Project Number: LDA-1301

# Quaboag River Bridge Replacement Design

## A Major Qualifying Project Report

**Submitted to the Faculty**

**Of**

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## Abstract

The Quaboag River Bridge located in Brookfield, MA is to be replaced through the Accelerated Bridge Program. In this Major Qualifying Project, alternative designs for the Quaboag River Bridge were investigated and evaluated based on a set of established criteria. As a result of the evaluation process a prestressed concrete, spread box girder design was created based on AASHTO LRFD Bridge Design Specifications. The proposed design includes a completed superstructure, substructure, 3D model and life-cycle cost analysis.

## Authorship

The Abstract, Authorship, Capstone Design, Introduction, and Background chapters were equally contributed to by Lauren D’Angelo, Madison Shugrue, and Mariah Seaboldt.  All other elements of the project were collaborated on, but headed and written individually.  The following individuals were responsible for the specific project elements listed below:

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## Capstone Design

In our Major Qualifying Project, alternative designs for a single span bridge were investigated and evaluated based on a set of established criteria. As a result of the evaluation process, we created a prestressed concrete, spread box girder design based on AASHTO LRFD Bridge Design Specifications. This design included a completed superstructure, substructure, 3D model and life-cycle cost analysis. To satisfy the ABET Capstone Design requirements, our project addressed realistic sustainable, environmental, ethical, manufacturability, economic, social, political, and health and safety constraints of the Quaboag River Bridge Replacement project. For our project's bridge design, we chose to use concrete girders rather than steel girders after researching sustainable, environmental, ethical, and manufacturability evaluation criteria. We also ran life-cycle cost analyses (LCCAs) of a concrete and steel girder bridge using the program *BridgeLCC* from the National Institute of Standards and Technology (NIST). Since Massachusetts will only fund a concrete sustainable bridge design, the LCCAs allowed political, economic, and social considerations of the replacement bridge project to be addressed. Ethical and health and safety considerations were met by abiding to AASHTO LFRD Bridge Design Specifications through the entirety of the project. With these combined efforts, our project satisfied the ABET Capstone Design requirements.

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## Executive Summary

The purpose of this Major Qualifying Project (MQP) was to complete a design for the Quaboag River Bridge in Brookfield, MA. This steel girder bridge is currently in the process of being replaced by the Massachusetts Department of Transportation (MassDOT) as part of the Accelerated Bridge Program (ABP) (MassDOT, 2010). The ABP's purpose is to determine whether deteriorating bridges are structurally deficient. In Massachusetts a bridge is considered structurally deficient if it is rated to be in a condition of 4 or less on a scale of 0-9 (0 being the lowest rating, 9 being the highest) (MassDOT, 2010). A deteriorating bridge is classified as a preservation, rehabilitation, or replacement project. Design of the maintenance, repair, or replacement project begins once the ABP classifies a bridge.

Inspection of the Quaboag River Bridge for the ABP determined that the bridge must be replaced to satisfy 50-Year-Flood-Plain requirements. The proposed MassDOT design for the bridge is a concrete spread box girder design that not only meets the flood requirements, but also improves the visibility for approaching traffic and the vertical geometry of the bridge (Depaola & Broderick 2012).

The purpose of this MQP was to create a supplementary design for the Quaboag River Bridge by investigating both a steel girder bridge design as well as a concrete bridge design. Once the two types of designs were evaluated, the full design of the bridge was completed using the selected material.

The two types of designs were initially evaluated using a decision method developed by Hunter and Stewart (2002). Through this method a set of criteria was established which consisted of life expectancy, environmental impact, ethics, and timeline. The criteria were used to evaluate the success or failure of a certain design. Once this set of criteria were established, a hierarchy of the criteria was determined, a baseline was established, each alternative was evaluated, and concepts were compared (Hunter & Stewart 2002). The two alternatives were compared to a "no build" scenario, and an index value was calculated for the three alternatives (steel, concrete and "no build"). The values obtained for the steel and concrete designs were compared to each other as well as to the no-build option to determine the superior design choice.

Ultimately through the evaluation processes, it was determined that the concrete bridge design was more beneficial than the steel bridge design. As a result, a completed concrete spread box girder bridge design was developed to replace the existing Quaboag River Bridge. The design of the concrete bridge consisted of both superstructure and substructure elements of the bridge. Due to time constraints, these two components were designed separately, using assumed load values for the substructure's design. The capacity of the substructure was compared to the superstructure's final loads when they were developed. The design was deemed adequate if the substructure capacity was greater than the superstructure loading.

The design of the superstructure consisted of prestressed concrete girders and as a result, included the determination of the live load and dead load effects on the girders, cross-sectional geometry of the girders and design of the presetressing steel, including the prestressing forces and tendon profiles. To determine the live load magnitude and effect on the girders, a design truck and design lane load from the American Association of State Highway Transportation Officials (AASHTO) LRFD Bridge Specifications were utilized in conjunction with *RISA-2D* software. The prestressed steel design was based on examples from Arthur Nilson's book Design of Prestressed Concrete and the Precast/Prestressed Concrete Institute's (PCI) Bridge Design Manual.

The final design of the bridge superstructure consists of six prestressed concrete spread box girders spaced 8' on center. The resulting girder cross-section was determined to be box beams measuring 45 inches deep and 54 inches wide with 12 inches thick flange and web walls. A total of 75 post-tensioned steel strands were selected to be used in the girders. These strands were specified to be combined into 15 tendons (5 strands per tendon) and placed into 5 post-tensioning ducts (3 tendons per duct). The ducts are to be placed a distance of 16.5 inches from the centroid of the girder cross section.

Development of the substructure design relied on *Microsoft Excel* sheets to facilitate the calculation of ultimate bearing capacities during the design process. The substructure had three design stages, each assuming the worst-case site scenario of an entirely "sandy soil" soil profile and a Factor of Safety of 3.5. Sandy soil was considered the worst case scenario since sandy soil is the least stable of soils and has the smallest internal friction angle. Assuming sandy soil resulted in an extremely low bearing capacity of the spread footing design, forcing the group to investigate a deep foundation option for the substructure.

The first design alternative that was investigated was a shallow spread footing foundation that resulted in an inadequate bearing capacity. The second design alternative, a deep pile foundation, resulted in a large bearing capacity that well-exceeded the load that was to be exerted from the superstructure.

The final substructure design consisted of an abutment wall, bridge seat, backwall, pile cap, and 10 piles. Appropriate flexural and shear reinforcement was designed for these elements according to AASHTO and the American Concrete Institute (ACI) specifications. The final bearing capacity of the substructure was 33.58 ksf which far exceeded loads exerted on the piles.

In addition to the structural bridge design, a cost analysis of the bridge was also completed. Using the software *BridgeLCC* from the National Institute of Standards and Technology (NIST), the lifecycle cost of the bridge design was determined for a 100-year service life. The inflation rate of 1.90% and real discount rate of 2.30% were taken directly from the Bureau of Economic Analysis and Office of Management and Budget government websites respectively. Results of the life-cycle cost analysis showed that the project's concrete bridge design would be both economical and sustainable.

The work completed through this MQP has satisfied the capstone design requirements of the Civil and Environmental Engineering Department of Worcester Polytechnic Institute (WPI). Although significant work has been completed for this MQP, future work that can be evaluated includes consideration of diaphragms and composite action in the superstructure as well as steel H piles for scour protection and wing walls for erosion control in the substructure. Recommendations to future MQPs include the use of *RISA-2D* or other similar software to aid in loading calculations, as well as completing the design of the superstructure and substructure simultaneously to facilitate the completion of the design in a more timely fashion.

# 1. Introduction

Roadway and highway bridges are essential components of modern infrastructure, however as time goes by, these pieces of infrastructure are growing increasingly deficient. As of December 2012, there were 151,497 deficient bridges across the United States (FHWA, 2013). Although this number is over 2,000 less than was reported a year earlier, it is clear that bridge rehabilitation and replacement is necessary across the United States.

Although upgrading bridges is essential to ensure America's infrastructure remains safe for public use, it is also important to perform construction projects efficiently with the least disruption possible. In Massachusetts, the Accelerated Bridge Program (ABP) was created to help renovate the State's bridge infrastructure. The Massachusetts Department of Transportation (MassDOT) and the Department of Conservation and Recreation (DCR) are jointly inspecting over 4,500 bridges to ensure that they meet federal standards. If the bridges do not meet federal standards, MassDOT and DCR are working to rehabilitate or replace the deficient bridges (Telegram & Gazette, 2012).

One of the bridges under inspection through the ABP was the Quaboag River Bridge in Brookfield, MA. The bridge was built in 1936 and after inspection it was determined that the bridge did not meet federal 50-Year Flood Plain requirements. As a result, MassDOT plans to replace the current steel girder bridge with a concrete design (Depaola & Broderick 2012).

It was the purpose of this Major Qualifying Project (MQP) to evaluate two designs to replace the Quaboag River Bridge. A steel girder and concrete girder design were evaluated based on a set of established evaluation criteria. These criteria included considerations for life expectancy, cost, environmental impact, timeline and ability for ethical design. After each of the alternatives was evaluated and compared to a "no build" scenario, the superior alternative was selected and a design of the superstructure and substructure was completed based on this alternative.

In addition to the superstructure and substructure designs, a life-cycle cost analysis (LCCA) was also completed. For comparison purposes both the steel and concrete alternatives were included in the LCCA. The two elements that most influenced the cost of the designs were the real discount rate and the inflation rate. These two components were investigated as part of this project to evaluate real world feasibility of the bridge designs.

The following report presents the work completed for this MQP on the Quaboag River Bridge. Background information is provided on the bridge itself as well as vital bridge components that were investigated during the project. In addition, the methods used to complete the project are outlined and described. The results obtained are presented through drawings of bridge components and a cumulative 3D model of the bridge design. LCCA data is also presented in the form of graphs that compare the different alternatives over a designated service life.

# 2. Background

## 2.1 Introduction

This background provides information about the existing and proposed Quaboag River Bridge that serves as the base for our project. It also presents information on precast spread box girder designs as well as a steel girder designs which were evaluated as replacement alternatives as part of this MQP. Additionally, information is provided regarding other project components such as the bridge deck, the substructure, the decommissioning and construction process, the evaluation criteria and the cost analysis process used for the bridge design.

### 2.1.1 Accelerated Bridge Program

As discussed in the Introduction, MassDOT and DCR have been working on inspecting and rating bridges throughout Massachusetts since 2008. These inspections are part of the ABP and consist of examining vital bridge elements such as the deck, superstructure, substructure, and rate of deterioration. In Massachusetts a bridge is considered structurally deficient if it is rated to be in a condition of 4 or less on a scale of 0-9 (0 being the lowest rating, 9 being the highest). “Structurally deficient” does not necessarily mean that a bridge is unsafe, though it does mean that the bridge has the potential of becoming unsafe to transportation if its deficiencies are not repaired or attended to (MassDOT, 2010).

Currently the ABP has identified over 500 bridges as structurally deficient. Once a bridge is identified as structurally deficient, it is classified as a preservation, rehabilitation, or replacement project. Preservation projects are the least costly and have the shortest design and construction period whereas rehabilitation and replacement projects involve the replacement of major bridge elements and are more design and cost extensive. (MassDOT, 2010)

### 3.1.2 Bridge No. B-26-002

The ABP bridge replacement project for Bridge No. B-26-002 is located on Fiskdale Road (Route 148) in Brookfield, Massachusetts (Figure 1). This bridge spans over the Quaboag River and does not meet federal 50-Year Flood Plain requirements. It needs to be raised to a minimum height of 6’ from the Quaboag’s average water level to pass federal standards. The proposed Quaboag bridge design from MassDOT provides 6 feet of clearance under the bridge, shifts the bridge easterly on its northern end, and eliminates a dip between itself and an existing CSX Bridge[1] (Figure 2), while aesthetically matching

[1] CSX is a railway freight corporation headquartered in Jacksonville, Florida (CSX, 2012)

the existing roadway connecting the CSX Bridge.  This proposed design improves visibility and vertical geometry of the roadway.  A comparison of the existing and proposed bridge design elements of the Quaboag River Bridge can be viewed in Table 1. (Depaola & Broderick, 2012)


Figure 2 - Location of Bridge - Brookfield, MA

Figure 1 - Dip between Existing Bridge and CSX Bridge

**Table 1 - Existing and Proposed Design Elements**

| Design Element | Existing Quaboag River Bridge | Proposed Bridge |
|---|---|---|
| ***# of Girders*** | 5W x 4L | N/A |
| ***# of I-Beams*** | 6 | 6 Type NEBT 1200 Beams |
| ***# of Spans*** | 2 | 1 of 98' |
| ***Abutments*** | Concrete Gravity abutments | Full Height Concrete Abutments |
| ***Approaching Roadway Shoulder*** | N/A | 4' |
| ***Approaching Roadway Width*** | 24' | 32' |
| ***Construction Period*** | Completed in 1936 | 2012-2013: 2 construction seasons |
| ***Cost*** | N/A | $8.35 mil |
| ***Drainage System Elements*** | N/A | Replaced for the new roadway profile |
| ***Height from River's Average Water Height*** | 5.5' | 6' |
| ***Length*** | 79' | 98' |
| ***Loading Tolerance*** | 18 tons for 2 axle; 22 tons for 3 axle; 34 tons for 5 axle | HL-93 loading in accordance with current AASHTO LRFD and MassDOT standards |
| ***Pier Elements*** | 1 Center Pier | Center pier will be removed to streambed level |
| ***Scouring Elements*** | N/A | Old Abutments will remain as a scouring element |
| ***Sidewalks*** | 6'W E side of bridge; 1'W safety-walk W side of bridge | Two 5.5' W sidewalks on either side of the bridge |
| ***Slope Stabilization*** | N/A | Modified riprap |
| ***Steel Railing*** | both sides of bridge | both sides of bridge: S3-TL4 Bridge Rail |
| ***Superstructure Material*** | N/A | Precast Concrete girders: Boxed Tee or spread box beam stringers with 8in HP 4000psi concrete deck and 3.5in hot mix-asphalt wearing surface |
| ***Utilities*** | N/A | Will be designed to accommodate a possible future water line |

As shown in Table 1, the major design changes of the bridge are:

- Type of superstructure design and material
- Extension of the bridge’s length

- Widening of the bridge
- Removal of the center pier
- Placement of the new abutments

The details concerning these major design changes and their options are discussed later in the Background. Other design elements, such as the type of railings, vehicle design loads, and sidewalk design follow Massachusetts State regulations. These changes are due to changes in the utility of the bridge. For example, larger trucks are now using the bridge than when it was constructed in 1936 so the vehicle loads must be changed to accommodate this new traffic. Also, there is more pedestrian traffic on the bridge because in this area the Quaboag River is a popular fishing and recreation location. Therefore railings will be type S3-TL4 and the sidewalks are designed to be 5.5’ on both sides of the bridge (Figure 2). These design changes will meet safety regulations for the foot-traffic using the bridge (MassDOT, 2010).

The Quaboag Bridge crosses a section of the river that is surrounded by a wide marsh with no clearly defined banks. Trout Brook is immediately upstream of the bridge site, and the river flows into Quaboag Pond downstream. This area of the river receives a significant amount of recreational river traffic during the fair to nice weather seasons. Directly north-east of the bridge is White’s Landing, a small Mom-and-Pop business. White’s Landing can accommodate approximately 5 vehicles in its gravel parking lot and has a small boat launch for its customers east of the bridge. Images of the Quaboag Bridge’s surrounding area can be viewed below (Figure 3).

**Figure 3 - (Left) Upstream from the Bridge and (Right) Downstream from the Bridge**

The marsh and lands adjoining the bridge are owned by the Massachusetts Department of Fish and Game, Division of Conservation and Recreation, the Consolidated Rail Corporation, and the business owners of White's Landing (Depaola & Broderick, 2012). Due to the mixed public and private ownership of the surrounding area and the wetlands, Brookfield needs to obtain the correct documentation to move forward with this project. These document requirements, which can be found on the Massachusetts Department of Environmental Protection (MassDEP) website, include wetland permits, wetland transmittal forms, and water quality certifications (MassDEP, 2012). Brookfield also needs to impose permanent and temporary easements on White's Landing to accommodate the bridge construction (Depaola & Broderick, 2012). This has created a debate between the town and White's Landing owners, significantly slowing progress of the project (Telegram & Gazette, 2012).

White's Landing owners want an identical bridge superstructure to the existing bridge so it does not affect their business, while Massachusetts wants a longer lasting structure that is economically beneficial. The similar steel superstructure design would last potentially for 25 years before needing attention or repair (Telegram & Gazette, 2012). It would also not require permanent easements on White's Landing, whereas a longer lasting concrete superstructure bridge would. A public hearing was held on May 15, 2012 to inform the community of the State's proposed design and its considerations. The design was not passed at a Brookfield town meeting on June 8, 2012 and was scheduled to be reviewed again on September 7, 2012. No local newspaper articles were posted about the outcome of the meeting as of December 2012. Brookfield must pass the State's concrete superstructure design at its next town meeting or the Massachusetts Department of Transportation will abandon the project and Brookfield will need to design and repair the bridge without Massachusetts funding (Telegram & Gazette, 2012).

## 2.2 Spread Box Girder Bridges

As discussed in the previous sections, the proposed design for the replacement of the Quaboag River Bridge is a spread box girder design (Federal Highway Administration, 2006). This type of design was used as one of the comparative designs for investigation in this project. The following subsections discuss aspects of this type of design and the main components that comprise a spread box girder bridge. Steel girder bridges, which were also evaluated as an alternative for the bridge replacement project, are discussed later in the Background.

### 2.2.1 Girder Design

A spread box girder bridge consists of prestressed concrete girders known as box girders. The girder has a rectangular cross-section with a rectangular void through the center. A typical box girder cross-section is depicted in Figure 4 below (Federal Highway Administration, 2006). Typical span lengths for box girders vary between 20 and 90 feet. Common cross-section widths of the girders are 36 or 46 inches, while the depth can range from 27 to 42 inches, and the thickness of the web wall varies between 3 and 6 inches (Federal Highway Administration, 2006).

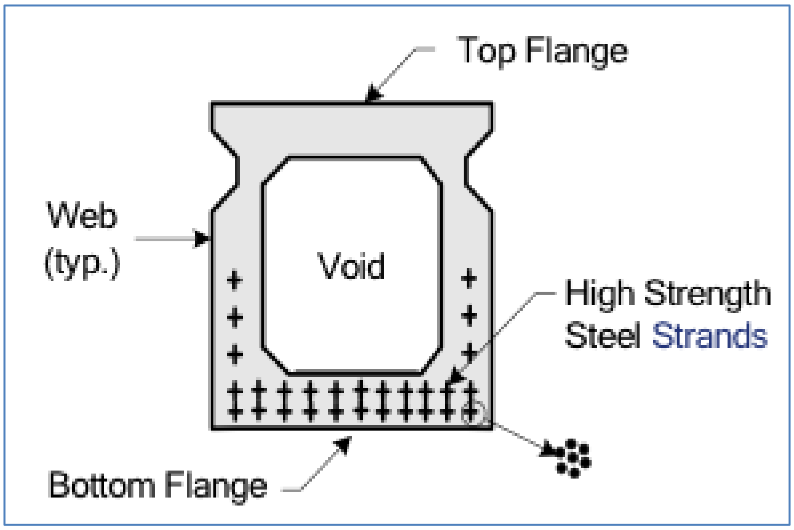


Figure 4 - Box Girder Cross Section (Federal Highway Administration, 2006)

Box girder bridges can be implemented in one of two ways. The configuration proposed for the replacement of the Quaboag River Bridge is known as a spread box design (Federal Highway Administration, 2006). This bridge uses box girders spaced across the width of the bridge, as shown in Figure 5 below. Box girders can also be used in the design of adjacent box girder bridges. As the name implies, these bridges have box girders placed next to each other with no spacing between them (Federal Highway Administration, 2006).

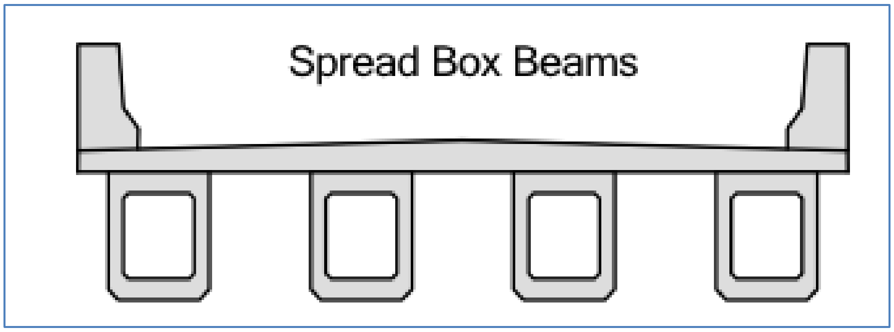


Figure 5 - Spread Box Bridge layout (Federal Highway Administration, 2006)

In spread box girder bridges the box girders are usually spaced between 2 to 6 feet apart. These bridges also may involve diaphragms between the box girders to provide additional reinforcement. Diaphragms may be placed at the midpoints, endpoints or at distances one third the length of the span. If the diaphragms are placed at the endpoints of the span, they will be located at the abutments or piers. The diaphragms can be made out of cast-in place concrete, precast concrete, or steel (Federal Highway Administration, 2006).

### 2.2.2 Steel Reinforcement Placing

Figure 4 illustrates typical placing for steel reinforcing bars in box girders. Common strand sizes for reinforcements used in box girders are 1/4, 3/8, 7/16 and 1/2 inch diameter steel rods. Strands are normally spaced every 2 inches, but both strand size and spacing can vary depending on concrete characteristics (Federal Highway Administration, 2006).

Other steel placement in box girders can occur if the girders are to be used as composite girders. Composite girders have additional steel stirrups that are placed at the top of the girder and extend out of the concrete. The purpose of these stirrups is to establish a mechanical connection between the box girders and the cast-in-place concrete deck; the steel will transfer horizontal shear forces between both the deck and the girder, and the two elements will act as a single section. Figure 6 illustrates the placement of the steel stirrups as well as possible placement of secondary reinforcing strands (Federal Highway Administration, 2006).

