Natural Foods Market, Kitchen +...
Monoosnock Country Club
George W Stanton Hwy
Michaels
CVS
McDonald's
Staples
UMass Memorial - HealthAlliance Hospital
LongHorn Steakhouse
Gourmet Donuts
Dollar Tree
North Leominster
Dunkin'
1174 North Main Street
6 min
2 miles
Doyle Field
Starbucks
Massachusetts 2

Westbound