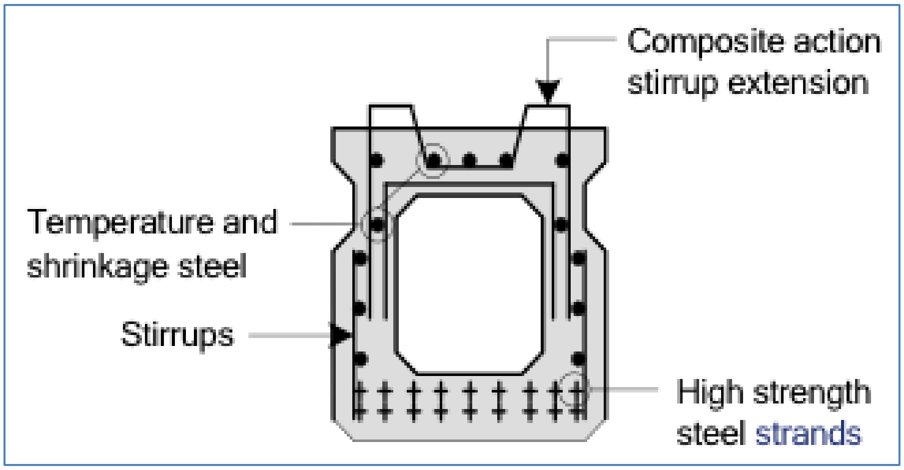


Figure 6 - Composite and Secondary Steel Placement (Federal Highway Administration)

### 2.2.3 Advantages and Disadvantages of Box Girders

The use of box girders and other precast bridge components can affect the bridge and construction process in both positive and negative ways. The physical design of the box girder results in a girder that is both strong and light. Since the center of the girder is hollow there is a reduced amount of dead load experienced by the structure. However, the girder still supports design moments and shears (Federal Highway Administration, 2006). The relatively shallow depth of the box girder also allows it to be used when there are low clearances between the bridge and whatever it is spanning. Typically these girders have a maximum depth around 42 inches (Federal Highway Administration, 2006).

Box girders can also save time during the construction process. Since they are precast concrete structures, they can be manufactured offsite, which allows other construction or planning processes to take place while the girders are being built. Once the girders are built they can be installed quickly with minimum disruption of traffic (Federal Highway Administration, 2006).

Although these girders can be advantageous in certain situations, there are some disadvantages to their use as a result of the concrete material used in the construction. The concrete girder can experience cracking due either to flexure, shear, or temperature changes. Shrinkage and delamination can also occur between the concrete and reinforcing bars. The concrete used in the box girders is also subject to spalling, efflorescence, collision damage, overload damage, and general wear and abrasion damage. Corrosion of the steel reinforcing strands can also cause problems within the girder, leading to loss of tensile strength and other issues (Federal Highway Administration, 2006).

Table 2 below summarizes the advantages and disadvantages discussed above for precast concrete box girders. Much more detail can be provided for box girders and spread box bridges, but the focus of this section is to provide an overview of the design. A similar overview of steel girder bridges is provided in the next section.

Table 2 - Advantages and Disadvantages of Precast Concrete Box Girders

| Advantages | Disadvantages |
|---|---|
| Center void reduces dead load | Concrete cracking |
| Reduces traffic disruption during construction | Delamination |
| Allows for low levels of clearance | Spalling, efflorescence, wear and abrasion damage |
| Saves time during construction | Collision and overload damage |
| | Corrosion of steel reinforcements |

## 2.3 Steel Box Girder Bridge

A steel box girder bridge design was the second comparative design considered in this project. The following subsections discuss the configuration options, primary and secondary members, stiffeners, fatigue and fracture critical areas, and the deck interaction of steel box girder bridges.

### 3.3.1 Steel Box Girder Design

Steel box girder bridges are supported by one or more welded steel box girders. Steel box girders can have either rectangular or trapezoidal cross sections (Figure 7). The cross section consists of two or more web plates connected to a single bottom flange plate. There are two span options for box girder bridges: simple spans of 75 feet or more, or continuous spans of 100 feet or more (Federal Highway Administration, 2006).

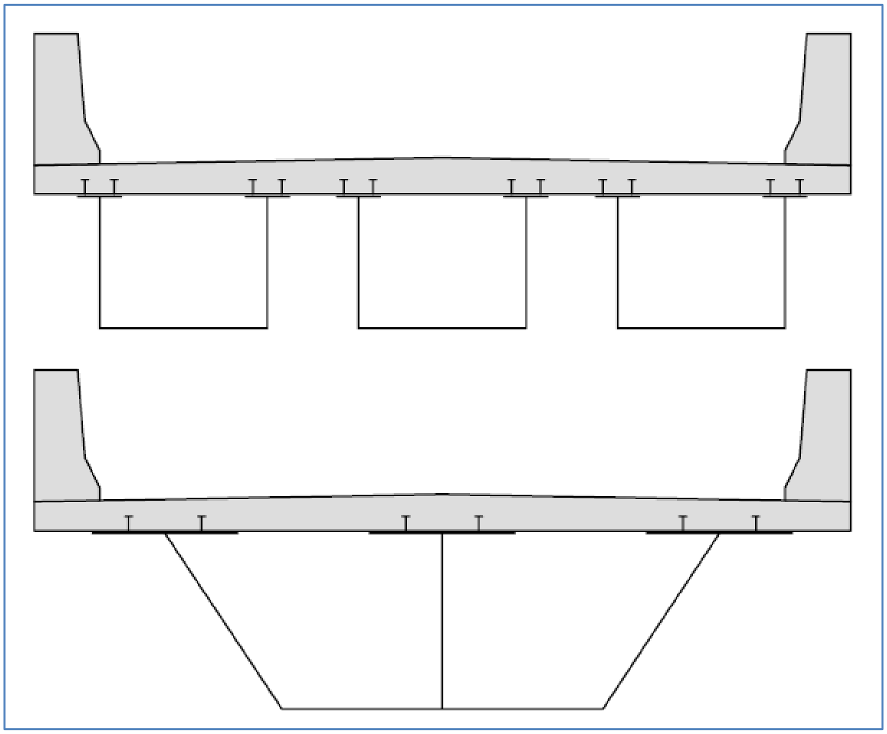

Figure 7 - Steel Box Girder Rectangular and Trapezoidal Cross Sections (Federal Highway Administration, 2006)

A steel box girder bridge can be designed with a single box configuration or a spread box configuration. Factors such as deck width, span length, terrain, and aesthetics need to be considered when determining the configuration. The primary members of box girder bridges are the box girders and all internal bracings. Diaphragms can consist of solid plates, rolled shapes, or cross frames constructed with angles, tee shapes, and plates (Federal Highway Administration, 2006). They can also be on the interior or exterior of the box. Exterior diaphragms are typically used between box girders on multi-box bridges.

Since steel box girder bridges are susceptible to buckling from diagonal compression effects due to torsional and shear forces, stiffeners must be used in areas of compression for the webs and bottom flange of large box shapes. The purpose of stiffeners is to increase the stability of the box girder by limiting the unsupported length of the web and bottom flange (Federal Highway Administration, 2006).

Fatigue and fracture are two types of failure that need to be accounted for in the design of a steel box girder bridge. Fatigue prone areas are:

- Welded attachments inside the box

- Attachment welds in the tension zone
- Butt welds in adjacent longitudinal stiffeners
- Intersecting welds between webs and flanges
- Field splices

Fatigue cracks can also result from web-gap distortion and out-of-plane distortion. The box girders are considered fracture critical members of box girder bridges when a span has two or less box girders making the structure nonredundant (Federal Highway Administration, 2006).

Steel box girder bridges have two deck options: a composite deck, or an orthotropic steel plate deck. A composite deck consists of the top flange plates incorporating shear connectors and a composite superstructure with a concrete deck. When using a composite deck, the deck and the superstructure work together to carry the live load (Figure 8). On the other hand, an orthotropic steel plate deck is comprised of a top flange consisting of a single plate extending the width of the box and a wearing surface on the top flange (Figure 9). Further detail of bridge decking materials is provided in the next section.

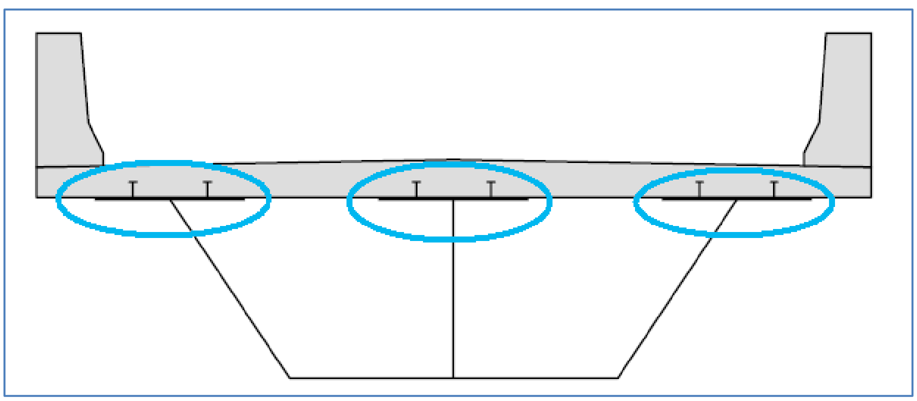

**Figure 8 - Box Girder Cross Section with Composite Deck (Federal Highway Administration, 2006)**

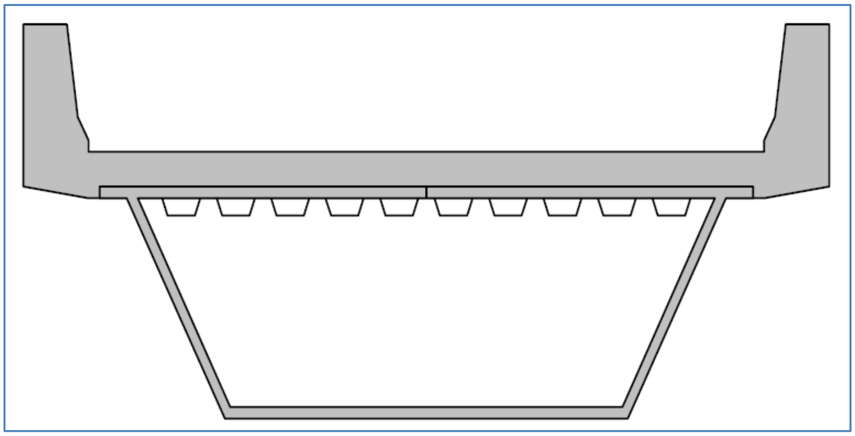

Figure 9 - Box Girder Cross Section with Orthotropic Steel Plate Deck (Federal Highway Administration, 2006)

## 2.4 Bridge Decks

This section provides information on the three types of bridge decks: timber, steel and concrete. First, an overview is provided of timber and steel decks. Typical designs, wearing surfaces, protective coatings and some advantages and disadvantages are briefly discussed for the steel and timber decks. However, concrete decks are discussed in more detail, since they are the most common type of bridge deck material.

### 2.4.1 Timber Decking

Timber is rarely used in modern bridge structures and decking. About 7% of all bridges in the National Bridge Inventory are timber and of steel bridges, only 7% of these bridges have timber decking (Federal Highway Administration, 2006). Even though timber isn't used very often in bridge design, there are still some advantages and disadvantages to its use. This section gives an overview of timber decking and why it may be used in certain situations.

#### *2.4.1.1 Types of Timber Decking*

There are multiple different types of timber decking to choose from (Federal Highway Administration, 2006). The following are the four main types of timber decking:

- Plank decks
- Nailed laminated decks
- Glued-laminated (Glulam) deck panels
- Stressed-laminated deck panels

- Structural composite lumber decks

These different types of timber decking utilize varying placement of wood along the bridge deck as well as different material make-ups. For example, plank decks and nailed laminated decks are made of timber planks, whereas glue-laminated, stressed-laminated and structural composite lumber are made up of composite wood pieces held together by an adhesive (Federal Highway Administration, 2006). Some of the deck types have members running transversely across the bridge while others run longitudinally (Federal Highway Administration, 2006). Some types, like Glulam, come in panels as opposed to planks (Federal Highway Administration, 2006).

#### *2.4.1.2 Wearing Surfaces*

In addition to the different types of timber decking available there are also different wearing surfaces used in conjunction with timber decking. In some instances, the wearing surface is another layer of timber that is placed along the projected wheel path of traffic (Federal Highway Administration, 2006). Other wearing surfaces include bituminous (asphalt) mixtures and concrete. However, bituminous mixtures and concrete are not used for certain types of timber decking because the deck can crack when the timber members deflect (Federal Highway Administration, 2006).

#### *2.4.1.3 Protective Coatings*

Wearing surfaces provide some protection to timber decks, but additional protective systems must be used to prevent decay (Federal Highway Administration, 2006). Water repellants, preservatives, fumigants, fire retardants and paint are all protective coatings that can be used on timber decking. Most of these coatings help prevent decay of the wood, but fire retardants have the important function of slowing the spread of fire through the decking. Steel decks and concrete decks do not utilize fire retardant coatings, but they do need protection from decay and corrosion (Federal Highway Administration, 2006).

#### *2.4.1.4 Advantages and Disadvantages*

Even though timber decks are not widely used, they do have certain advantages. Although timber does decay, it is resistant to deicing chemicals, which will harm concrete and steel decks (Federal Highway Administration, 2006). Timber is also a renewable material, easy to fabricate and lightweight, which helps reduce dead load (Federal Highway Administration, 2006).

However, there are also some disadvantages to timber, which may be why it is not as commonly used. As mentioned above the fire hazard presented by timber is more than that of steel or concrete. Timber is also susceptible to insect, fungus and parasite damage, and may deflect excessively or split (Federal Highway Administration, 2006). Other possible disadvantages include:

- Checks
- Shakes
- Loose connections
- Surface depressions
- Chemical attack

Timber decking is not very widely used in bridge construction, however in certain situations it may be the right choice for design and material.

### 2.4.2 Steel Decking

Steel decking is used more frequently than timber decking, but is still not as widely used as concrete decking. Steel decks are often found on older bridges and may be used for rehabilitation of bridge decks or for bridges with very long spans (Federal Highway Administration, 2006). Steel decks are also often used if dead load is a concern in a bridge design because steel decks weigh less than concrete decks (Federal Highway Administration, 2006).

#### *2.4.2.1 Types of Steel Decking*

Each type of steel decking utilizes different designs. Some designs leave the steel members of the deck and superstructure exposed, while others can be partially or fully filled with concrete. The four main types of steel bridge decks are (Federal Highway Administration, 2006):

- Orthotropic decks
- Buckle plate decks
- Corrugated steel flooring
- Grid decks.

Grid decks can be subcategorized as: welded grid decks, exodermic grid decks, and riveted grid decks (Federal Highway Administration, 2006).

#### *2.4.2.2 Wearing Surfaces*

Like timber decks, steel decks can have different wearing surfaces. If grid decks are open (not filled with concrete) then the serrated edges of the grating act as the wearing surface. If a steel deck is filled with concrete then the concrete acts as the wearing surface. Otherwise asphalt may be used, especially in orthotropic decks (Federal Highway Administration, 2006).

#### *2.4.2.3 Means of Protection*

Steel decks must be protected from wear and corrosion because they are often more exposed to weather and loading conditions than other types of decking, and they sometimes leave the superstructure exposed as well (Federal Highway Administration, 2006). A variety of paints can be used to protect steel from moisture, oxidation, and chlorides. Paint is usually applied as a primer, intermediate coat, and then topcoat. The steel can also be protected through a galvanization process, which coats the steel in zinc. Galvanized steel will not corrode as fast because the outer coating of zinc will corrode instead of the steel. Some steel decks are also protected through an epoxy coating that shields them from corrosive elements. However, epoxy coatings on steel decks are rare (Federal Highway Administration, 2006).

#### *2.4.2.4 Advantages and Disadvantages*

As with timber and any other type of design, there are disadvantages and advantages to the use of steel decks. As mentioned above, steel is lighter in weight than concrete decks, reducing the demands on the superstructure for a given span and live load. However, since a steel deck often leaves the superstructure exposed, it can result in more corrosion and a shorter lifespan of both the deck and bridge. In addition to corrosion, the other main structural disadvantages to steel are fatigue cracks and other stress cracks (Federal Highway Administration, 2006).

### 2.4.3 Concrete Decks

The most common type of bridge decking is a concrete deck. Concrete can be formed into different shapes and as a result, it can be used effectively in many different types of bridge construction (Federal Highway Administration, 2006). Concrete decks can also be composite or non-composite. If composite, the deck is physically joined to the superstructure, creating a stronger and stiffer structure (Federal Highway Administration, 2006). The following subsections discuss the types of concrete decks currently in use as well as wearing surfaces, protective systems and other aspects of the decks.

#### *2.4.3.1 Concrete Deck Types*

Concrete decks can be broken up into two main types: reinforced cast-in-place and precast. Precast concrete decks include normal precast panels, prestressed precast panels and prestressed precast panels with cast-in-place top (Federal Highway Administration, 2006).

Cast-in-place concrete decks are made onsite by placing wet concrete into either permanent or temporary forms. Temporary forms are usually made of wood, while permanent, or stay-in-place, forms are made of corrugated metal. Before the concrete is placed into the forms, steel reinforcing bar (rebar) is laid. The concrete is then placed over these bars, and when it cures, the deck is established (Federal Highway Administration, 2006). The purpose of rebar is to help increase the tensile strength of the concrete. Without this rebar concrete would have very weak tensile strength and would not be a good choice for bridge decking. The primary reinforcing bars are laid so that they will be at the top and bottom of the concrete deck. Primary bars carry the main tensile stress developed within the concrete. Secondary bars are placed perpendicular to the primary reinforcement and serve to stabilize the concrete against stresses that develope as a result of temperature changes and shrinkage (Federal Highway Administration, 2006).

Precast deck panels are also made of reinforced concrete, but instead of being formed onsite, they are formed and cured offsite. The panels are then brought to the construction site and put in place when necessary. They are attached to the superstructure with either mechanical clips or shear connectors. Prior to being attached though, the panels are leveled using leveling bolts or grout (Federal Highway Administration, 2006).

Prestressed precast panels are constructed in the same way as regular precast panels; however, prestressed panels also have prestressed steel reinforcement. Tension is applied to the prestressing components before the panels are formed and are held in tension until the concrete has cured. As a result, once the panels are formed, the tensioned bars are exerting compression forces on the concrete itself. This helps to reduce the amount of cracking experienced by the concrete (Federal Highway Administration, 2006).

Prestressed precast panels with a cast-in-place top are simply prestressed panels that have been put in place as the bridge deck and are then overlaid with a cast-in-place top. The panels act as forms and the cast-in-place overlay becomes composite with both the deck and the superstructure (Federal Highway Administration, 2006).

### *2.4.3.2 Other Similar Deck Materials*

Two newer deck materials are being used in similar ways as concrete decks. Fiber reinforced polymer (FRP) uses glass fibers as reinforcement in polyester or vinyl ester resins. Similar to precast panels FRP decks are usually formed in panels at a factory offsite and shipped to the construction site. The panels are then put together using adhesives and attached to the superstructure. FRP decks can be compositely attached to the superstructure through the use of grout (Federal Highway Administration, 2006).

Fiber reinforced concrete (FRC) is also another new type of bridge deck material. In FRC decks, Portland cement is combined with polypropylene fibers. The addition of these fibers helps reduce cracking of the concrete due to shrinkage and increases the impact strength of the concrete once it's cured. Steel reinforcing bars may or may not be used in an FRC bridge deck (Federal Highway Administration, 2006).

### *2.4.3.3 Wearing Surfaces*

As with steel and timber decking, concrete decks also utilize wearing surfaces. Either a concrete or asphalt wearing surface is normally used for concrete decks. Concrete wearing surfaces can either be integral or overlay. An integral concrete wearing surface is cast with the deck. Once this integral surface has worn down it is replaced with an overlay (Federal Highway Administration, 2006). Overlay concrete surfaces are cast after the concrete deck is in place and used. Some overlay types are:

- Low slump dense concrete (LSDC)
- Latex modified concrete (LMC)
- Lightweight concrete (LWC)
- Fiber reinforced concrete (FRC)

Each concrete overlay has different characteristics and is used for different reasons (Federal Highway Administration, 2006). FRC was discussed in the previous subsection, but is often used as a deck surface in order to prevent cracking. LSDC has a low water to cement ratio and as a result cures very quickly. LSDC is so dense that it doesn't allow penetration by chlorides and can be effective in areas where deicing products are used. However, it is subject to cracking so an LSDC overlay must be resurfaced after about 25 years (Federal Highway Administration, 2006).

LMC is a mixture of Portland cement and latex solids. LMC is more expensive to make than LSDC, but is easier to place (Federal Highway Administration, 2006). The addition of latex into a cement concrete mixture reduces the amount of mixing water needed. With less water necessary, the resulting concrete has a high compressive strength, meaning it will experience less cracking and be more resilient against corrosive agents like water and chlorides (BASF Corporation, 2011).

LWC incorporates lighter aggregates within its mixture and has a higher amount of entrained air. As the name suggests this makes for a significantly lighter product, reducing the dead load experienced by the structure. LWC is not only used in overlays, but in precast and cast-in-place bridge decks as well (Federal Highway Administration, 2006).

In addition to using concrete surfaces, asphalt is often used atop concrete bridge decks. Asphalt layers can often be between 1 and 2.5 inches thick and may be placed after a waterproof membrane is laid on the deck. This membrane helps prevent the penetration of corrosive agents into the concrete (Federal Highway Administration, 2006).

#### *2.4.3.4 Means of Protection*

The main protection for concrete decks involves preventing the steel reinforcing bars from corroding. Sealants can be placed atop concrete decks to help stop chlorides from penetrating the deck and corroding the steel. Common sealants include boiled linseed oil and elastomeric membranes (Federal Highway Administration, 2006).

Some steel reinforcing bars are made to help prevent corrosion and therefore deterioration of concrete decks. Bars with an epoxy coating will resist corrosion from chemicals and moisture and as discussed in steel decking, some steel bars undergo a galvanization process. Stainless steel bars and fiber reinforced polymer bars are also sometimes used for reinforcement because they do not corrode. Fiberglass reinforced polymer bars are significantly lighter than steel bars and as a result, may also be used in concrete decks (Federal Highway Administration, 2006).

Waterproof membranes can be used to protect concrete decks and prevent corrosion of steel reinforcing bars. Two types of membranes are self-adhering membranes and liquid waterproofing membranes. These membranes will help reduce cracking of the concrete and penetration of water into the deck (Federal Highway Administration, 2006).

#### *2.4.3.5 Advantages and Disadvantages*

Concrete decks are the most commonly used bridge decks for several reasons. For example, they can be molded to fit many different shapes and sizes and sometimes can be cast offsite, improving concrete quality and saving construction time. Concrete is also a low cost material that does not require painting for long-term durability. There are many different types of concrete to choose from, allowing a designer to pick and choose the option that best works for a given project (Federal Highway Administration, 2006).

There are some disadvantages to the use concrete that must be taken into consideration. The main disadvantage comes from the corrosion of the steel reinforcing bars that are used within the deck. The corrosion of these bars can result in a loss of tensile strength within the deck. The deck is also subject to cracking, scaling, spalling and other problems associated with environmental exposure of the concrete (Federal Highway Administration, 2006).

#### *2.4.3.6 Deck Conclusions*

Regardless of the deck a designer chooses, there will be advantages and disadvantages involved. Although concrete decks are most commonly used in bridge design, there may be instances where steel or timber decks are desired. Concrete decks have proven to be the most versatile of designs, even though there are certain disadvantages that come along with their use.

## 2.5 Substructure

The substructure of a bridge supports all the elements of the superstructure. Its purpose is to transfer the loads from the superstructure to the foundation, soil, or rock (Rossow, 2007). The main elements of the substructure are the foundations, abutments, wing walls, scour protection, piers, and bearings.

### 2.5.1 Foundations

All foundation designs must meet three requirements (Chen & Duan, 2000):

1) Provide adequate safety against any structural failures
2) Provide adequate bearing capacity of the soil beneath the foundation with a specific factor of safety design

3) Achieve acceptable total or differential settlements under the working load.

There are two types of foundations, shallow and deep. Shallow foundation classifications include spread footings, strap footings, combined footings, and mat or raft footings. These types of foundations provide support entirely from their bases. Deep foundations classifications include piles, shafts, caissons, and anchors. These foundations occupy relatively smaller surface ground areas and can usually take larger loads than shallow foundations (Chen & Duan, 2000).

### 2.5.2 Abutments

Abutments are located at the end of a bridge, to provide end support of the superstructure and to retain the approaching roadway embankment. They are classified according to their locations with respect to the approaching roadway embankment. Common abutment types are full height, stub, open, and integral abutments. Figures 10 through 13 display these different types of abutments. Abutments are typically constructed with one or more of the following materials: plain cement concrete, reinforced concrete, stone masonry, steel, or timber (Rossow, 2007).

Full and stub abutments are used for bridges with shorter spans or if there are issues with the surrounding terrain. Stub abutments may be used to keep abutments away from the roadway or waterway. They also reduce the cost of the substructure, but increase the cost of the superstructure (Rossow, 2007).

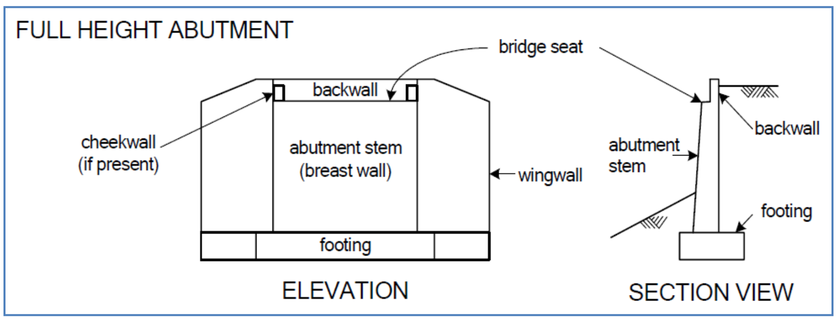


Figure 10 - Full Height Abutment Elevation and Section Views (Barker & Puckett, 1997)

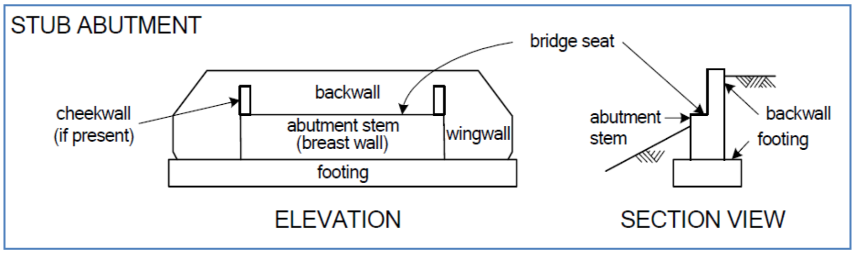


Figure 11 - Stub Abutment Elevation and Section Views (Barker & Puckett, 1997)

Open abutments, also referred to as spill-through abutments, have the approaching roadway embankment extend on a slope between the bridge seat and "through" the supporting columns. The topmost part of the embankment is actually retained by the abutment cap (Rossow, 2007). Open abutments generally have lower cost than full height or stub abutments since most of the massive construction and heavy reinforcement of the substructure is eliminated. An additional advantage to open abutments is that they have the potential to be converted into a pier if more spans need to be added to the bridge over time (Rossow, 2007). However, excessive erosion or scour may occur over the time around the abutment, eventually filling the open space in the abutment with soil and rock (Rossow, 2007). Open abutments are discouraged near stream and river beds as they are susceptible to erosion (Rossow, 2007).

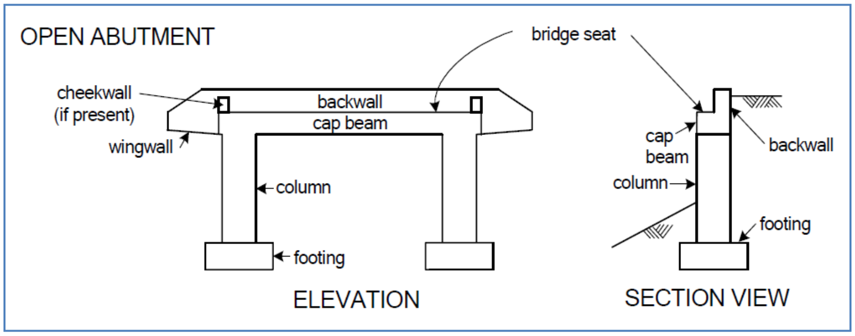


Figure 12 - Open Abutment Elevation and Section Views (Barker & Puckett, 1997)

In some cases bridge design fails because the superstructure and substructure's expansion devices work improperly. Integral abutments are becoming more popular to eliminate this expansion failure (Rossow, 2007). The superstructure and substructure are integral components and act as one unit without the use of an expansion joint. Relative movement is instead accommodated by the pavement joints and approaching roadway slabs (Rossow, 2007). Although an advantage of integral abutments is that they lack bearing devices and joints that require maintenance, they have the disadvantage of frequent cracking due to settling and over compaction of backfill (Rossow, 2007).

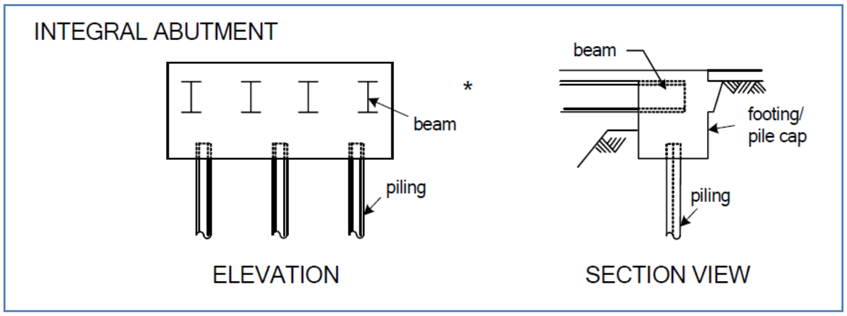


Figure 13 - Integral Abutment Elevation and Section Views (Barker & Puckett, 1997)

Below, Table 3 outlines the primary elements of an abutment and their function.

Table 3 - Abutment Elements (Rossow, 2007)

| Element | Function |
|---|---|
| Bridge Seat | provides a bearing area that supports the superstructure |
| Backwall | retains the approaching roadway; provides support for the approach slab |
| Footing/Pile Cap | transmits weight of the abutment to the supporting soil/rock |
| Cheek Wall | mostly aesthetics; can provide bearings protection for elements |
| Abutment Stem | supports the bridge seat; retains soil |
| Deep Foundations | transmits weight of the abutment to the supporting soil/rock |

### 2.5.3 Wing Walls

Wing walls are rigid gravity or semi-gravity retaining walls thatare adjacent or attached to an abutment. Wing walls may be cast monolithically with the abutment or separately from the abutment with an expansion joint. If the wing wall is cast separately from the abutment it is considered an independent wing wall or retaining wall. Wing walls are commonly used to stabilize an abutment or as a way to direct a stream under a bridge or opening (NYDOT, 2013).

There are a few options for wing wall design. Wing walls can be straight, extending straight back at a 90 degree angle to the abutment, or they can be splayed at any angle to the abutment. Wing walls can also be designed with plain concrete or reinforced concrete as seen in Figure 14. The design is dependent on the forces acting on the wing wall. The forces to be considered are (AASHTO, 2012):

- Lateral earth and water pressures, including any live and dead load surcharge
- The self weight of the abutment/wall
- Loads applied from the bridge superstructure
- Temperature and shrinkage deformation effects
- Earthquake loads as specified in section 3 and elsewhere in these specification

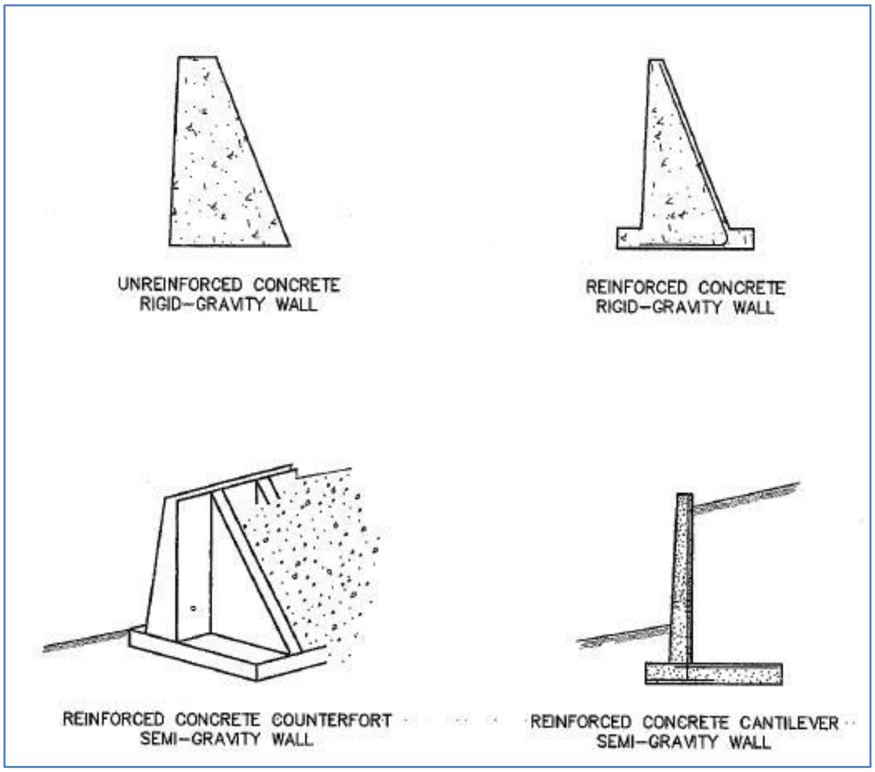


**Figure 14 – Rigid-Gravity and Semi-Gravity Abutments with Reinforced and Unreinforced Concrete (AASHTO, 2012)**

The most common issue to look for in wing walls are cracking, excessive bending or shear stress in the wall, and rotation of the wall structure.

### 2.5.4 Scour Protection

Bridge abutments and piers that are adjacent to streams, flood plains, or water are susceptible to structural failures due to scouring action (Barker & Puckett, 1997). Scour is a site design consideration while designing the substructure of the bridge. One type of scour in a river is due to the lateral shifting of the channel. This is most common at the outside of each bend in a meandering river due to the higher velocity of the stream (Barker & Puckett, 1997). Another type of scour occurs due to the erosion of the river bed during periods of high flow. The maximum depth of scour can be predicted by observation of the river bed during periods of high flow. A third type of scour comes from areas of high velocity in the river due to obstructions such as piers (Barker & Puckett, 1997).

The most common type of scour protection is the use of riprap. Riprap is a sustaining wall of stones or chunks of concrete that are used to prevent slope erosion. Other types of sour protection include gabions, articulated concrete blocks, and grout filled mattresses. Placement of abutments and foundations can also serve as scour protection methods (Barker & Puckett, 1997).

### 2.5.5 Piers

A pier is located between the ends of a bridge. It is designed to support the bridge at intermediate intervals with minimal interference to road or water traffic passing under the bridge. A pier is generally constructed with only one column and supported by one footing (Rossow, 2007).

The existing Quaboag River Bridge is supported by one solid shaft pier near the center of its span. This pier is to be removed to streambed level in the State's proposed replacement bridge design. Since the proposed design is of a single span, there will be no piers in the substructure of the bridge.

### 2.5.6 Bearings

Bridge bearings provide an interface between the superstructure and substructure of a bridge (Rossow, 2007). Their primary functions are to transmit loads from the superstructure to the substructure, allow rotation of the longitudinal bridge girders or beams caused by loading, and permit horizontal movement of the superstructure due to thermal expansion and contraction (Rossow, 2007). The main forces on a bridge bearing are its self-weight, traffic loads, wind loads, and earthquake loads (Chen & Duan, 2000). A bridge bearing consists of four basic elements:

- The sole plate

- The bearing or bearing surface
- The masonry plate
- The anchorage

Table 4 describes each of these elements. All bearings have at least the bearing or bearing surface, though not all bearing have all four of these elements.

**Table 4 - Primary Bearing Elements (Rossow, 2007)**

| Element | Description |
|---|---|
| Sole Plate | Steel plate attached to bottom of girders or beams |
| Bearing or Bearing Surface | Secured to the sole plate; provides transmittal of forces from sole plate to masonry plate |
| Masonry Plate | steel plate attached to the bearing seat; distributes vertical forces to substructure unit |
| Anchorage | bolts that connect the bearing to the superstructure; restrains masonry plate from horizontal movement; can be used to resist transverse movement |

Bearings can be fixed, which restrict translational movements and allow rotational movements, or expansionary, which allow both translational and rotational movements (Chen & Duan, 2000). Different types of bearings can be classified as: sliding plate, roller, rocker, pin and link, elastomeric, or pot bearings (Rossow, 2007). Sliding plate bearings typically provide longitudinal movement on bridges with spans of 15 meters or less (Chen & Duan, 2000). Roller bearings are composed of one or more cylindrical rollers between two parallel steel plates. Singular rollers accommodate both translational and rotational movements whereas multiple rollers work only with translational movements (Chen & Duan, 2000). Rocker bearings come in a variety of designs. Most consist of a pin at its top to allow rotational movement and a curved surface at its bottom to allow translational movement of the bridge (Chen & Duan, 2000). Pin and link bearings are usually found in steel bridges and are used to accommodate rotational movements (Chen & Duan, 2000). Elastomeric bearings consist of both plain and laminated neoprene pads to transmit both types of movement (Chen & Duan, 2000). Lastly, pot bearings comprise of a plain elastomeric disk that is confined in a steel "pot" ring that is able to transmit translational loads (Chen & Duan, 2000). These different types of bearings have comparatively different loading capacities and costs as shown in Table 5. Bearings need to be routinely inspected to ensure they still work for their intended purpose – translational and/or rotational movement (Rossow, 2007).

Neglecting bearing inspection can lead to bridge failures and ethical questions in the bridge's maintenance (Rossow, 2007).

Table 5 - Bearing Type Capacities and Cost (Chen & Duan, 2000)

| **Bearing Type** | **Load** | | **Relative Translation** | | ***Rotational Max (rad.)*** | **Costs** | |
|---|---|---|---|---|---|---|---|
| | *Min (kN)* | *Max (kN)* | *Min (mm)* | *Max (mm)* | | *Initial* | *Maintenance* |
| Sliding Plate | 0 | >10,000 | 25 | >10 | 0 | Low | Moderate |
| Single Roller | 0 | 450 | 25 | >10 | >0.04 | Moderate | High |
| Multiple Roller | 500 | 10,000 | 100 | >10 | >0.04 | High | High |
| Pin and Link | 1,200 | 4,500 | 0 | 0 | >0.04 | Moderate | High |
| Elastomeric | 0 | 450 | 0 | 15 | 0.01 | Low | Low |
| Pot | 1,200 | 10,000 | 0 | 0 | 0.02 | Moderate | High |

## 2.6 Decommissioning of the Bridge

The decommissioning, or the removal of an existing bridge, is a necessity in the case of this project since the Quaboag River Bridge is a bridge replacement project. Things to consider when removing a bridge include type of the demolition, traffic management, and environmental impact to the surrounding area (Gedeon, 1995). One specific consideration, the removal of the existing center pier, is also discussed in this section.

### 2.6.1 Types of Bridge Demolition

The type of demolition a project uses depends on the type of bridge being demolished and the circumstances surrounding it. The most common demolition methods are detailed in Table 6. Each method is unique, with different applications, advantages, and disadvantages (Abudayyeh, Sawhney & Buchanan, 1998).

Table 6 - Comparison of Demolition Methods (Abudayyeh, Sawhney & Buchanan, 1998)

| Method | Applications | Advantages | Disadvantages |
|---|---|---|---|
| Hydraulic Hammers | Demo of bridge decks, piers, slabs, and pavements | High production rate, greater mobility, operable in all weather | Noise, dust, vibrations |
| Whiphammers | Bridge deck removal | High production rate | High energy input |
| Crushers | Full and partial bridge removal | No dust, low noise, no vibrations, great mobility, operable in all weather, rapid and safe cutting of rebar | |
| Water Jet Cutting | Partial removal of deteriorated concrete in bridge decks | Minimum labor, low noise, no dust, high production rate, no vibration, remaining concrete surface irregular allowing good bonding to new concrete | Rebar shadow problems, cost, needs large quantities of water, and disposal of the water that is mixed with debris |
| Blasting and mini blasting | Full and partial bridge removal | Speed, short durations of noise and dust | Dust, noise, vibrations, flying debris, and dangerous |
| Sawing and Cutting | Partial removal of deteriorated concrete in bridge decks | No dust, no vibration, and produces clean edges | Difficulties arise around rebar, cost |
| Mechanical Splitters | Full and partial bridge removal | No vibration, inexpensive, little dust, remaining concrete undamaged, and can be used underwater | Time consuming and requires the use of breakers to expose rebar |
| Chemical Splitters | Full and partial bridge removal | No vibration, no noise, safe, and non-explosive | More expensive than mechanical splitting, requires more time, not operable in cold weather |
| Jackhammers | Partial removal of deteriorated concrete in bridge decks | Easy to use | Slow, noise, dust, and remaining concrete and rebar may be damaged |

A combination of methods can be used in bridge demolition. The types of methods used depend on the financial, site, structural, existing concrete, environmental, worker and public safety, recycling, and disposal limitations of a bridge (Abudayyeh, Sawhney & Buchanan, 1998).

#### *2.6.1.1 Demolition of Center Pier and Abutments*

Since the proposed design of the Quaboag River Bridge consists of a single span, the existing center pier under the bridge needs to be removed. The pier is estimated at 5' in width, and made entirely of concrete. The State proposes removing the pier down to the stream level, which would allow river traffic to move more freely under the bridge (Depaola & Broderick, 2012). Removal of the pier also removes a constriction on the river water flow that can alleviate upstream flooding. The existing abutments are also proposed to be partially removed to provide additional clearance under the bridge.

This will also enlarge the channel and increase its flow capacity. Remnants of the abutments will act as scour protection to the new bridge. The removal of the center pier as well as the partial removal of the abutments may consist of one of the methods listed in Table 6. As both parts of the substructure are in or close to water, environmental and worker safety concerns are of the most importance in the removal of this pier.

### 2.6.2 Traffic Management

Whenever bridge construction is performed, drivers are faced with unexpected traffic conditions. These changes can be hazardous, therefore making traffic management important during bridge construction (Rossow, 2005). Worker and traffic safety, public relations, and cost are the three most important factors associated with traffic management (BDE, 2002).

#### *2.6.2.1 Worker and Traffic Safety*

To ensure worker and traffic safety, traffic control procedures are set by a work-site manager and serve to (Rossow, 2005):

- Warn drivers and pedestrians of any hazards
- Advise traffic of the proper way to travel through construction
- Inform roadway users of the changes in traffic regulations of the surrounding area
- Guide traffic through/around the work-site
- Define areas where traffic should not operate

Additional onsite safety procedures include: short safety meetings each morning prior to the beginning of daily construction, proper use of tools and equipment, and the following of OSHA's safety regulations for construction workers (Rossow, 2005).

#### *2.6.2.2 Public Relations*

During road and bridge construction it is important the public remains informed of the changing traffic conditions of the area (BDE, 2002). The public can remain informed through the local media, the MassDOT website, town meetings and informal hearings, surveys and brochures, as well as contact to the surrounding land owners (BDE, 2002). Keeping the public informed of changing road conditions maintains adequate traffic flow through the roadway under construction since drivers may use alternate routes whenever possible and can avoid driver aggravation due to unexpected delays.

### *2.6.2.3 Cost*

Costs associated with traffic management include the building and removal of a temporary runaround, using a local route detour and structurally upgrading its roadway, paying for an accelerated construction progress, or providing stage construction that, may result in increased unit costs (BDE, 2002). When determining the cost for onsite options, a designer must consider:

- Right of way costs
- Additional construction costs
- Environmental effects
- Vehicular delays
- User costs
- Crash potential

A designer also should consider the effects of unofficial detours when designing a traffic management system. However, in the Quaboag River Bridge project, there is no detour route available, so the traffic manager will not need to consider this factor because if there is no detour route available, the traffic manager will need to be proactive in accommodating periods of peak flow and directions of travel (BDE, 2002).

### *2.6.2.4 Quaboag River Bridge Traffic Management Plan*

Traffic management can be managed in different ways when it comes to bridge replacement projects. A common way to manage traffic during these projects is to construct a temporary bridge along-side the existing bridge. Temporary bridges are designed to be easily assembled and taken apart to ensure vehicular and pedestrian traffic does not interfere with nearby bridge construction. Although useful, this type of traffic management is not possible in the Quaboag River Bridge's case due to tight site conditions and possible environmental hazards to the surrounding wetlands. Instead, the traffic management plan is to build the bridge in two stages. Stage one will maintain vehicular and pedestrian traffic on the western side of the existing structure, while the eastern side is removed and rebuilt. Stage two will shift traffic to the newly constructed section of the bridge, while the remainder of the existing bridge is removed and the western portion of the proposed bridge is constructed (Depaola & Broderick, 2012).

### 2.6.3 Environmental Impact

Before a contract to decommission a bridge is awarded, an evaluation to assess the impact of the concrete removal by the river, stream, or waterway needs to be performed (Gedeon, 1995). The assessment varies from project to project and greatly depends on the size and condition of the waterway and size of the bridge to be decommissioned (Gedeon, 1995). In some cases the aggregate used in the bridge concrete is the same size and type as that found in the waterway, creating little environmental impact on the area (Gedeon, 1995). In other cases, where debris fragments are larger, they are transported and placed in open water to serve as a fish attractor reef (Gedeon, 1995). Overall, recycling is highly encouraged in concrete removal when a bridge is approved for decommissioning.

In the case of the Quaboag Bridge, the shallow streambed and tight site are environmental considerations that the project managers need to consider when demolishing the existing bridge. There could be adverse effects to the fish and water life populations and soil quality if too much bridge debris falls into the stream. To avoid any adverse effects on the environment, a tray like receptacle or container, called a catch basin, can be used to catch falling debris from the bridge deck demolition (LaBounty, 2011). The debris that falls into the catch basin is supported by a crane system and placed into a dump truck for hauling and disposal (LaBounty, 2011). This catch system allows crushed concrete pieces and rebar of the existing bridge to fall conveniently below, without an undesirable and hazardous cleanup process. Catch basins satisfy environmental concerns and would help keep the Quaboag River debris free during decommissioning.

## 2.7 Constructability

"Constructability" is to obtain broader knowledge of building methods early in the design process to have construction of the project run smoothly (Rowings, Harmelink & Buttler, 1991). Bridges are often designed to be of high quality and safety standards but with little attention to the construction methods and details (Rowings, Harmelink & Buttler, 1991). Construction issues that are encountered in the field can be avoided by considering efficient building strategies before site work begins (Rowings, Harmelink & Buttler, 1991). By incorporating construction knowledge into design, costly change orders, budget overruns, scope growth, and litigations can be avoided (Rowings, Harmelink & Buttler, 1991). The areas of construction knowledge that are listed below prove beneficial in the design process of a project (Rowings, Harmelink & Buttler, 1991).

- Availability and cost of materials
- Availability and cost of skilled labor
- Constraints and costs of transportation
- Understanding of various construction methods

If these areas of knowledge are considered, construction of a project is more efficient. This is due to alternatives considered for construction strategies before the project is built, saving time in the field if a problem were to arise. Overall, the development and application of constructability concepts has the potential for creating better designs (Rowings, Harmelink & Buttler, 1991).

## 2.8 Building Information Modeling

*Building information modeling* (BIM) is a digital representation of physical and functional characteristics of a facility. BIM shares knowledge and displays information that allows invested parties to work together in a timely and costly manner. BIM programs are considered to be interoperable. Interoperability is the ability of two or more programs to operate in a reciprocal manner. It is important to the success of the use of BIM because it allows individuals and systems to access, identify, and integrate information across many systems (Salazar, 2012). Examples of BIM programs are *Revit*, *AutoCAD Civil 3D*, *Primavera*, and *Microsoft Project*.

### 2.8.1 Revit

*Revit* is 3D BIM software. It is considered 3D software because it produces design, analysis, and documentation of a building model (Salazar, 2012). Revit concepts are parametric objects, families, and categories. Parametric objects are objects that can change size, material, and graphic look while remaining the same object. Every parametric object is belongs to a family. Revit has model categories and annotation categories. Model categories cover all physical objects found in buildings. The program *Revit Structures* can be used to model bridges and retaining walls by modeling rebar, creating structural models and drawing views, composing them in a drawing, and generating reports. Revit can also be used in collaboration with other programs to create 4D (includes time) or 5D (includes time and cost) construction models.

### 2.8.2 AutoCAD Civil 3D

*AutoCAD Civil 3D* is an object oriented interface. Objects represent survey, design, and construction elements. The objects it uses include points, surfaces, alignments, profiles, pipe networks, and corridor models. *Civil 3D* also allows for easy labeling of points, surfaces, parcels, and alignments (Salazar, 2012). These objects visually show the project and their labels verbalize the information. *Civil 3D* can be used to create site and transportation designs. It can also be used in collaboration with other programs to create a 4D or 5D construction model.

### 2.8.3 eSPAN140

*eSPAN140* is a web-based design tool for short-span steel bridges up to 140 feet. *eSPAN140* features steel fabrication and erection details including rolled beam, plate girder, corrugated steel pipe and structural plate (AISC, 2012).

### 2.8.4 Primavera

*Primavera* can be used with *AutoCAD Civil 3D*, *Revit*, or any other 3D BIM to create a 5D model because it supports the addition of time and cost information to a project. It can create presentations, time schedules, cost control, site development plan, plan diagrams, and underlays (Salazar, 2012).

### 3.8.5 Microsoft Project

*Microsoft Project* can also be used with a 3D BIM to create a 5D model. It integrates time and cost in an organized manner. It is compatible with all other Microsoft software such as *Excel*, *Share Point*, and *Visual Studio*. It features schedules, timelines, cost analysis, visual planners and management resources, single entry mode (Microsoft, 2010).

## 2.9 Evaluation Criteria for Bridge Designs

There are different ways of evaluating engineering designs. Whether that design is a bridge, building, roadway, etc., different alternative designs must be evaluated using set criteria. These criteria can be developed by the engineer, a federal or state government, developer or another party associated with the design. All of these different parties may have different evaluation criteria they wish to exercise, or they may weight one criterion more heavily than others. Often one of the most valuable criteria is the cost of the proposed project, but there are other aspects that may be evaluated as well. Whether it's aesthetics, environmental impact, sustainability or some other criterion, each proposed design will be evaluated and a single design will be chosen that best suits the project's needs.

### 2.9.1 Overview of Evaluation Criteria

As a result of the large array of criteria from which to choose, a system was developed to evaluate the two alternative bridge designs. The following categories were used to conduct an evaluation of the designs:

- Cost
- Life Expectancy (sustainability)
- Environmental impact
- Ethics
- Timeline

The cost of the bridge was an estimate of the total cost for each design. Life expectancy of the two bridges includes the time until the bridge needs to be replaced as well as the time to any major maintenance that will need to be conducted on the bridge. As discussed previously, the environmental impact of each design includes the impact of the methods used to remove the center pier; however, it also includes a measure of the carbon dioxide emissions and energy consumption of the material production for each design. The ethics of each bridge design were evaluated by the magnitude of the design loads for each design. Since both bridge designs would have been designed with the same level of ethics in mind, this project evaluated how close to ethical which type of ridge could be designed. In other words, the capability of designing a more ethical a steel or concrete bridge with group member's background knowledge was considered. Finally, how long the design and construction process of each design would take was evaluated.

This section is meant to give a brief overview of the chosen evaluation criteria used for the two proposed designs. The evaluation process is elaborated further in the next chapter.

### 2.9.2 Life-Cycle Cost Analysis

Since the cost of a construction project is often the determining factor in the evaluation of alternative designs, a more in-depth analysis was conducted for this evaluation criterion. The cost consisted of the overall material cost for the two different designs, the labor and construction costs as well as the maintenance and operation costs of the bridge during its lifetime. A LCCA was conducted to determine the bridge maintenance and repair costs for each design.

LCCA is a process by which the future costs of a proposed design are evaluated and compared to other alternative designs. The steps to conducting this type of cost analysis are as follows (US Department of Transportation, 2002):

- Establish design alternatives
- Determine design activities
- Estimate both agency and user costs
- Determine life-cycle costs for each alternative design
- Analyze results

Once the alternative designs are established, the activities associated with each design must be determined. These activities include both the initial design and construction activities as well as operations and maintenance activities that will need to occur in the future after the project has been completed and been in use (US Department of Transportation, 2002). The cost of both the initial and future activities must then be determined as well as any specific costs experienced by users. Once all of the initial and future costs are determined for each design, they are converted into present-day costs and compared (US Department of Transportation, 2002).

#### *2.9.2.1 NIST BridgeLCC Software*

Software is often used to aid in LCCA. One example of a LCCA program is *BridgeLCC*, produced by the National Institute of Standards and Technology (NIST) (NIST, 2011). *BridgeLCC* was developed to help engineers assess bridge designs that use new-age construction materials. However, the program also serves to compare alternative designs that use conventional materials, which is how it was utilized for the purposes of this project (NIST, 2011). The use of *BridgeLCC* for this project required certain assumptions to perform a LCCA (Table 7).

There are three stages in a bridge's life-cycle considered by *BridgeLCC*:

- Initial Construction
- Operations, Maintenance, & Repair (OM&R)
- Decommissioning

Initial Construction includes construction, easements, right of way, traffic management, and labor costs during the construction of the bridge. OM&R is the bridge's expected service life, 100 years in the case of the Quaboag River Bridge. Costs associated with OM&R include extreme event costs, such as floods, earthquakes, hurricane, or tornado damages, and routine maintenance and repair costs. Additional assumptions needed for OM&R are the inflation and real discount rates as they are considered over time in the *BridgeLCC* program.

**Table 7 - Assumptions Required Per Lifecycle Stage**

| Stage in Bridge Lifecycle | Assumption Needed for *BridgeLCC* |
|---|---|
| **Initial Construction** | Accidents per million vehicle-miles |
| | Average daily traffic (ADT) |
| | Cost per vehicular accident |
| | Driver delay costs |
| | Easements and right of way costs |
| | Length of workzone |
| | Speed limit of Fiskdale Rd |
| | Total man-labor hours |
| | Worker's wages |
| | Workzone speed limit on Fiskdale Rd |
| **Operations Maintenance, & Repair (OM&)** | Routine maintenance and repair costs |
| | Extreme event costs |
| | Inflation rate |
| | Real discount rate |
| | Service life of bridge |
| **Decommissioning** | Decommissioning of bridge cost |

Each cost assumption was assigned a bearer of cost. The bearers in *BridgeLCC* were: Agency, User, and Third Party. For the purposes of this project, the Agency was the Commonwealth of Massachusetts, the User was the Town of Brookfield, and the Third Party was surrounding landowners of the Quaboag River Bridge. Depending on which design of the bridge was being evaluated, each cost was assigned to a bearer to better forecast the life-cycle costs. *BridgeLCC* reported on the incurred costs for each bearer, making it necessary to assign a bearer to each cost for a more accurate LCCA. The cost of the steel and concrete designs were estimated and compared through the use of NIST's *BridgeLCC*. The methodology for how this will be done is discussed in the next chapter.

## 2.10 Conclusions

This chapter discussed the following topics in bridge design: superstructure design, substructure design, decommissioning of existing bridges, manufacturability of designed bridges, BIM, and evaluation criteria for bridge designs applicable to this project. Each topic was summarized for readers to gain a general understanding of elements involved in bridge design.

The next chapter discusses methods of how this MQP began designing a single span bridge. The reader can anticipate seeing procedures of superstructure and substructure design. The next chapter also discusses the importance of five evaluation criterions: life expectancy, environmental impact, ethics, timeline, and cost.

# 3. Methodology

## 3.1 Introduction

The purpose of this methodology is to provide an overview of how this MQP was developed. A spread box concrete superstructure was designed according to specifications from the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications 2012 Manual and based on the Quaboag River Bridge's intended use. Before designing the superstructure, similar steel and concrete bridge designs were evaluated to determine which system would be the "best" choice for this project design. This evaluation was modeled after a process created by George Hunter and Robert Stewart for value-based decision making (2002).

## 3.2 Value-Based Evaluation and Selection of Options

One of the goals of this project was to determine the better choice between a steel and concrete bridge design for the proposed Quaboag River Bridge replacement project. In order to make that determination, a list of criteria was first developed to evaluate different aspects of the designs. This list of criteria was briefly outlined in the background, but this section discusses the development and application of the criteria in more detail. It focuses on how each criterion was used to evaluate the bridges. Once the criteria were established, a hierarchy of the criteria was determined, a baseline was established, alternatives were evaluated, and concepts were compared (Hunter & Stewart 2002).

### 3.2.1 Criteria

As discussed in the background, similar steel and concrete bridge designs were evaluated on the basis of life expectancy, environmental impact, personal ethics, timeline, and cost. Life expectancy, environmental impact, personal ethics, and timeline were considered "evaluation criteria" and are referred to as such throughout the rest of the report. Cost was used as a direct comparison between the two designs, but was also considered design criteria for the purposes of this project. The following paragraphs are dedicated to how the designs were evaluated for each individual criterion.

#### *3.2.1.2 Life Expectancy*

Life expectancy was considered to be how long (in years) each of the bridge designs would last before needing to be replaced with routine inspection and maintenance. The concept is referred to as the "sustainability" of each bridge throughout the project as well. In order to determine the life-cycle of each bridge further research was conducted on past MassDOT bridge projects as well as construction material resources. These resources helped determine the expected service life for different materials and the overall life expectancy of different types of bridges.

### *3.2.1.3 Environmental Impact*

Another evaluated aspect is the environmental impact of each design. For each system, both the environmental impacts of the different construction processes were considered, as well as the impact resulting from the structural materials used in the bridge design.

The environmental impacts of the bridge materials were evaluated based on the amount of carbon dioxide ($CO_2$) emissions produced and the energy consumed through the production of the materials used. The amount of $CO_2$ produced and the energy consumed by construction equipment/processes accounts for a very small percentage of the total emissions and energy consumed (Itoh, Sunuwar, Hirano, Hammad & Nishido, 2000). As a result, construction processes were evaluated through other aspects besides $CO_2$ emissions and energy consumption, such as the environmental impact by other construction emissions like cement production and steel beam painting, and recyclability of the materials (Horvath & Hendrickson, 1998).

### *3.2.1.4 Personal Ethics*

Steel and concrete bridge designs were ethically evaluated based on detail of loading that was required versus the capabilities of the group. This was determined by collaborating past completed civil engineering classes with project-based learning of new knowledge and skills. Since time was limited, the possibility for completing the work was assessed and considered "personal ethics." The capability to design for Strength I, II, III, or IV, Extreme I or II, Service I, II, or III, or for Fatigue I or II was assessed as part of the evaluation process.

### *3.2.1.5 Timeline*

Timeline was the fourth evaluation criteria. Time estimates necessary to complete the construction of were prepared through research of similar bridge projects. The MassDOT Quarterly Report was the primary reference for this information (MassDOT, 2012). A shorter estimated timeline rated better in the final evaluation than a longer timeline.

### *3.2.1.1 Cost*

Cost was considered a criterion for the purposes of this project; however, it was not an evaluation criterion. Once a performance rating was obtained through the use of the evaluation criteria, explained later on in this section, it was divided by the overall cost of the project. The cost of each bridge was estimated through comparison of past and similar bridge projects constructed by the MassDOT. For the steel design three projects of similar dimensions to the bridge design were found on the MassDOT Accelerated Bridge Program Quarterly Update. The three projects are shown in Table 8 below. The costs of these projects were averaged to establish a $3.527 million estimate for the steel design cost. For the concrete design the $3.710 million cost estimation for the actual Quaboag River replacement project was used. Use of these costs in the performance rating matrix is explained later on in this section.

Table 8 - Steel Bridge Design Costs (MassDOT, 2012)

| ***Project*** | ***Structure Length*** | ***Construction Period*** | ***Cost (million $)*** |
|---|---|---|---|
| Maynard Bridge Replacement Project | 115 ft | 1.5 year | 3.826 |
| Northbridge Bridge Replacement Project | 100 ft | 1 year | 3.665 |
| Uxbridge Bridge Replacement | 112 ft | 1.5 year | 3.091 |
| | | ***Average*** | **3.527** |

### 3.2.2 Determination of Hierarchy

After evaluation criteria were developed a hierarchy of the criteria was determined. A criterion's relative importance was developed in relation to the other evaluation criteria. From there, the Performance Criteria Matrix was developed, shown in Table 9. The reasons why certain criteria were chosen over others are presented in Table 10 (Table 45 in Appendix B).

Table 9 - Performance Criteria Matrix

| Criteria | A | B | C | D | TOTAL | Weighting % |
|---|---|---|---|---|---|---|
| **A) Life Expectancy** | A | A | C | a | 2 | 28.6% |
| **B) Environmental Impact** | | B | C | b | 1 | 14.3% |
| **C) Personal Ethics** | | | C | c | 3 | 42.9% |
| **D) Timeline (construction process)** | | | | D | 0 (1) | 14.3% |
| | | | | Total: | 7 | 100.0% |

Table 10 - Criteria Hierarchy Comparison

| Criteria Comparison | Higher Importance | Reasons |
|---|---|---|
| LE vs. EI | LE | - if the bridge lasts for 75+ years, another bridge will not need to be built, resulting in less construction in the future |
| | | - it is in Brookfield's best interests to have a longer lasting bridge because it's cost effective |
| LE vs. PE | PE | - safety is more important than the life expectancy of a bridge |
| | | - if the bridge is ethical, it means that it has a longer life expectancy because it was designed for the loads on the bridge |
| EI vs. PE | PE | - human safety is more important than environmental concerns |
| LE vs. T | LE | - if the road is a well-traveled road, a designer should be more concerned with the bridges life expectancy than construction process so that future construction can be avoided |
| EI vs. T | EI | - it is important to protect the marsh |
| | | - Fiskdale road is not too busy with traffic flow |
| | | - Brookfield may have to spend a lot of money later on cleaning up the marsh if they are not careful during the construction process |
| PE vs. T | PE | - prioritize no one getting hurt on the job site |
| | | - focus on building a safe bridge and not speed through the construction process |
| | | - Fiskdale road is not too busy with traffic flow |

Once the hierarchy of evaluation criteria was developed, which criteria were "more important" was able to be determined (Hunter & Stewart, 2002). The number of votes was tallied and assigned a weighted percentage in each category, as shown above in Table 9. Timeline, although receiving 0 points, was given a value of 1 to be weighted 14.3% because, although it did not dominate in any comparison of the categories, it was still important enough to include in the evaluation criteria and therefore should be assigned a weight (Hunter and Stewart, 2002).

### 3.2.3 Establishment of a Baseline

To establish a baseline, a rating rubric was first designed to create a guideline for the ratings (Table 11) (Table 46 Appendix B). This rubric was primarily used to later compare a steel versus concrete bridge design; however it was also used as a guideline to what a "no build" scenario would produce. Once a rating rubric was established, a "no build" scenario was considered for the Quaboag River Bridge and individually rated by each group member in each evaluation criteria category, as shown in Table 47 in Appendix B. The ratings were averaged, rounding to the nearest whole number, to obtain group ratings for each evaluation criteria of the "no build" scenario. Each criterion's rating was multiplied by its weighted value, determined by the hierarchy matrix, and then the scores for the four criteria were summed together to obtain the "no build" scenario's total performance rating. The ratings of the steel and concrete designs were later compared to the value of the "no build" scenario.

## Table 11 - Rating Rubric

| Category | Exceptional (10-8) | | Neutral (7-4) | | Poor (3-1) | |
|---|---|---|---|---|---|---|
| | ***Steel*** | ***Concrete*** | ***Steel*** | ***Concrete*** | ***Steel*** | ***Concrete*** |
| **Life Expectancy** | *Scale:* | *Scale:* | *Scale:* | *Scale:* | *Scale:* | *Scale:* |
| | - Greater than 75 years | - Greater than 75 years | - 75 years | - 75 years | - Less than 75 years | - Less than 75 years |
| | - paint every 5 years or less | | - paint every 8 years | | - paint every 10 years or more | |
| **Environmental Impact (Average of the following)** | *Scale: (Energy Consumption)* | | *Scale: (Energy Consumption)* | | *Scale: (Energy Consumption)* | |
| | - very little energy is consumed during the manufacuring | | - a moderate amount of fossil fuels are consumed | | - large consumption of fossil fuels | |
| **Energy Consumption** | - very little electricty is needed | | | | - consumes a large quantity and variety of resources | |
| | - hardly consumes any resources | | | | - large amounts of electricty needed for production | |
| **CO2 Emissions** | | | | | | |
| | *Scale: (CO2 Emissions)* | | *Scale: (CO2 Emissions)* | | *Scale: (CO2 Emissions)* | |
| **Other Emissions** | - very little CO2 is produced | | - some CO2 is produced | | - large amounts of CO2 produced | |
| **Recyclability** | *Scale: (Other Emissions)* | | *Scale: (Other Emissions)* | | *Scale: (Other Emissions)* | |
| | - little or no other emissions are produced | | - some pollution from other chemicals occurs | | - large emissions from other types of toxic | |
| | *Scale: (Recyclability)* | | *Scale: (Recyclability)* | | *Scale: (Recyclability)* | |
| | - most or all of the material can be recycled | | - some amount of the material can be recycled | | - no part of the material can be recycled | |
| | - a large amoung of other recycled materials can be used | | - some other recycled materials can be used in | | - other recycled materials cannot be used in | |
| **Personal Ethics** | Scale: | *Scale:* | *Scale:* | *Scale:* | *Scale:* | *Scale:* |
| | - Use of traffic barriers for co | - proper use of shoring/rip rap during construction | - Some traffic protection during construction | - Some Use of shoring/rip rap | - No scaffolding used | - no shoring/rip rap used |
| | - Designed with Strengths I-V, Extreme Events I&II, Service I-IV and Fatigue I&II | - Designed with Strengths I-V, Extreme Events I&II, Service I-IV and Fatigue I&II | - Designed with Stength I, II, & III and Service I & II | - Designed with Stength I, II, & III and Service I & III | -Designed with Strength I | -Designed with Strength I |
| | - include scour protection | - include scour protection | - include some scour protecti | - include some scour protection | - no scour protection considered | - no scour protection considered |
| **Timeline** | *Scale:* | *Scale:* | *Scale:* | *Scale:* | *Scale:* | *Scale:* |
| | - 8 month construction period | - 8 month construction period | - 1 year constructin period | - 1 year construction period | - over 1 year contruction period | - over 1 year constructino period |

### 3.2.4 Evaluating Alternatives

The steel and concrete design alternatives were evaluated in the same manner as the "no build" scenario. Each scenario was rated based on the conducted background research. Each design was rated individually by each group member before averaging the ratings as to ensure no influence on the ratings. The group members' ratings were averaged to the nearest whole number and were compiled into one performance rating matrix (Table 12). To make the next step in the evaluation process simpler, a Performance Measures Forms was made of the performance ratings. This allowed the group to better evaluate which design would be best for the project.

When evaluating the environmental impact criterion, the criterion was broken into four subcategories: energy consumption, $CO_2$ emissions, other emissions, and recyclability. Each subcategory was rated individually and then the four rates were averaged to obtain the overall criterion score. An example of the worksheet is shown in Table 13 (Table 47 in Appendix B).

Table 12 - Performance Rating Matrix

| Criteria | Unit Of Measure | Criteria Weight | Concept | Performance Rating | | | | | | | | | | Total Performance |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | |
| Life Expectancy | Years | 28.6 | No Build | | | | | | | | | | | 0 |
| | | | Steel | | | | | | | | | | | 0 |
| | | | Concrete | | | | | | | | | | | 0 |
| Environmental Impact | Qualitative | 14.3 | No Build | | | | | | | | | | | 0 |
| | | | Steel | | | | | | | | | | | 0 |
| | | | Concrete | | | | | | | | | | | 0 |
| Personal Ethics | Qualitative | 42.9 | No Build | | | | | | | | | | | 0 |
| | | | Steel | | | | | | | | | | | 0 |
| | | | Concrete | | | | | | | | | | | 0 |
| Timeline | Months | 14.3 | No Build | | | | | | | | | | | 0 |
| | | | Steel | | | | | | | | | | | 0 |
| | | | Concrete | | | | | | | | | | | 0 |

| Criteria | Unit Of Measure | low | | | | Rating | | | | | high |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Life Expectancy | Years | | | 50 | | 75 | | | 100 | | |
| Timeline | Months | 16 | 15 | 14 | 13 | 12 | 11 | 10 | 9 | 8 | 7 |

Table 13 - Environmental Impact Subcategories Rating Matrix

| Criteria | Concept | Performance Rating | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Energy Consumption | No Build | | | | | | | | | | |
| | Steel | | | | | | | | | | |
| | Concrete | | | | | | | | | | |
| CO2 Emissions | No Build | | | | | | | | | | |
| | Steel | | | | | | | | | | |
| | Concrete | | | | | | | | | | |
| Other Emissions | No Build | | | | | | | | | | |
| | Steel | | | | | | | | | | |
| | Concrete | | | | | | | | | | |
| Recyclability | No Build | | | | | | | | | | |
| | Steel | | | | | | | | | | |
| | Concrete | | | | | | | | | | |
| | | | | | | | | | | | |
| Average | No Build | | | | | | | | | | |
| | Steel | | | | | | | | | | |
| | Concrete | | | | | | | | | | |

### 3.2.5 Comparing Alternatives

After the performance ratings were obtained for the “no build”, steel, and concrete alternatives the concepts were compared. The performance ratings were entered into a Performance Rating Matrix for easy comparison amongst the scenarios. Each scenario’s total performance rating was then divided by its estimated cost. The results of the comparison are discussed in the Chapter 4.

## 3.3 Superstructure Design

After the comparison of construction alternatives was completed, a prestressed concrete system was identified as the best solution for the Quaboag River Bridge design. The design of the bridge superstructure was completed first by developing a preliminary design, followed by completing a more detailed design, where the aspects of the preliminary design were adjusted to meet certain requirements. These requirements included moment capacities developed by the loading conditions as well as section properties to accommodate prestressing steel in the primary girders. Software was utilized during the final design process and helped facilitate adjustment of the design.

### 3.3.1 Preliminary Design

The preliminary design of the chosen bridge was developed based on AASHTO LRFD Bridge Specifications and the Precast/Prestressed Concrete Institute’s (PCI) Bride Design Manual (2012 & 2003). The initial design of the chosen bridge included:

- Preliminary girder design (cross-sectional dimensions)
- Potential girder spacing
- Sketch including primary superstructure components (girders, deck, sidewalks and parapets)

Some preliminary design dimensions were also obtained from the Massachusetts Department of Transportation. This information was used in order produce a design similar to that of the actual intended design (Depaola & Broderick, 2012).

Developing this initial design provided cross-section drawings that were used to evaluate the structure as a whole and determine the viability of the initial design. In order to develop the final design, information from the preliminary design was utilized in conjunction with dead and live load calculations as well as prestressed steel calculations in order to develop the proposed design. This design process and calculation procedure is discussed in the next section.

### 3.3.2 Development of Final Design

Completion of the bridge design was conducted by determining dead and live load conditions as well as by developing a design for the prestressed steel in terms of the required number of steels strands and the location of the strands within the cross-section of the members. The final design was developed after multiple iterations and adjustments of member sizes, concrete strength, and other properties. As previously indicated, software was utilized for the live load calculations in order to facilitate the iterative process.

#### *3.3.2.1 Dead Load Calculations*

The dead load was calculated according to the AASHTO LRFD Bridge Design Specifications (2012). The loads from the cast-in-place deck and steel parapets were determined and applied to the bridge. The dead load for the concrete deck was calculated using a concrete density of 150 pound per cubic foot (pcf), an overall slab thickness of 7 inches, and a slab width of 47 inches. The slab width refers to the transverse dimension of the concrete slab. For this design, it was assumed the slab would cover the entire transverse width of the bridge. The asphalt dead load was calculated using a density of 140 pcf, a depth of 4.5 inches, and a transverse width of 32 inches. The depth of the asphalt layer was an assumed value based on common asphalt thickness, while the width of the asphalt was based on the assumption that the asphalt layer would only cover the roadway width (NYDOT, 2004). The rest of the concrete deck and asphalt properties (i.e. weight and slab thickness) were assumed based on AASHTO LRFD Bridge Design Specifications. These values were assumed to facilitate the design of the bridge. If these properties were changed, it would affect the dead load calculations for the design, and as a result the design would have to be altered.

The parapet dead load was calculated in three parts: pickets, cross pieces, and posts. The pickets' dead load was calculated using a steel unit weight of 495 pcf, a width and depth of 1.5 inches, and a height of 38.5 inches. The pickets were spaced 0.5825 feet apart. The dead load for the cross pieces was calculated using two different sized bars screwed on top of the pickets, horizontal to the ground. The dead load for the posts was calculated using a W6 x 25 steel size which weighs 25 pounds per foot and a height of 3.625 feet. Once these values were calculated, they were combined to obtain the uniform dead load for the parapets. Hand calculations for the parapets can be viewed in Figure 38 in Appendix C.

The self-weight of the girders was also necessary for the final design of the bridge. However, since the cross-sectional properties of the girders were under investigation for the final design, the dead weight of the girders was calculated using a spreadsheet. A girder width and thickness were selected, and the self-weight of the girders was calculated for varying depths. Similar to the dead load calculation of the concrete deck, a unit weight of 150 pcf was used for the concrete girders. Utilizing the variable dead load of the girders as well as the dead load for the rest of the structural components, a total uniform load and moment could be calculated. Although distribution factors were developed for the live load calculations, for dead load calculations it was assumed that each girder would carry an equal portion of the load. The calculations for the dead load can be seen in Appendix C of this report.

#### *3.3.2.2 Live Load Calculations*

Live load effects were considered per AASHTO LRFD Bridge Design Specifications. The loading consisted of a simulated design truck and a design lane load of 0.64 kips per foot that was distributed along the length of the girders based on calculated distribution factors. The design truck loading can be seen in Figure 15 below, while Table 14 outlines the equations used for calculation of the distribution factors (AASHTO, 2012 & PCI, 2003). The distribution factor for interior beams was taken to be the higher of the two equations presented in Table 14. The distribution factor equation for exterior girders is also displayed in Table 14. The parameters S, d and L are the girder spacing, girder depth and bridge length, respectively. The calculations of the distribution factors can be seen in Appendix C of this report.

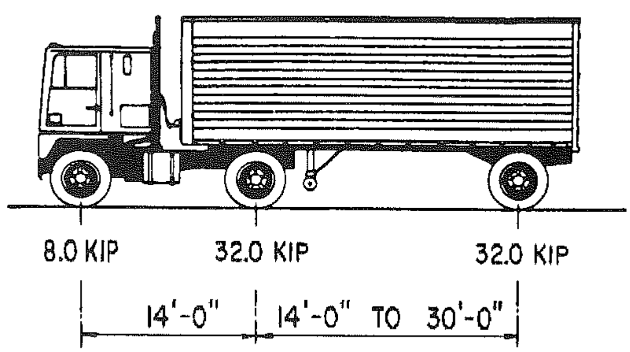


**Figure 15 - Design Truck used for Live Load Conditions (obtained from: AASHTO LRFD Bridge Design Specifications)**

Table 14 - Distribution Factors for Live Load Conditions (AASHTO, 2012 & PCI, 2003)

| | |
|---|---|
| Interior Beam Distribution Factor (One Lane Loaded) | $\left(\frac{S}{3.0}\right)^{0.35}\left(\frac{Sd}{12.0L^2}\right)^{0.25}$ |
| Interior Beam Distribution Factor (Two or More Lanes Loaded) | $\left(\frac{S}{6.3}\right)^{0.6}\left(\frac{Sd}{12.0L^2}\right)^{0.125}$ |
| Exterior Beam Distribution Factor | $\left(\frac{S-d_1}{S}\right)+\left(\frac{S-d_1-6}{S}\right)$ |

The equation in Table 14 for the exterior beam distribution factor was obtained using the lever rule. The parameters in the equation in Table 14 correspond with Figure 16 (PCI, 2003). The lever rule is a way of evaluating the effect of the wheel load on the exterior girder. A transverse section of the bridge is treated as a simply supported beam and the moments are summed about the exterior girder. The point loads displayed in Figure 16 are meant to represent the position of the wheel loads of the design truck.

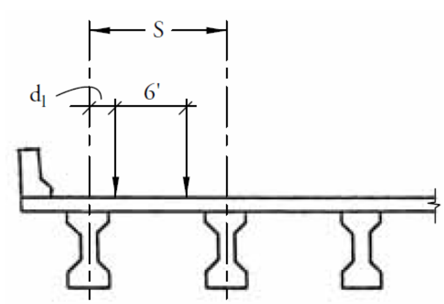


Figure 16 - Representation of the Lever Rule (obtained from: PCI Bridge Design Manual)

The distribution factors discussed above were used in conjunction with the design live loads, and the information was input into structural design software. The use of this software facilitated the solution of the maximum live load moment experienced by the girders. The software also made it possible to easily adjust girder properties and cross-sections during the design iterations.

### *3.3.2.3 RISA-2D Software*

The software used to analyze the live load conditions of the bridge was *RISA-2D*. This software allowed for the modeling of members and the application of the design truck and design lane loads (RISA Technologies, 2013). Using the software, the axle loads of the design truck were moved along the length of a girder in order to determine at which point the girder experienced the maximum moment. The analysis was repeated, varying the distance between the two 32-kip axle loads from 14 to 30 feet. After all of the analyses were completed, the maximum moment experienced by the girder was identified and used in the design of the girder cross-section and prestressing steel. During the design of the prestressing steel, iterations were needed for the girder cross section. As a result of the utilization of *RISA* software the maximum live load moment experienced by the girders could easily be recalculated to coincide with the design of the prestressing steel.

### *3.3.2.4 Concrete Deck Reinforcement Design*

The mild reinforcement steel for the deck was designed by following a one-way slab example in Design of Concrete Structures by Nilson, Darwin, and Dolan. The maximum moments used to calculate for the reinforcement steel were found using *RISA* software. Once the moments were found, the maximum practical reinforcement ratio was calculated with the following equation:

$$\rho_{0.005} = 0.85\beta_1 \left(\frac{f_c'}{f_y}\right)\left(\frac{\epsilon_u}{\epsilon_u + 0.005}\right)$$

Where:

$\rho_{0.005}$= maximum practical reinforced ratio

f'c = strength of concrete

$f_y$ = strength of steel

$\epsilon_u$ = maximum strain (concrete)

$\beta_1$ = a concrete stress block parameter

$\epsilon_u$ always has a value of 0.003 and the $\beta_1$ value was determined using table 3.1 in Design of Concrete Structures. The ρ value calculated was used to determine the minimum required depth in the following equation:

$$d^2 = \frac{M_u}{\emptyset \rho f_y b \left(1 - 0.59\rho \frac{f_y}{f_c'}\right)}$$

Where:

*d* = minimum required depth

$M_u$ = maximum moment

Φ = strength reduction factor

*b* = length (of section)

The value of *b* equals 12 inches to calculate per foot of the deck. After solving for *d*, the minimum required depth was compared to the effective depth which is the assumed thickness of the slab minus cover. If the minimum required depth is larger than the effective depth then that value will be used for the remainder of the calculations which was the case with our deck. The group then assumed a stress block depth of *a* to be used in the following equation to calculate for the area of steel required per foot of the deck.

$$A_s = \frac{M_u}{\emptyset f_y \left(d - \frac{a}{2}\right)}$$

Where:

$A_s$ = minimum required area of reinforced steel per foot

*a* = depth of stress block

$A_s$ was then input into the following equation to calculate the depth of the stress block, *a*.

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

Determination of $A_s$ was an iterative process that proceeded until the calculated value of *a* was sufficiently close to the assumed value of *a*. After the $A_s$ was calculated for both the maximum positive and negative moment, reinforcement steel was selected for the top and bottom of the deck. Reinforcement steel was selected to meet the required area of reinforcement per foot of the deck as opposed to the area of reinforcement required for the entire deck (Nislon, Darwin & Dolan, 2010).

### *3.3.2.5 Prestressed Steel Design*

The method used for the design of the prestressed steel in the bridge girders is shown in the flowchart in Figure 17. This process was conducted through the use of a spreadsheet in order to facilitate the iteration of design parameters. The spreadsheet can be seen in Appendix C of this report.

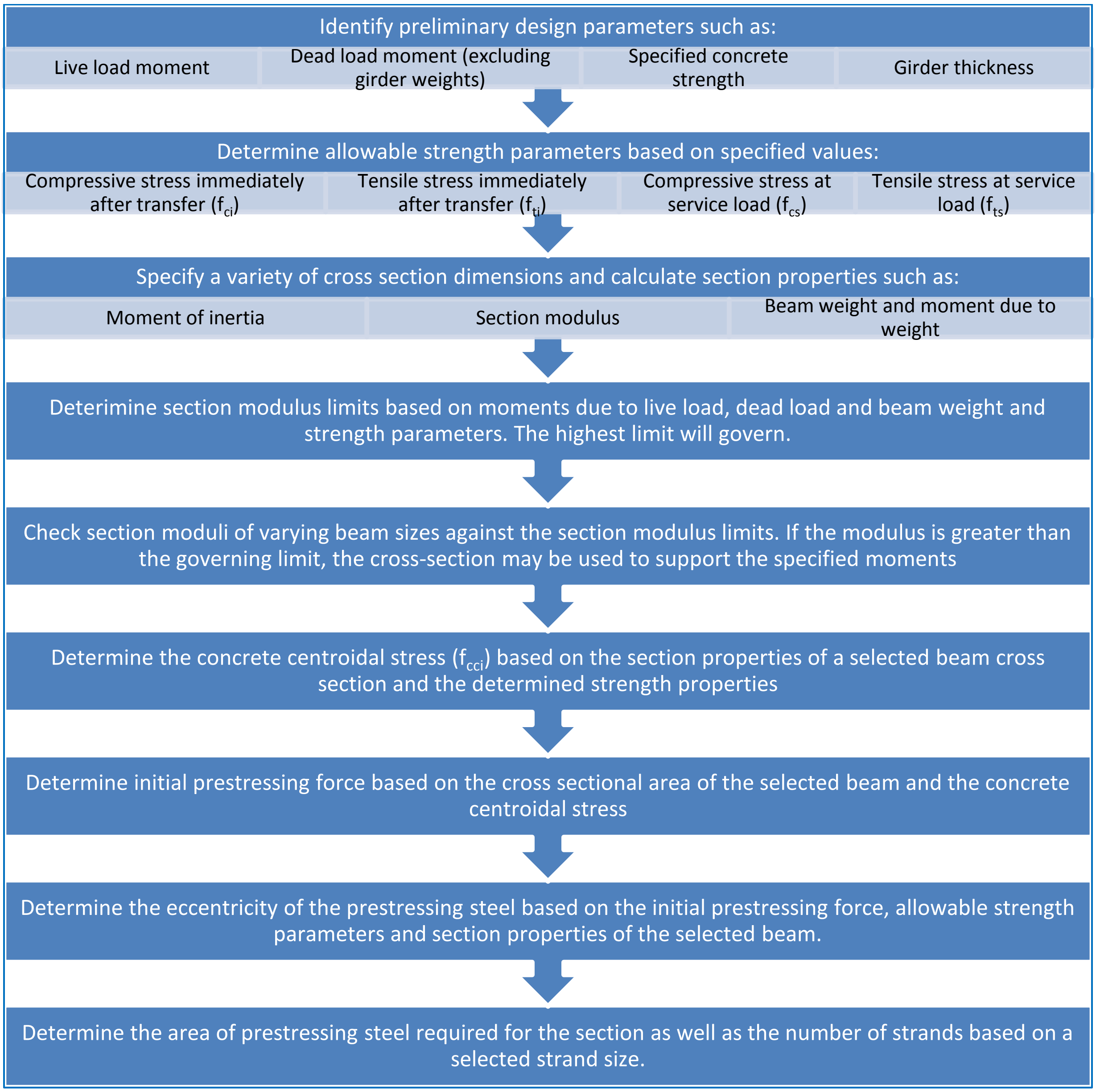


**Figure 17 - Flowchart of Prestressed Steel Design Process (Nilson, 1978)**

The process described in the flowchart above was obtained from Arthur Nilson's textbook

Design of Prestressed Concrete (1978). As described in the flowchart, multiple beam cross sections were investigated for the moments present in the design. In determination of the initial live load moment, the spacing and cross section properties of the girders were based on those of the preliminary design. Once different cross sections were evaluated, the distribution factors were updated to reflect the new spacing requirements and cross section dimensions. Ultimately, a cross section was selected to support the moments developed in both the interior and exterior girders of the bridge.

In addition to the use of Design of Prestressed Concrete by Nilson, specifications from the PCI Bridge Design Manual were also utilized (2003). The PCI manual provided information such as strand sizing, spacing, and other limitations. The use of these specifications allowed for the complete design of the prestressing steel for the spread box girders.

Due to time constraints the design of the substructure was conducted separately from the design of the superstructure.This required assumed design load values for the substructure. After the two designs were completed, the capacity of the substructure was compared to the actual design loads produced by the superstructure. The design process for the substructure is discussed in the next section, while the results of the two designs are presented later in Chapter 4 Results.

## 3.4 Substructure Design Method

The substructure was designed simultaneously with the superstructure. This was mostly due to time constraints but was feasible with extensive use of *Microsoft Excel* spreadsheets. The substructure consisted of four major parts: a foundation, a full height abutment wall, reinforcement in the substructure, and a bearing pad. The full height abutment was referred to as three parts: the backwall, stem wall, and bridge seat. Each of these parts had reinforcement designed for the substructure. The foundation considered two options: a shallow spread footing foundation and a deep pile foundation. The pile cap was designed with reinforcing bars. The reinforcement throughout the entire substructure was designed for flexure and shear. Finally the bearing pad was designed as a fabric reinforced pad that extended the width of each of the spread box concrete girders. The flow chart in Figure 18 shows the design sequence of the substructure.

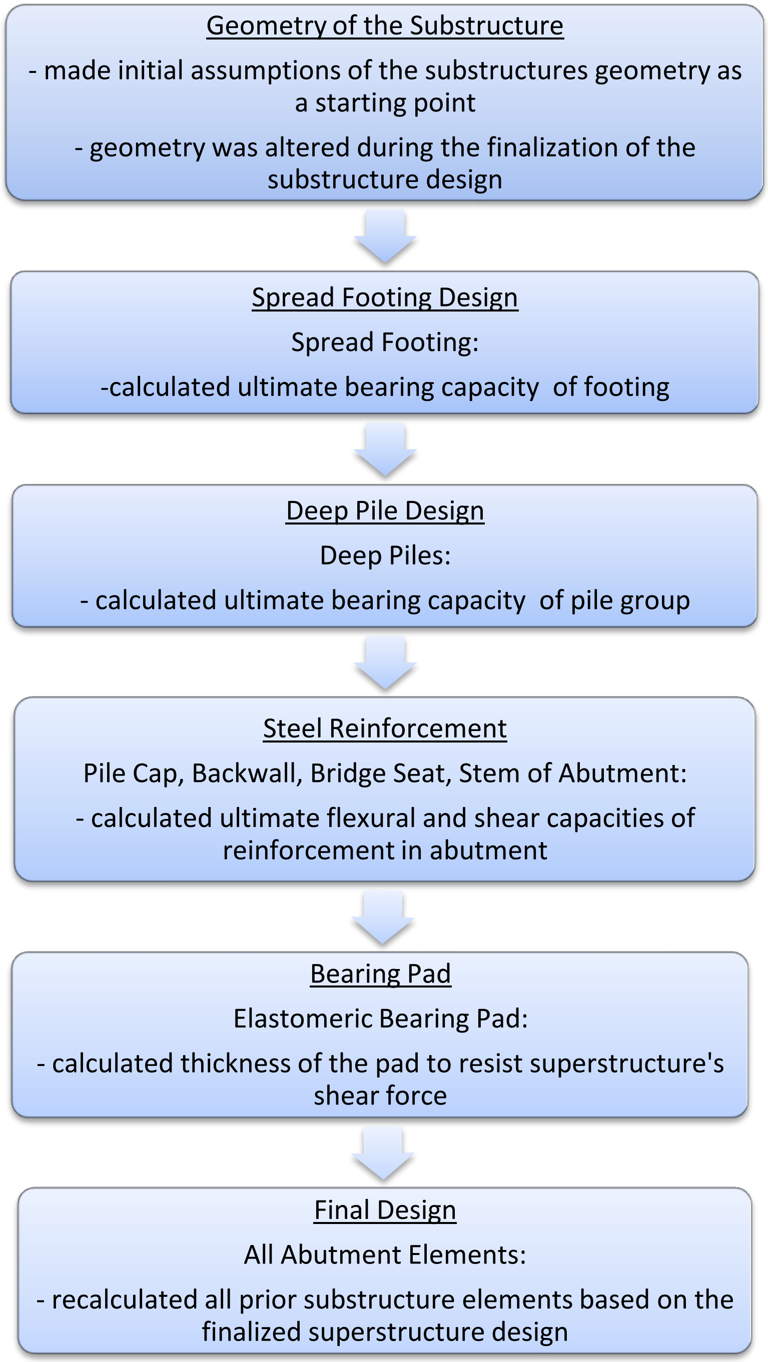


**Figure 18 - Design Sequence of Substructure**

Each part of the substructure design roughly followed the same procedure:

1. Assumptions on site conditions and bridge dimensions were made.
2. Calculated the needed capacities for design of the particular substructure element.

3. Concluded whether the design was adequate for this project based on comparison to similar project or background research.

Once conclusions were made on a substructure design, the process was repeated on the next design alternative. It is important to note that the final design of the bridge's substructure was generated using the finalized live and dead loads for the completed superstructure. This required bearing capacities of the substructure to be recalculated and the entire substructure to be redesigned. Since *Excel* spreadsheets were used throughout the design process, the correct loads were simply input into *Excel* to alter substructure dimensions to obtain the final design.

### 3.4.1 Spread Footing Foundation Design

To support the superstructure of the bridge a conventional spread footing was decided upon as an initial design due to its popularity discovered through background research. This type of foundation also offered simple calculations to determine an approximate ultimate bearing capacity, which was needed for a conclusion about the foundation's adequacy. Three main steps were taken during the spread footing design process:

1. Made site conditions and spread footing dimension assumptions.
2. Calculated for the ultimate bearing capacity of the spread footing.
3. Made conclusions on the ultimate bearing capacity based on background research.

#### *3.4.1.1 Assumptions for Footing Design*

There were two categories of assumptions needed for the simple footing design. The first category was the site conditions at the Quaboag River. These assumptions were based off the supposition that the soil in the Quaboag River area was classified as "sandy soil." This assumption subsequently affected all other soil assumptions, which were directly taken from *Foundation Design Principles and Practices* (Coduto, 2001). Soil assumptions were used in the calculation of the spread footing's ultimate bearing capacity. Table 15 details the assumptions made and their respective symbols.

Table 15 - Soil Assumptions Made for the Footing Substructure Design (Coduto, 2001)

| Soil Assumption | Symbol | Value | Unit |
|---|---|---|---|
| Specific Gravity soil | $G_S$ | 2.7 | - |
| Unit weight water | $\gamma_w$ | 62.4 | pcf |
| Unit weight dry soil | $\gamma_d$ | 120 | pcf |
| unit weight | $\gamma$ | 133 | pcf |
| effective friction method | $\phi'$ | 0 | º |
| bearing capacity factor $N_c$ | $N_c$ | 5.7 | - |
| bearing capacity factor $N_q$ | $N_q$ | 1 | - |
| bearing capacity factor $N_Y$ | $N_Y$ | 0 | - |
| effective cohesion | $c'=c_T=S_u$ | 250 | psf |

The second category of assumptions was the spread footing dimensions. These assumptions were originally based off a similar LFRD substructure example put forth by the Arizona Department of Transportation (ADOT) as a starting point for the design (ADOT, 2012). The assumptions used in the ADOT example were altered to be proportional to the Quaboag Bridge's superstructure length and width. The pertinent footing assumptions used in the spread footing design are shown in Table 16.

Table 16 - Footing Assumptions Made for the Spread Footing Substructure Design (ADOT, 2012)

| Footing Assumptions | Symbol | Value | Unit |
|---|---|---|---|
| footing base | b | 12 | feet |
| vertical distance from ground water | d | 4 | feet |

### 3.4.1.2 Calculations for Footing Design

Calculations for each substructure element were done by *Microsoft Excel.* All assumptions and their respective symbols were recorded on a master assumption sheet (Appendix E). Whenever a calculation needed to use an assumption, the assumption was taken directly from the master sheet. This way if an assumption was to be altered, all calculations using that assumption would automatically change requiring no recalculation. Separate spreadsheets were created for each substructure element so that all required calculations were done in the elements' respective spreadsheet. For example, all spread footing calculations for the bearing capacity were done in one sheet with assumptions linked to the master sheet.

The ultimate bearing capacity of the footing design was calculated using the Terzaghi Method (Coduto, 2001). The soil properties in Table 17 were required in calculating the bearing capacity. These soil properties were calculated using the process outlined in Appendix D. The final equation used to calculate the ultimate bearing capacity of the footing was:

$$Q_{ult.} = c'N_c + \sigma'_{zd}N_q + 0.5\gamma' bN_\gamma$$

Table 17 - Soil Property Calculations (Coduto, 2001)

| Soil Property | Symbol | Value | Unit |
|---|---|---|---|
| unit weight prime | $\gamma'$ | 70.6 | pcf |
| void ratio | e | 1.404 | - |
| pore water pressure | u | 249.6 | psf |
| vertical effective stress at depth d | $\sigma'_{zD}$ | 282.4 | psf |

#### 3.4.1.3 Conclusions on the Footing Design

The ultimate bearing capacity of the spread footing design was determined inadequate based on its comparison to the dimensioned substructures' dead load. Since the bearing capacity of the footing was much less than that of the dead load, it was determined that a conventional spread footing design would be inadequate for the purposes of the project.

### 3.4.2 Deep Pile Design

Piles were chosen as a deep foundation design to the Quaboag Bridge. Through research piles were decided upon due to the depths they can extend into the soil and the high bearing capacities they can carry. The pile axial load capacity, pile group axial load capacity, and the allowable bearing capacity were calculated for a deep pile design. Calculating these capacities was more extensive than that of the footing design, although the process roughly followed the same three main steps:

1. Assumed the same site conditions as for the spread footing design and made nearly identical substructure dimensioning assumptions.
2. Calculated for the pile axial load capacity, pile group axial load capacity, pile allowable bearing capacity, and ultimate pile group bearing capacity of the design.
3. Made conclusions on the allowable bearing capacity based on comparison to the footing design's bearing capacity.

#### *3.4.2.1 Assumptions for Pile Design*

As in the spread footing design, the two categories of assumptions that were need in the deep pile design were soil property and substructure dimension assumptions. Since the calculation of the load and bearing capacities were more extensive than in the footing design, more assumptions were needed to be made. Any site condition assumptions regarding soil properties remained the same. All assumptions were input into the master soil and dimension *Excel* spreadsheet as they were for the design of the footing.

Soil properties were assumed that the Quaboag River is surrounded by sandy soil. Any additional assumptions that were needed to be made for the pile design were taken directly from the Coduto's textbook of Foundation Design Principles and Practices, as in the footing design. A noteworthy assumption was that the site's soil profile was considered to be entirely "sandy soil" which is not the case of the Quaboag Bridge area. However, sandy soil was considered the entirety of the soil profile as a poor scenario for the pile design. The internal friction angle of the soil ($\Phi = 30^o$) resulted in lower bearing capacities than other soil profiles. Table 18 shows the soil assumptions made and their respective symbols.

Table 18 - Soil Assumptions Made for the Deep Pile Substructure Design (Coduto, 2001)

| Soil Assumptions | Symbol | Value | Unit |
|---|---|---|---|
| Specific Gravity soil | $G_S$ | 2.70 | - |
| unit weight of concrete | $\gamma_c$ | 150.00 | pcf |
| Unit weight water | $\gamma_w$ | 62.40 | pcf |
| Unit weight dry soil | $\gamma_d$ | 120.00 | pcf |
| unit weight | $\gamma$ | 133.00 | pcf |
| effective friction method | $\phi'$ | 30.00 | o |
| soil foundation interface friction angle | $\phi_f$ | 21.00 | o |
| Poisson's Ratio | v | 0.30 | - |
| Soil modulus of elasticity | $E_S$ | 7300.00 | psi |
| coefficient of lateral earth pressure | K | 0.50 | - |
| effective cohesion | $c'=c_T=S_u$ | 250.00 | psf |
| inclination of ground surface above the wall | $\beta$ | 0.00 | o |
| factor of safety | F | 3.5 | - |
| over consolidation ratio | OCR | 1 | - |

Many more substructure dimension assumptions were needed for the pile design calculations than for the spread footing design. Dimension assumptions were again based off the ADOT substructure example (ADOT, 2012). Since the ADOT superstructure was of similar size to this project's preliminary superstructure, most dimension assumptions were appropriate for this project. Pile sizes were chosen from common steel pile sections in the United States, and the pile material selected was rough steel for typical design results (Coduto, 2001). The number of rows of piles and number of piles per row was assumed based off background research and symmetry for ease of calculations. All substructure dimensions required and their respective symbols and values are detailed in Table 19. Like the soil assumptions, these were input into the master assumptions spreadsheet in *Excel*.

Table 19 - Pile Cap, Abutment, and Pile Assumptions Made for the Deep Pile Substructure Design

| Dimension Assumptions | Symbol | Value | Unit |
|---|---|---|---|
| pile cap base | b | 12 | ft |
| pile cap height | h | 3 | ft |
| abutment wall height | H | 18 | ft |
| stem wall height (to bridge seat) | $h_s$ | 14 | ft |
| backwall height | $h_b$ | 4 | ft |
| height of top of footing to water surface | $d/z_w$ | 4 | ft |
| stem width | w | 4 | ft |
| bridge seat | s | 2.5 | Ft |
| backwall width | $w_b$ | 1.5 | ft |
| pile cap length | l | 52 | ft |
| length of abutment wall | L | 48 | ft |
| pile length | D | 35 | ft |
| pile diameter | B | 1.167 | ft |
| pile area of face | $A_F$ | 1.07 | ft |
| steel pile area (PP14x1.00) | $A_S$ | 0.283 | $ft^2$ |
| diameter of pile | $d_{pile}$ | 14 | in |
| number of rows of piles | m | 3 | - |
| number of piles per row | n | 13 | - |
| pile spacing from edge | $s_e$ | 2 | ft |
| pile spacing - interior spacing | $s_i$ | 4 | Ft |

### *3.4.2.2 Pile Design Calculations*

The design calculations for the pile axial load capacity, group axial load capacity, allowable bearing capacity, and ultimate group bearing capacity were computed using *Excel* spreadsheets similar to the footing design process. Equations used were taken directly from Coduto's textbook, *Foundation Design Principles and Practice.* The equations and processes used to calculate for the different capacities can be viewed in Appendix D. The final equations used to calculate the pile axial load capacity, group axial load capacity, allowable bearing capacity, and ultimate bearing capacity respectively were:

$$P_a = \frac{P_t + P_s - w_{\frac{pc}{N}}}{F}$$

$$P_{ag} = \eta P_a mn$$

$$Q_{allowable} = q_t + f_s - DL_{pc}$$

$$Q_{ult} = \eta(Q_{allowable})mn$$

Where:

$P_a$ = axial load capacity per pile

$P_{ag}$ = axial group load capacity

$Q_{allowable}$ = allowable single pile bearing capacity

$Q_{ult}$ = ultimate pile group bearing bearing

All other variables are defined in Appendix D.

#### 3.4.1.3 Conclusions on the Pile Design

Comparing the bearing capacities of the deep pile design to the footing design, it was concluded that a pile design would be adequate for the project. Therefore the final design was an alteration of the pile design, discussed later in this chapter.

### 3.4.3 Reinforcement Design

Steel reinforcement in the substructure was designed after the pile design was completed in three parts: reinforcement in the pile cap, under the bridge seat, and at the top of the abutment's backwall. All three parts utilized a precast bridge design example put forth by Modjeski and Masters, Inc. (Modjeski, & Masters, 2003). The design example provided proper equations and processes in designing the reinforcement.

The design processes for the pile cap, bridge seat, and backwall followed three main steps:

1. Assumed site conditions, bridge dimensions, and amount, dimensions, and material properties of the rebar.
2. Calculated the nominal flexural and shear capacities and ultimate flexure and shear experience by the substructure design.

3. Made conclusions based on the nominal flexural and shear capacities and ultimate flexure and shear loads.

#### *3.4.3.1 Assumptions for Reinforcement Design*

Site conditions and bridge dimensions assumptions remained unchanged from the pile design during the reinforcement design. During the first design of the rebar, dead loads and live loads from the superstructure were not yet available. Since the rebar design also used *Excel* spreadsheets dead loads and live loads were temporarily input into *Excel* as 1 kip each. This was for the purposes of including the dead and live load values in calculations, and was not considered as the final calculations of the rebar design for this project. In the final design, the loads from the superstructure were corrected, as to calculate for the actual reinforcement required in the substructure, discussed later in this chapter.

Reinforcing bar assumptions that were made include the size, amount, and strength of the bars used in each of the substructures elements. Table 20 displays the assumptions used and what element of the substructure they were used in.

**Table 20 - Assumptions Used in Design of the Reinforcement in the Substructure**

| Variable Assumption | Symbol | Value | Unit | Substructure Element |
|---|---|---|---|---|
| strength resistance factor | $\phi$ | 0.9 | - | all |
| strength of steel | $f_y$ | 60000 | psi | all |
| strength of concrete | $f'_c$ | 4000 | psi | all |
| shear multiplication factor | $\lambda$ | 2 | - | all |
| unit weight of asphalt | $\gamma_{asphalt}$ | 140 | pcf | backwall |
| effective friction angle of soil | $\phi'$ | 30 | o | backwalll |
| unit weight of dry soil | $\gamma_d$ | 120 | pcf | backwall |
| passive coefficient of lateral earth pressure | $K_p$ | 3.00 | - | backwall |
| reinforcing bars | - | 2 #11 bars | - | pilecap |
| reinforcing bars | - | 1 # 9 bar | - | backwall |
| reinforcing bars | - | 12 # 6 bars | - | bridge seat |

#### *3.4.3.2 Calculations for Reinforcement Design*

Each element of the substructure used the same nominal equations for flexural and shear capacities:

$$M_n = A_s f_y (d_e - {}^{a}\!/_{2})$$

$$V_c = 2\sqrt{f_c'}bd_v$$

Where:

$M_n$ = nominal flexural moment capacity

$A_s$ = area of rebar

$D_e$ = effective depth

$a$ = neutral axis

$V_c$ = nominal shear capacity

$b$ = design width of element

$d_v$ = effective shear depth

Appendix D details the complete calculation processes for the pile cap, bridge seat, and backwall reinforcement.

#### *3.4.3.3 Conclusions on Reinforcement Design*

Since the actual live and dead loads from the superstructure were not available during the initial design of the reinforcement, and a temporary value of 1 kip was used for both loads, only provisional conclusions were drawn. Drawing conclusions at this point in the reinforcement design was for procedural purposes; it created a method for determining whether or not the final design's reinforcement would be adequate.

Once the nominal flexural and shear capacities were calculated, the procedural checks made were:

$$M_{ultimate} < \phi M_n$$

$$V_{ultimate} < \frac{\varphi V_c}{2}$$

If $M_{ultimate}$ and $V_{ultimate}$ were less than the factored flexural and shear capacities, it was decided that the rebar used in the substructure elements was adequate and with stand-in superstructure loads. These equations were deemed appropriate for use in the final design process to properly determine the adequacy of the rebar design in the substructure.

### 3.4.4 Bearing Pad Design

The bearing pad design began once the superstructure live and dead loads were finalized. The design process consisted of four main steps:

1. Determined what type of bearing pad would be used.
2. Assumed bridge dimensions and site conditions.
3. Calculated the minimum thickness of the chosen bearing pad.
4. Made conclusions based on the bearing capacity of the chosen pad.

Since a bearing translates the loads from the superstructure to the substructure, it was important to design the bearing properly. For reassurance in the final design, the bearing capacity was recalculated although no dimensions and loads had changed from its original design.

#### *3.4.4.1 Types of Bearing Pads*

As there are several different types of bridge bearings, the first step was to choose a type for application. The choice was made based off prior background research. Table 5, duplicated below in Table 21, detailed bridge bearings' capacities, translations, rotational maximums, and costs. An elastomeric bearing pad was chosen to as it has the lowest costs compared to the other bearing types.

**Table 21 - Bearing Type Capacities and Cost (Chen & Duan, 2000)**

| **Bearing Type** | **Load** | | **Relative Translation** | | ***Rotational Max (rad.)*** | **Costs** | |
|---|---|---|---|---|---|---|---|
| | *Min (kips)* | *Max (kips)* | *Min (in)* | *Max (in)* | | *Initial* | *Maintenance* |
| Sliding Plate | 0 | >2,250 | 1 | >0.40 | 0 | Low | Moderate |
| Single Roller | 0 | 100 | 1 | >0.40 | >0.04 | Moderate | High |
| Multiple Roller | 115 | 2,250 | 4 | >0.40 | >0.04 | High | High |
| Pin and Link | 270 | 1,000 | 0 | 0 | >0.04 | Moderate | High |
| Elastomeric | 0 | 100 | 0 | 0.60 | 0.01 | Low | Low |
| Pot | 270 | 2,250 | 0 | 0 | 0.02 | Moderate | High |

### 3.4.4.2 Assumptions for Bearing Pad Design

Table 22 shows the assumptions required to design the elastomeric bearing pads. All assumptions were taken from a design example about bridge bearings (Mellon & McKee, 1994). Fabric reinforced pads were selected to design for due to a simpler process of calculations.

**Table 22 - Bearing Pad Assumptions**

| Assumptions | Symbol | Value | Unit |
|---|---|---|---|
| Temperature movement & concrete shortening coefficient | $\alpha_T$ | 0.000008 | - |
| Moderate temperature zone rise and fall | T | 50 | o |
| Maximum pressure the bearing pad can withstand | $P_{max\ load}$ | 0.8 | ksi |
| Modulus of rigidity of concrete | $G_c$ | 3000.00 | psi |
| Assumed length of pad | $L_a$ | 16 | in |

### 3.4.4.3 Calculations for Bearing Pad Design

Since the dead and live loads from the superstructure were finalized, calculations for the elastomeric bearing pad were completed using an *Excel* spreadsheet and then checked by hand calculations. The design process of determining the required thickness of the pad can be found in Appendix D.

The final equation in determining the adequacy of the bearing pad thickness was:

$$V_D = \frac{G_c L_a w_{bf} \Delta_T}{t_f} > \frac{DL_{super}}{6} = V_{max}$$

Where $V_D$ was the design shear capacity of the pad with the final thickness accounted for, and $V_{max}$ was the maximum shear that the bearing pad needed to support.

### 3.4.4.4 Conclusions on Bearing Pad Design

Once the bearing pad's shear design capacity exceeded the maximum shear, the pad's dimensions were deemed adequate for the bridge design. If the pad was inadequate, the pad's thickness was increased, and the equation process in Appendix D reworked using the new pad thickness.

### 3.4.5 Final Design of Substructure

Once the superstructure was finalized, an iterative process was completed through use of the *Excel* spreadsheets to obtain the final deep pile substructure design's appropriate dimensions and reinforcement.  The flowchart in Figure 19 shows the sequence of altering and finalizing the substructure design.

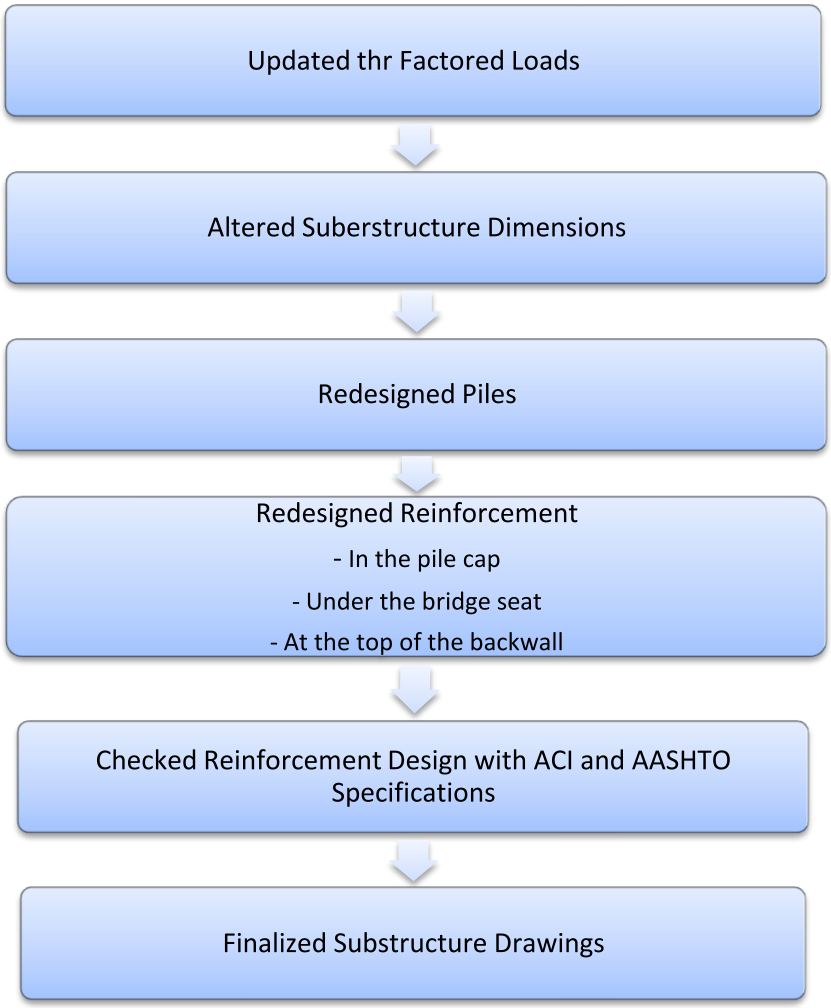


**Figure 19 - Design Sequence of Substructure's Final Deep Pile Substructure Design**

#### *3.4.5.1 Calculations for Factored Load*

The superstructure's bearing loads were taken directly from the superstructure's unfactored dead and live load calculations discussed previously in Chapter 3.3.2.  Substructure loads were also a consideration in the pile and pile cap reinforcing design.  Calculations with ACI considerations required all loads to be factored using the standard equation:

$$\omega = 1.2DL + 1.6LL$$

Where:

$\omega$ = factored load

DL = dead load

LL = live load

Table 23 displays the finalized load values that substructure elements used in their final respective designs.

Table 23 - Finalized Superstructure Loads Required for Substructure Element Calculations

| Provided Loads | Symbol | Value | Unit | Element |
|---|---|---|---|---|
| DL of superstructure | $DL_{super}$ | 1683 | k | piles/pilecap |
| DL of substructure | $DL_{sub}$ | 392 | k | piles/pilecap |
| Maximum factored girder DL | $P_u$ | 221 | k | bridge seat |
| Maximum factored girder LL | $\omega_u$ | 1.58 | k | bridge seat |
| Maximum girder DL + LL | $P_{Str\text{-}I}$ | 216 | k | backwall |
| Maximum factored girder DL + LL | $P_{Str\text{-}II}$ | 345 | k | backwall |

### *3.4.5.2 Alteration of Substructure Dimensions*

The substructure dimensions were first altered based on ethical concerns. Since the foundation design was based off an ADOT bridge design project for simplicity of design and calculation, the abutment dimensions did not satisfy the Massachusetts 50-year flood plain height requirement, a major ethical consideration. After revisiting prior background research regarding dimension requirements of the bridge, the backwall and bridge seat heights were changed to meet MassDOT requirements.

The original deep pile substructure's dimensions were also designed conservatively to ensure that it would sustain the dead and live loads from the superstructure. During redesign, the width and length of the backwall, abutment wall, and bridge seat were altered to be less than the original deep pile design's dimensions. This created a more economical substructure design because a smaller design would incur lesser material costs for the project. Also, smaller dimensions required less concrete needed for the substructure's design, resulting in less dead load from the substructure to be carried by the pile cap and piles. Before the dimensions were completely finalized, checks of the steel reinforcement were made, explained later in Section 3.4.5.4, to ensure the substructure would be able to withstand the superstructure's factored loads.

### *3.4.5.3 Pile Redesign*

The original pile design was considered adequate once the completed superstructure factored live and dead loads were calculated. Since the capacity was so much greater than the loading, the pile design was deemed an overdesign and required change to be more economical. It is important to note that a factor of safety of 3.5 was used during the pile group design. This allowed for no ethical concerns to arise when the bearing capacity of the piles nearly equaled the factored dead and live loads acting on the piles.

For economic considerations, it was desired to use fewer and shallower piles in the substructure's design. Fewer and shallower piles meant smaller construction fees that would be incurred by the design. The bearing capacity of the piles was designed to be just greater than the factored loads acting on them since no ethical concerns would arise. To obtain this proximity in bearing capacity and load, the individual piles were slightly enlarged in diameter and thickness, the depth of the pile group was significantly diminished, and the number of piles per pile group was decreased by nearly 4/5 in comparison to the original pile design.

### *3.4.5.4 Reinforcement Redesign*

The redesign of the reinforcement was simple with the use of the *Excel* spreadsheets used in the original pile design. Reinforcement in the pile cap, bridge seat, and backwall was altered based on the factored loads presented in Table 23 in Section 3.4.5.1. The amount of rebar was determined through the checks shown in Appendix D. Once the design capacities satisfactorily exceeded the factored loading on the substructure, the reinforcement was considered adequate.

For economic considerations, the bearing and shear capacity of the reinforced substructure was designed to be as close to the factored loads as possible. Since ACI equations and AASHTO requirements were considered throughout the reinforcement design, no other ethical considerations were required.

### *3.4.5.5 Reinforcement Design Verification*

After checking ACI and AASHTO requirements, it was found that the reinforcement at the top of the backwall, bridge seat, and the pile cap met all minimum requirements and no redesign was required in those areas. For reinforcement along the stem wall, ACI and ASSHTO required the following ratio of rebar to concrete area:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} = 0.0018bh$$

Where:

$A_s$ = area of reinforcing steel

$b$ = width of stem

$h$ = height of stem wall

$f_y$ = strength of steel (assumed 60 ksi)

The reinforcement design was considered finalized since it satisfied this condition, as well as the minimum spacing requirements set by ACI and AASHTO.

#### *3.4.5.6 Drawing Finalization*

The last step in design of the substructure began once the dimensions, piles, and reinforcement of the substructure was all considered finalized. Rough drawings were completed throughout the entire substructure design process for visual purposes of progress. The final pile foundation substructure design was hand drafted in multiple elevation and plan views with a ½" and ¼" scale depending on the section view.

### 3.4.6 Conclusion

Methods in developing the substructure were complex and involved a lengthy process with many equations. The three major steps in each element of the substructure were:

1. Made site and dimension assumptions for particular elements.
2. Calculated necessary capacities of the particular elements.
3. Made conclusions based on acting loads and design capacities.

The initial design of the substructure was investigated simultaneously with the design of the superstructure due to extensive use of *Excel* spreadsheets. These spreadsheets made the substructure's final design process quick and simple in comparison to alternative hand calculations.

## 3.5 *BridgeLCC* Life-Cycle Cost Analysis Method

A LCCA was completed using the National Institute of Standards and Technology (NIST) program, *BridgeLCC* 2.0, which is a LCCA program that was explained earlier in the Background Chapter of this report. This program determined the cost effectiveness of a concrete spread box girder design in relation to an alternative steel girder design. Two notable elements of *BridgeLCC* were making cost assumptions and selecting the bearer for each cost. These two elements were essential in the program creating accurate LCCAs for the concrete and steel designs.

### 3.5.1 Assumptions for Life-Cycle Cost Analysis

Assumptions were needed to complete a LCCA of the two designs. It was important to use reliable sources for certain assumptions such as inflation and real discount rates. Examples of assumptions and their respective sources are shown in Table 24.

Table 24 - Assumptions and Respective Sources Utilized in BridgeLCC

| Assumptions | Source |
|---|---|
| 50-Year Flood Event | Estimation based on past Accelerated Bridge Projects (ABPs) |
| Accidents per Million Vehicle-Miles | MassDOT.gov |
| ADT by 2113 | Approximation based on prior knowledge of traffic flow |
| Agency Cost | MassDOT.gov |
| Average Daily Traffic (ADT) | MassDOT.gov |
| Concrete Bridge Final Disposal Cost | Estimation based on background research |
| Cost per Accident | Approximation based on MassDOT accident report website |
| Cost to Rehabilitate Steel Bridge Design | Estimation based on past ABPs |
| Disposal Cost | Approximation based on background research |
| Driver Delay Cost | *Traffic and Highway Engineering* N. Garber & L. Hoel |
| Easements and Right of Way Costs | Telegram and Gazette (Telegram and Gazette 2012) |
| Inflation | Bureau of Economic Analysis |
| Length of Workzone | Google Maps |
| Operations, Maintenance, and Repair Cost | Estimation based on past ABPs |
| Real Discount Rate | Office of Management and Budget |
| Service Life | NIST *BridgeLCC* |
| Speed of Road | Google Maps |
| Total Man-Labor Hours | Approximation based on prior knowledge of construction management |
| User Cost | MassDOT.gov |
| Worker's Wages | ENR.construction.com |
| Workzone Accidents per Million Vehicle-Miles | MassDOT.gov |
| Workzone Speed of Road | MassDOT.gov |

Any assumptions that have a source called “Approximation” were not vital to the *BridgeLCC* analysis. These assumptions were entered into *BridgeLCC* as approximations by the group to create a more realistic LCCA of the two designs. Any assumptions labeled “Estimations” did result in note-worthy changes in the LCCAs. These estimations were best-guess values that were made to gather results for LCCA of both of the designs. The strategy of making approximations and estimations was:

1. Determining whether the assumption would be used for both of the designs or one design by prior background knowledge.
2. Determining if the assumption would be a low cost or high cost based on prior background knowledge.
3. Researching possible assumption values on government websites.
4. Researching possible assumption values by looking at past Massachusetts Accelerated Bridge Projects incurred costs by bearer.
5. Making a best-guess approximation or estimation with information collected through steps 1-4.

The next chapter discusses the results of the LCCA and roles the approximations and estimations played in the program.

### 3.5.2 Costs by Bearer

In the *BridgeLCC* program there were three types of bearers: Agency, User, or Third Party. These bearers represented the Commonwealth of Massachusetts, the Town of Brookfield, or the surrounding Landowners of the Quaboag River Bridge, respectively. Each cost element was assigned to a certain bearer. It was important to assign each bearer the appropriate cost as Massachusetts will only fund a bridge replacement project if it has a life expectancy of at least 75 years (Telegram & Gazette, 2012). The concrete girder bridge design in this project would have a 100-year life expectancy while the steel I-beam design would have a 25 year life expectancy before major renovations would need to be made. Therefore, the majority of costs of the concrete girder design would be borne by Massachusetts while Brookfield would incur the majority of costs of the steel design. This was considered while using *BridgeLCC*. Table 25 below shows costs required by the program and the bearer of each cost. The different costs incurred by the Agency or User are discussed in the next chapter.

Table 25 - Cost by Bearer of Concrete and Steel Designs

| Cost | Design | |
|---|---|---|
| | *Concrete* | *Steel* |
| 25 Year Rehabilitation | N.A. | Brookfield |
| 50-Year Flood | N.A. | Brookfield |
| Construction | Massachusetts | Brookfield |
| Decommissioning Disposal | Massachusetts | Brookfield |
| Easements & Right of Way | Landowners | Landowners |
| Final Disposal | Massachusetts | Brookfield |
| Operations, Maintenance, and Repair | Brookfield | Brookfield |

# 4. Results

## 4.1 Introduction

This section presents the results obtained for the replacement design of the Quaboag River Bridge. The evaluation of solution options and the superstructure design, substructure design and LCCA for the selected option were all completed using the processes described in the Methodology Chapter.

## 4.2 Value-Based Evaluation and Selection of Options

A completed performance rating matrix and environmental impact subcategories rating matrix are displayed in Tables 26 and 27, respectively. These tables were completed individually by each group member in order to develop the performance rating matrix summary of results. The summary results for the performance rating matrix are shown below in Table 28. Without cost considered, a steel bridge would be 50% better to design and hypothetically construct than leaving the existing Quaboag Bridge untouched. Comparatively a concrete bridge would be 88% better to design and build than the "no build" design.

Table 26 - Performance Rating Matrix Results

| Criteria | Unit Of Measure | Criteria Weight | Concept | Performance Rating 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | Total Performance |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Life Expectancy | Years | 28.6 | No Build | | 2 | | | | | | | | | 57.2 |
| | | | Steel | | | | | | | 7 | | | | 200.2 |
| | | | Concrete | | | | | | | 7 | | | | 200.2 |
| Environmental Impact | Qualitative | 14.3 | No Build | | | | | | | | | 9 | | 128.7 |
| | | | Steel | | | | 4 | | | | | | | 57.2 |
| | | | Concrete | | | | | 5 | | | | | | 71.5 |
| Personal Ethics | Qualitative | 42.9 | No Build | 1 | | | | | | | | | | 42.9 |
| | | | Steel | | | | 4 | | | | | | | 171.6 |
| | | | Concrete | | | | | | | 7 | | | | 300.3 |
| Timeline | Months | 14.3 | No Build | | | | | | | | | | 10 | 143 |
| | | | Steel | | | | | | | | | 9 | | 128.7 |
| | | | Concrete | | | | | | | | | 9 | | 128.7 |

| Criteria | Unit Of Measure | low 1 | 2 | 3 | 4 | Rating 5 | 6 | 7 | 8 | 9 | high 10 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Life Expectancy | Years | | | 50 | | 75 | | | 100 | | |
| Timeline | Months | 16 | 15 | 14 | 13 | 12 | 11 | 10 | 9 | 8 | 7 |

Table 27 - Environmental Impact Subcategories Rating Matrix Results

| Criteria | Concept | Performance Rating | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Energy Consumption | No Build | | | | | | | | | | 10 |
| | Steel | | 2 | | | | | | | | |
| | Concrete | | | | | | 6 | | | | |
| CO2 Emissions | No Build | | | | | | | | | 9 | |
| | Steel | | 2 | | | | | | | | |
| | Concrete | | | | | | 6 | | | | |
| Other Emissions | No Build | | | | | | | | | | 10 |
| | Steel | | | 3 | | | | | | | |
| | Concrete | | | | 4 | | | | | | |
| Recyclability | No Build | | | | | | | | 8 | | |
| | Steel | | | | | | | | | 9 | |
| | Concrete | | | | 4 | | | | | | |

| | | |
|---|---|---|
| **Average** | No Build | 9 |
| | Steel | 4 |
| | Concrete | 5 |

Table 28 - Performance Rating Matrix Summary of Results

| CRITERIA | Performance | No Build | Steel | Concrete |
|---|---|---|---|---|
| **Life Expectancy** | Measure | Years | Years | Years |
| | Rating | 2 | 7 | 7 |
| | Weight | 28.6 | 28.6 | 28.6 |
| | Contribution | 57.2 | 200.2 | 200.2 |
| **Environmental Impact** | Measure | Qualitative | Qualitative | Qualitative |
| | Rating | 9 | 4 | 5 |
| | Weight | 14.3 | 14.3 | 14.3 |
| | Contribution | 128.7 | 57.2 | 71.5 |
| **Personal Ethics** | Measure | Qualitative | Qualitative | Qualitative |
| | Rating | 1 | 4 | 7 |
| | Weight | 42.9 | 42.9 | 42.9 |
| | Contribution | 42.9 | 171.6 | 300.3 |
| **Timeline** | Measure | Months | Months | Months |
| | Rating | 10 | 9 | 9 |
| | Weight | 14.3 | 14.3 | 14.3 |
| | Contribution | 143 | 128.7 | 128.7 |
| ***Total Performance:*** | | **371.8** | **557.7** | **700.7** |
| Net change in Performance: **(no build to steel/concrete %)** | | **0%** | **50%** | **88%** |
| `Net Change in Performance **(steel to concrete %)** | | **-** | **0%** | **20%** |

The value-based selection process is shown below in Table 29. Here a cost factor is included in the analysis. By dividing total performance by cost (in $ millions) the value index was obtained. The value index shows that a concrete bridge design would be 19% better to design than a steel bridge design. This allowed the group to begin designing the bridge's superstructure design with concrete spread box beam girders

**Table 29 – Value-Based Performance Rating Matrix**

| Concept | Total Performance | Total Cost (millions of $) | Value Index (P/C) | % Value Improvement |
|---|---|---|---|---|
| No Build | 371.8 | - | - | |
| Steel | 557.7 | 3.527481 | 158.10 | |
| Concrete | 700.7 | 3.710339 | 188.85 | **19%** |

(Concrete is a better choice than steel by 19%)

## 4.3 Superstructure Design

The procedures used to develop the superstructure design of the concrete spread box girder bridge were discussed in the Methodology Chapter of this report. This section presents the results of the preliminary and final design of the superstructure. The preliminary design was developed using estimated girder sizes and cross section properties. These estimated values were then adjusted and updated to support the loads applied to the bridge and complete a final design.

### 4.3.1 Preliminary Design

As discussed previously, the preliminary design of the chosen concrete spread box girder bridge was determined through the use of AASHTO LRFD Bridge Design Specifications and the PCI Bridge Design Manual (2012 & 2003). Table 30 summarizes the preliminary design specifications developed for the bridge replacement in Brookfield, MA.

Table 30 - Preliminary Design Specifications

| Quantity | Value | Source Referenced |
|---|---|---|
| Girder Type | Spread Box Beam | FWHA 2006, AASHTO 2012 |
| Cross Section Type | Rectangular | AASHTO 2012 (Cross-Section b) |
| Cross Section Width | 48 in | PCI 2003 |
| Cross Section Height | 33 in | PCI 2003 |
| Web Wall Thickness | 5 in | PCI 2003 |
| Cross Section Moment of Inertia | 105219 $in^4$ | N/A |
| Girder Spacing (Clear Width) | 4 ft | PCI 2003 |
| Girder Spacing (On Center) | 8 ft | PCI 2003 |
| Total Number of Girders | 6 | N/A |
| CIP Concrete Deck Depth | 7 in | AASHTO 2012 |
| Depth of Asphalt Wearing Surface | 4.5 in | NYSDOT 2004 |
| Overhang Length | 1.5 ft | N/A |

The preliminary design of the bridge can be viewed in Figures 21 and 22. Figure 21 shows the cross section of the superstructure while Figure 22 shows the cross section of the preliminary box beam. This preliminary design was based on assumed dimensions obtained from the PCI Bridge Design Manual (2003). As discussed in the Methodology, this preliminary design was used to determine initial dead load and live load conditions.

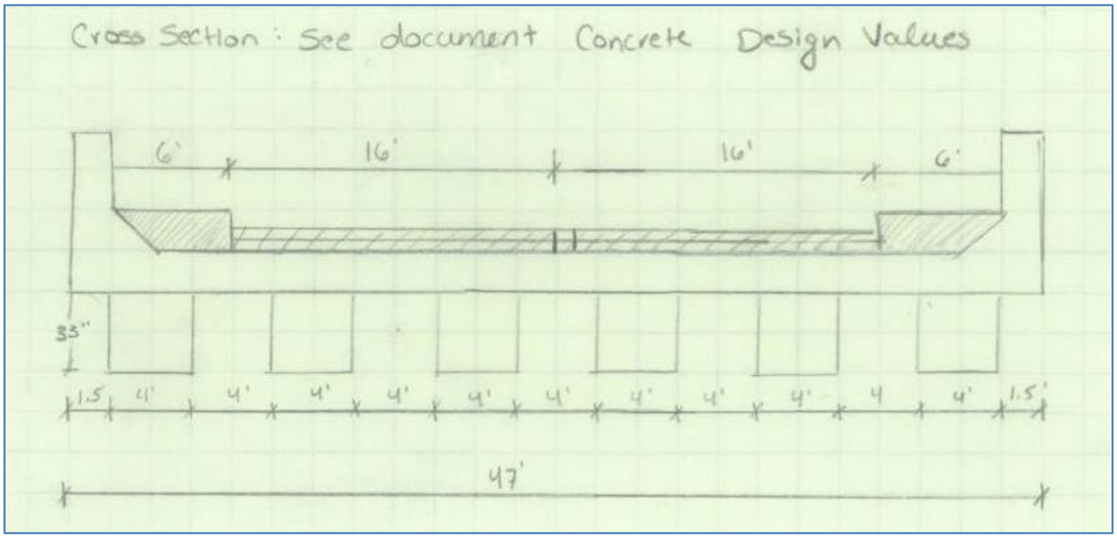


Figure 21 - Preliminary Bridge Design

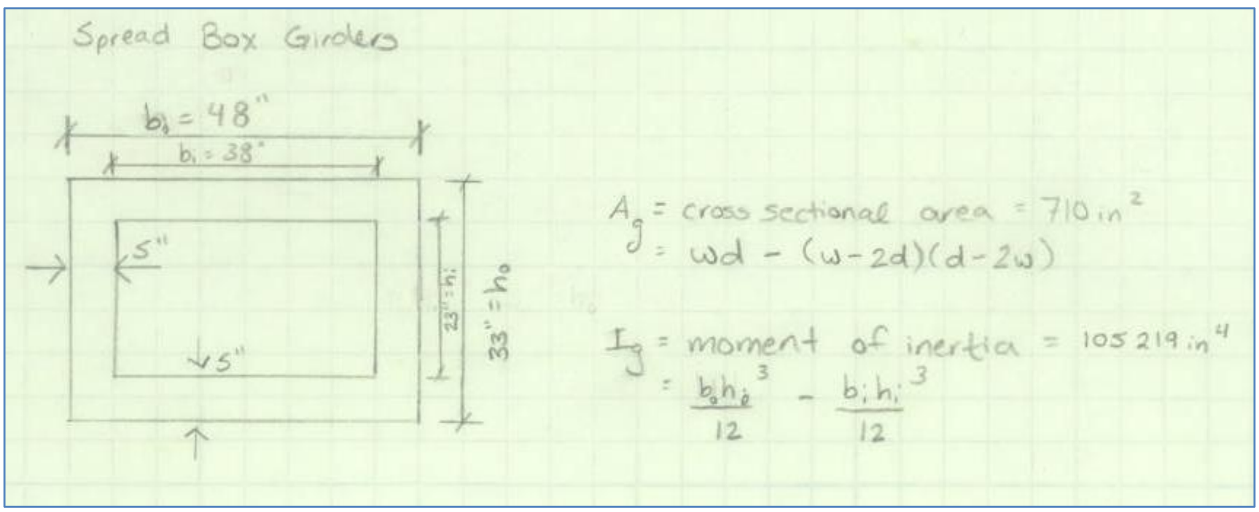


Figure 20 - Preliminary Spread Box Concrete Girders

### 4.3.2 Final Design

An iterative process was conducted to determine the superstructure's final design. The dead load and live load moments experienced by the superstructure were calculated from the preliminary design using the process outlined in the Methodology Chapter. The process involved a mix of spreadsheet tools and *RISA-2D* software. The preliminary design was deemed inadequate because it could not support the applied moments. As a result, the following steps were taken to increase the capacity of the section:

- Increase concrete compressive strength
- Increase the width and depth of the spread box beams
- Increase the web wall and flange thickness of the spread box beams

Although the increased size of the girders resulted in a larger dead load moment, the added capacity of the members eventually resulted in an adequate design.

#### *4.3.2.1 Dead Load Calculations*

As previously discussed the dead load of the concrete deck, asphalt wearing surface, and railings were calculated and distributed to the bridge girders. Table 31 displays the various dead loads calculated for the design. The girder weight was calculated during the iterative design process for multiple girder cross-sections. The value shown in Table 31 is the resulting dead load of the final girder design. The spreadsheet used during the calculation process can be seen in Appendix C where the dead load of other investigated cross-sections is displayed.

**Table 31 – Dead Load Values for Final Design**

| Bridge Component | Dead Load (lb/ft) |
|---|---|
| Railings | 126 |
| Asphalt Wearing Surface | 1680 |
| Concrete Deck | 4113 |
| One Girder | 1875 |

The dead load moment applied to the superstructure was calculated in two steps. A moment was first calculated based on the dead load of the railings, wearing surface and concrete deck. This moment was calculated to be 1184 k-ft and was constant for each of the evaluated cross-sections. This moment was calculated based on the assumption that each girder would support an equal portion of the load. Additionally, a separate moment was calculated due to the girder weight. The moment due to the girder weight for the final design was calculated to be 2251 k-ft. The rest of the girder weight moments can be seen in the spreadsheet in Appendix C.

### *4.3.2.2 Live Load Calculations*

As discussed in the previous chapter, the live load conditions of the superstructure were determined using the design truck shown in Figure 15 as well as a 0.64 k/ft design lane load. The distribution factors for the interior and exterior beams were calculated based on the equations in Table 14 and then applied to these two loading conditions. The resulting distribution factors and adjusted loads are shown in Table 32 below. For the interior girders, the larger resulting value of the two distribution factor equations was chosen to be used during the design in order to produce the maximum loading condition. The calculations that were completed to obtain the results in Table 32 can be seen in Appendix C.

**Table 32 - Distribution Factors and Live Loads for Final Design of the Interior and Exterior Girders**

| Interior Girder | Value |
|---|---|
| Distribution Factor | 0.56 |
| Rear Axle Load (k) | 17.92 |
| Front Axle Load (k) | 4.48 |
| Design Lane Load (k/ft) | 0.358 |
| Exterior Girder | |
| Distribution Factor | 0.69 |
| Rear Axle Load (k) | 22.08 |
| Front Axle Load (k) | 5.52 |
| Design Lane Load (k/ft) | 0.442 |

The live loads shown in Table 32 were applied to girders using the *RISA-2D* software discussed in the Methodology. The design lane load was applied longitudinally to the girders as a distributed load and the design truck axle loads were applied as a moving load. The loads of the design truck were then moved along the length of the girder, and the reaction forces within the girder were calculated for each location. Figure 22 below displays the applied loads for the exterior girder analysis. The interior girder analysis was conducted in a similar manner.

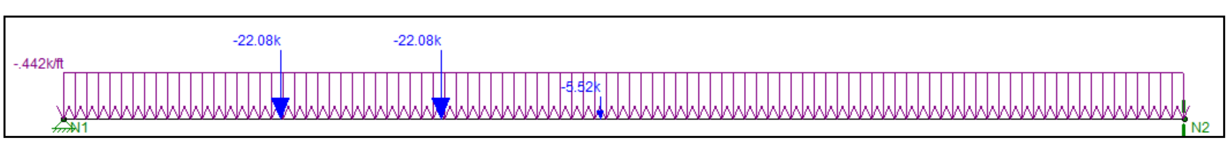


**Figure 22 - Application of Loads in RISA - 2D (Exterior Girder)**

It was determined that the maximum moment the girder experienced occurred in the center of the beam when the rear axles of the truck were spaced 14 feet apart. The resulting maximum moments for the interior and exterior girders are shown in Table 33.

Table 33 - Maximum Live Load Moments for Interior and Exterior Girders

| Girder Location | Maximum Live Load Moment (k-ft) |
|---|---|
| Interior | 1261 |
| Exterior | 1554 |

As has been discussed previously, the determination of these moments required the use of an iterative process. Changes in the size and spacing of the girders also changed the distribution factors, and ultimately the applied loads. As a result, once a new girder size was selected, the loads and girder cross-section were input into the *RISA* software to determine the new maximum moment. If this new moment required a new girder size, the process was repeated until an adequate girder size and spacing was obtained. The moments shown in Table 33, are those that were obtained for the final girder size. This iterative process was aided by the use of two spreadsheets (one for the exterior girder design and one for the interior girder design). These two spreadsheets can be seen in Appendix C.

#### *4.3.2.3 Cross-Section Details*

The cross-section dimensions of the spread box girders were obtained using the loads presented in the previous subsections and through the processes described in the Methodology chapter. Multiple cross-sections were evaluated during this project in order to determine the most adequate design. The final cross-sectional design of the bridge can be seen in Figure 23. A detailed view of the interior and exterior girder cross-section can be seen in Figure 24. Both the interior and exterior girders have a depth of 45 inches and a width of 54 inches. The web-wall and flange thicknesses of the girders are each 12 inches. The value "e" shown in Figure 24 corresponds to the profile of the prestressing steel, which will be discussed in the next subsection.

Although it was decided to use interior and exterior girders with the same cross-sectional dimensions, through independent analysis of the interior and exterior girders it was determined that the interior girders would meet the proper strength requirements with a depth of 43 inches. Using a depth of 43 inches would reduce the amount of concrete used in the interior girders, which would also decrease the cost of these girders slightly. However, for constructability it was decided that the height of the interior girders should be increased the extra two inches so that the cross-section was identical to the exterior girders. This allowed for a more conservative design of the girders as well as resulted in the need for only one prestressing design.

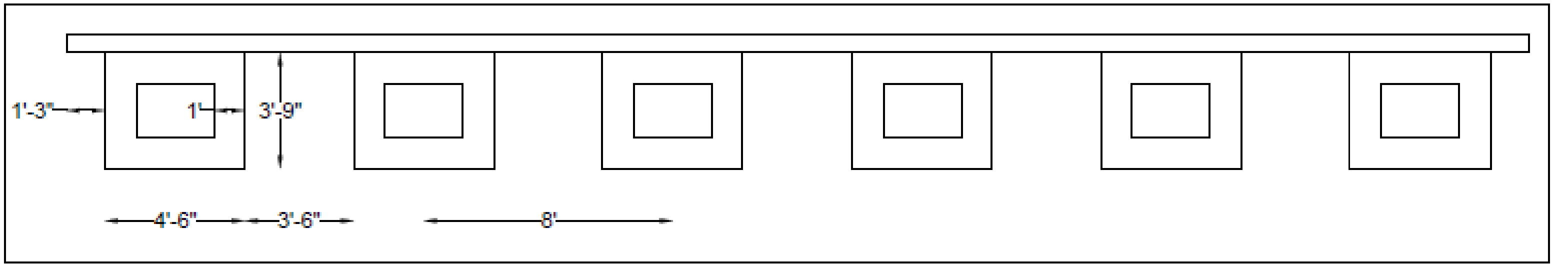


Figure 23 - Cross-Section of Final Bridge Design

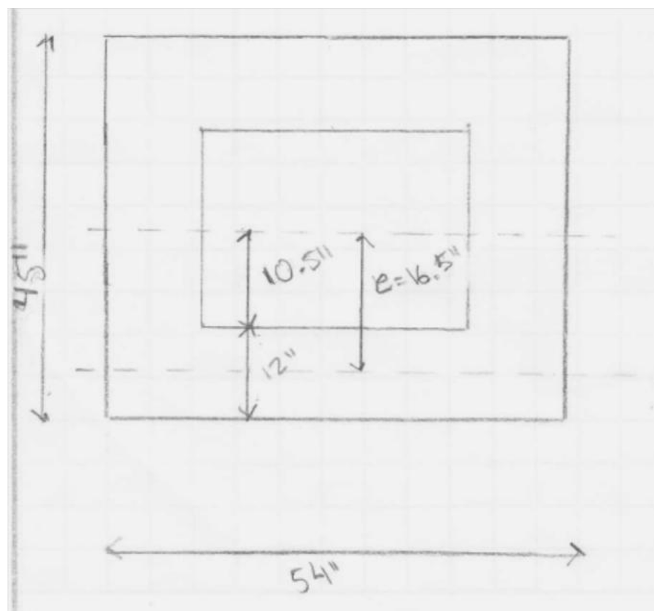


Figure 24 - Detail Cross-Section of Interior and Exterior Girders

### *4.3.2.4 Concrete Deck Reinforcement Design*

The reinforcement steel was calculated for a 12 inch section by 7 inch thickness of the deck. *RISA-2D* software was used to produce a maximum negative and positive moment for the deck using the dead and live loads calculated in the previous sections of this report. Table 34 shows the variables used and the calculations results.

**Table 34 - Summary of Reinforcement Steel Calculations**

| Variable | Value |
|---|---|
| Max Negative Moment | 29.654 k-ft |
| Max Positive Moment | 32.793 k-ft |
| $f'_c$ | 4 ksi |
| $f_y$ | 60 ksi |
| $\beta_1$ | 0.85 |
| $\epsilon_u$ | 0.003 |
| $\Phi$ | 0.9 |
| b | 12 in |
| $\rho_{(0.005)}$ | 0.0181 |
| *d* | 6.32 in |
| *a (assumed)* | 2.00 in |
| *a(calculated)* | 2.01 in |
| $A_s$ for positive moment | 1.37 $in^2$ |
| $A_s$ for negative moment | 1.24 $in^2$ |

Since the assumed depth of the stress block, *a*, was within 0.01 inches to the calculated *a* value, calculations were completed with the assumed *a* value. One No. 11 bar was selected with a cross-sectional area of 1.56 $in^2$ to satisfy the minimum required area of reinforcement required per foot of the deck for the maximum positive moment. A No. 10 bar was selected with a cross-sectional area of 1.27 $in^2$ to satisfy the minimum required area of reinforcement required per foot of the deck for the maximum negative moment. The No. 11 bars will be placed 2.5 inches from the bottom of the deck and spaced 12 inches apart for the entire length of the deck. The No. 10 bars will be placed 2.5 inches from the top of the deck and spaced 12 inches apart for the entire length of the deck. The bars were both placed 2.5 inches from the top and bottom of the deck to meet AASHTO minimum cover requirements.

#### *4.3.2.5 Prestressed Steel Design*

Through background research it was determined that the spread box girders would be prestressed concrete members. Post-tensioned steel strands were chosen and designed using the process and resources discussed in the Methodology Chapter. As a result of the steel strands being post-tensioned, ducts were used in order to house the strands and allow for tensioning after the concrete has set.

As discussed in the Methodology, the book Design of Prestressed Concrete by Arthur Nilson was utilized to determine the location and number of prestressed steel strands needed in the box girders shown in Figures 23 and 24 (1978). The value "e" shown in Figure 24 represents the eccentricity of the steel strands, or in other words, the distance the strands are located from the center of the girder cross-section. This distance will vary with changes in cross-section dimensions and concrete strength properties.

The strength parameters discussed in the Methodology were based on a specified concrete compressive strength of 8000 psi. The values of the other parameters used in the design of the girders and prestressed steel are shown in Table 35. Following Table 35, Table 36 displays the strand properties calculated for the spread box girders.

**Table 35 - Design Values for Prestressed Steel Design**

| Parameter | Symbol | Value |
|---|---|---|
| Specified compressive strength of concrete (psi) | f'c | 8000 |
| Compressive strength of concrete at time of initial prestressing (psi) | f'ci | 6000 |
| Allowable compressive stress immediately after transfer (psi) | fci | 3600 |
| Allowable tensile stress immediately after transfer | fti | 232 |
| Allowable compressive stress at service load | fcs | 3600 |
| Allowable tensile stress at service load | fts | 537 |

Table 36 - Steel Strand Properties

| Property | Value |
|---|---|
| Grade of steel | 270 |
| Required area of prestressed steel ($in^2$) | 16.04 |
| Size of steel strands – diameter (in) | 0.6 |
| Cross-sectional are of strands ($in^2$) | 0.217 |
| Total number of required strands | 74 |

As can be seen in Table 36, a strand size of 0.6 inches was chosen to be used for the bridge design. This value was obtained per the PCI Bridge Design Manual because this size would result in fewer required strands and would take full advantage of the concrete strength (2003). The steel grade shown in Table 36 was also obtained from the Bridge Design Manual because this grade has a minimum ultimate strength of 270 ksi and is most often used in prestressed bridges (2003).

Once the size and required number of strands were determined, the spacing requirements of the prestressed strands were designed. According to the PCI Bridge Design Manual, steel strands used in post-tensioning are often formed into tendons that include a minimum of one strand to a maximum of 55 strands (2003). In addition, these tendons are then placed in post-tensioning ducts that must be at least ¼ inch larger than the nominal diameter of the tendon, unless multiple tendons are used in which case the ducts should be at least twice the cross-sectional area of the tendons (PCI, 2003).

As a result of the PCI specifications, the chosen design for the prestressed steel consists of five post-tensioning ducts that contain three tendons. Each tendon consists of five total strands, resulting in 75 strands for the entire girder. Although this is one more strand than is required, the extra strand will improve constructability since all tendons are the same and will add strength to the girder. The complete design specifications for the prestressed steel are shown in Table 37 and Figure 25. As can be seen in Table 37, the total cross-sectional area of the duct is 7.07 $in^2$, which larger than twice the cross-sectional area of the tendons.

Table 37 - Prestressed Steel Design Specifications

| | |
|---|---|
| Number of Strands | 75 |
| Strands per Tendon | 5 |
| Number of Tendons | 15 |
| Tendons per Duct | 3 |
| Number of Ducts | 5 |
| Duct Diameter (in) | 3 |
| Duct Area ($in^2$) | 7.07 |
| Total Tendon Area ($in^2$) | 3.26 |
| On Center Duct Spacing (in) | 9 |

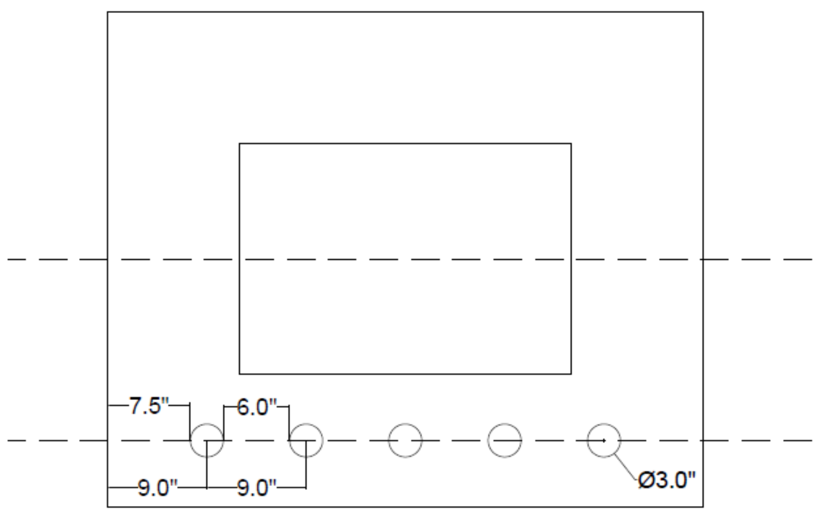


Figure 25 - Location and Size of Post-Tensioning Ducts

This section has presented the complete design of the bridge superstructure. Some areas of design could have been considered and explored further, but were not part of the design due to time constraints. Additional areas of design and recommendations for future work are discussed in the Conclusions chapter of this report. As discussed in the Methodology, the substructure was designed concurrently with the superstructure based on assumed design load values, and the resulting capacity was compared to ensure it could support the required loads. The design of the substructure will be presented in the next section.

## 4.4 Substructure

The substructure has three sets of results: spread footing design, deep pile design, and final design results. Each of these has a bearing capacity and elevation drawing for reference. The spread footing and deep pile initial designs served as comparison results for the final structure as explained in Chapter 3.4.

It is important to note that the substructure's results were entirely conditional based on site assumptions. After a site visit to the Quaboag River Bridge, it was determined that the soil surrounding the bridge was sandy soil. The soil profile was considered entirely sandy soil making the substructure over designed but with sufficient capacity that can now be refined.

### 4.4.1 Spread Footing Design

The spread footing design was an initial assessment of the necessity of a deep foundation for the bridge. A spread footing was chosen as a shallow foundation to test. The footing design of the spread footing and full height abutment is shown in Figure 26.

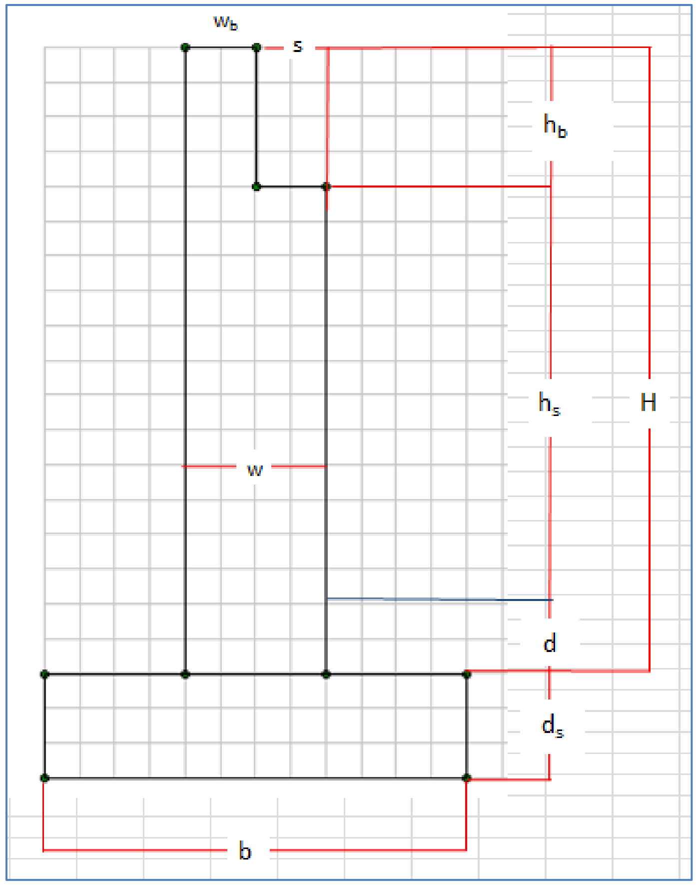


Figure 26 - N-S Elevation View of Spread Footing Design

By the Terzaghi Method, the ultimate bearing capacity of the footing and dead load of the originally dimensioned substructure were calculated to be 1.71 ksf and 3.15 ksf respectively (Appendix D). A shallow footing was determined to be inadequate because the bearing capacity was smaller than the dead load of the substructure. Hand calculations were completed on the effects of the water table and saturated soil on the spread footing. These effects resulted in undesirable foundation integrity. Therefore the footing design was deemed entirely unfavorable for this project. Therefore, no reinforcement was designed for the footing design as it was not to be investigated further.

### 4.4.2 Deep Pile Design

The deep pile design doubled as a test of the adequacy of a deep foundation for the bridge and a framework for creating *Microsoft Excel* spreadsheets to simplify the calculations for the final design, which was a modified version of the deep pile design. The pile design was purposely overdesigned since the loads from the superstructure were not yet available. A pile design was considered adequate for the substructure foundation because the pile bearing capacity was much greater than the dead load of the substructure. Table 38 displays the calculated values of the pile capacities and dead load of the substructure. Reinforcement was also conservatively designed with the dimensions of the original deep pile foundation substructure (Table 39). Figure 27 and Figure 28 display the N-S and E-W elevation views of the integral abutment and pile group.

**Table 38 - Unfactored Dead Load, Pile Capacity and Pile Group Capacity of Original Deep Pile Substructure Design**

| Element | Description | Value | Unit |
|---|---|---|---|
| Unfactored DL of substructure | Load per square foot of the substructure: abutment wall, backwall, and pile cap | 3.15 | ksf |
| Single pile bearing capacity | Bearing capacity of a single pile, given the height of the abutment from the top of the soil, and assumed unit weight of the soil | 36.75 | ksf |
| Group bearing capacity | Allowable group bearing capacity of the pile group designed | 1021.62 | ksf |
| Remaining group bearing capacity | Bearing capacity that the superstructure can exert on the substructure before failure | 1018.47 | ksf |

**Table 39 – Original Deep Pile Substructure Reinforcement Design**

<table>
<tr><th rowspan="2">Abutment Element</th><th colspan="2">Flexural R.F.</th><th colspan="2">Shear R.F.</th></tr>
<tr><th>Bars</th><th>Spacing</th><th>Bars</th><th>Spacing</th></tr>
<tr><td>Bottom of pile cap</td><td>2 #11 bars</td><td>48 in</td><td rowspan="2">#5</td><td rowspan="2">10 in</td></tr>
<tr><td>Top of pile cap</td><td>4 # 8 bars</td><td>18 in</td></tr>
<tr><td>Abutment wall – top of backwall</td><td>1 # 9 bar</td><td>18in</td><td rowspan="2">#4</td><td rowspan="2">10in</td></tr>
<tr><td>Abutment wall</td><td>12 # 6 bars</td><td>18 in</td></tr>
</table>

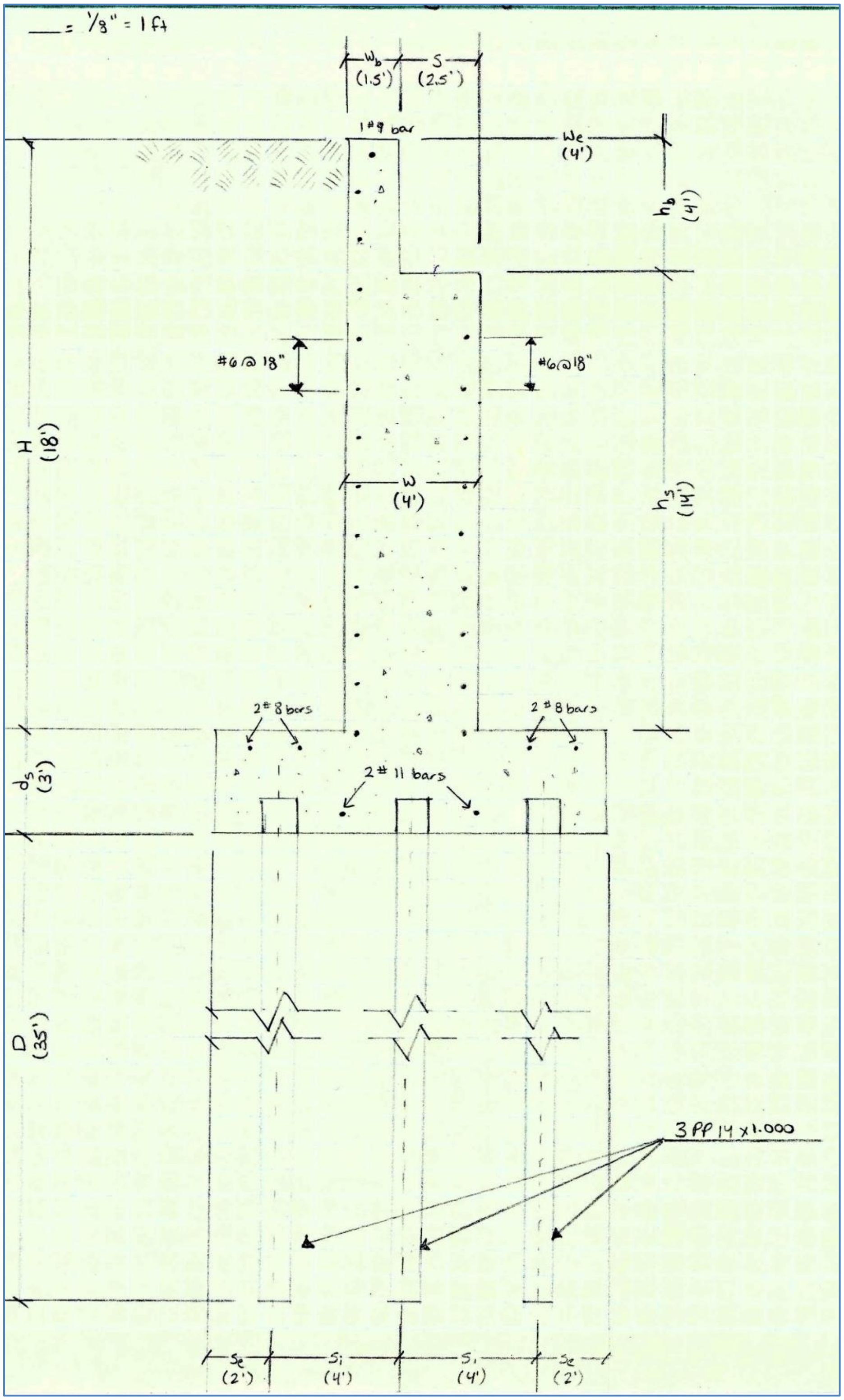


Figure 27 - N-S Elevation View of Original Deep Pile Substructure Design (1/8" = 1')

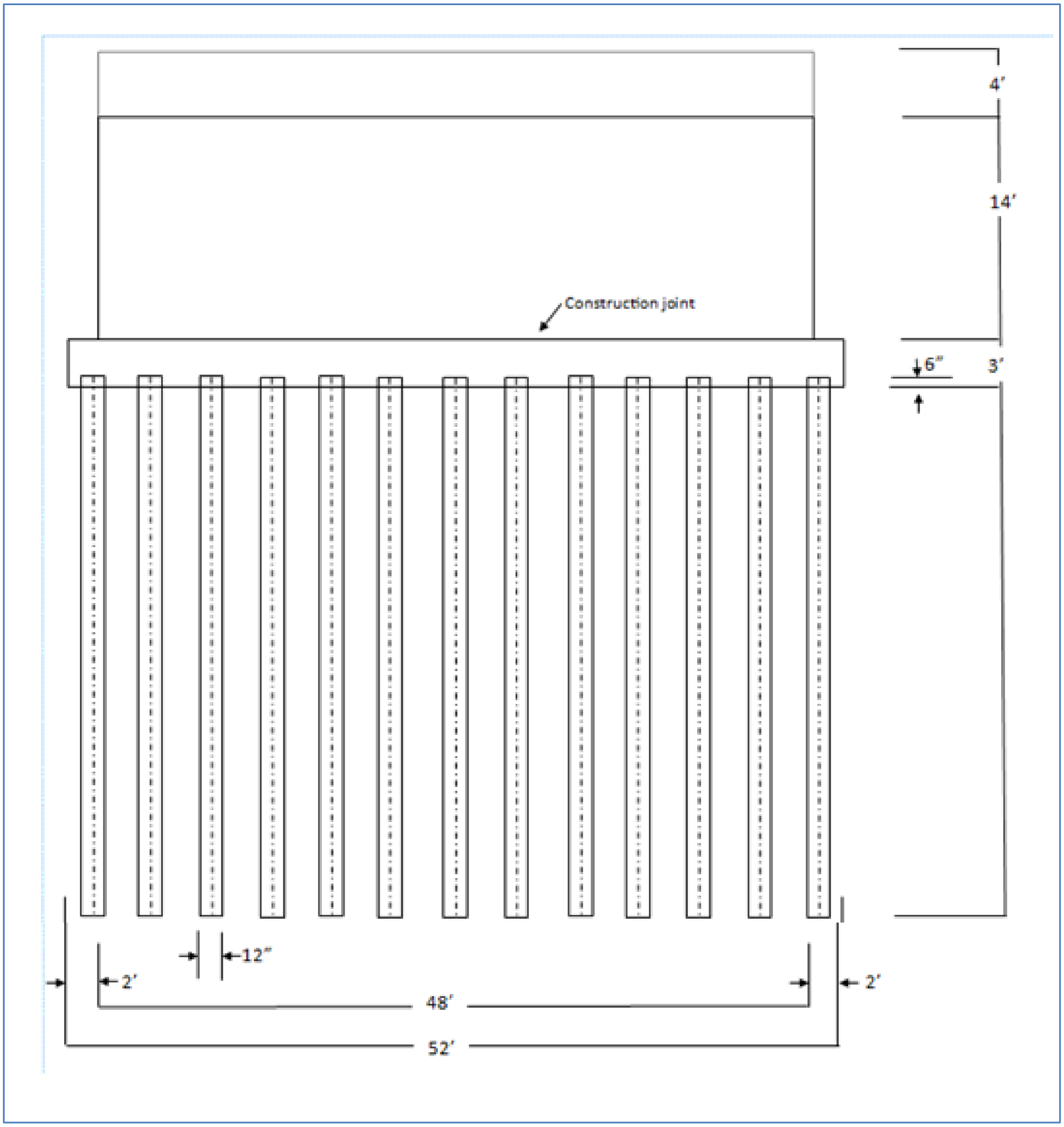


Figure 28 - E-W Elevation View of Substructure Original Deep Pile Substructure Design

### 4.4.3 Bearing Pad

The bearing design was designed after the superstructure's loads were finalized. Appendix D shows the complete set of calculations of the fabric reinforced elastomeric bearing pad. Figure 29 below shows the fabric reinforced bearing, drawn in a 1/8" = 1" scale that would sit under each girder.

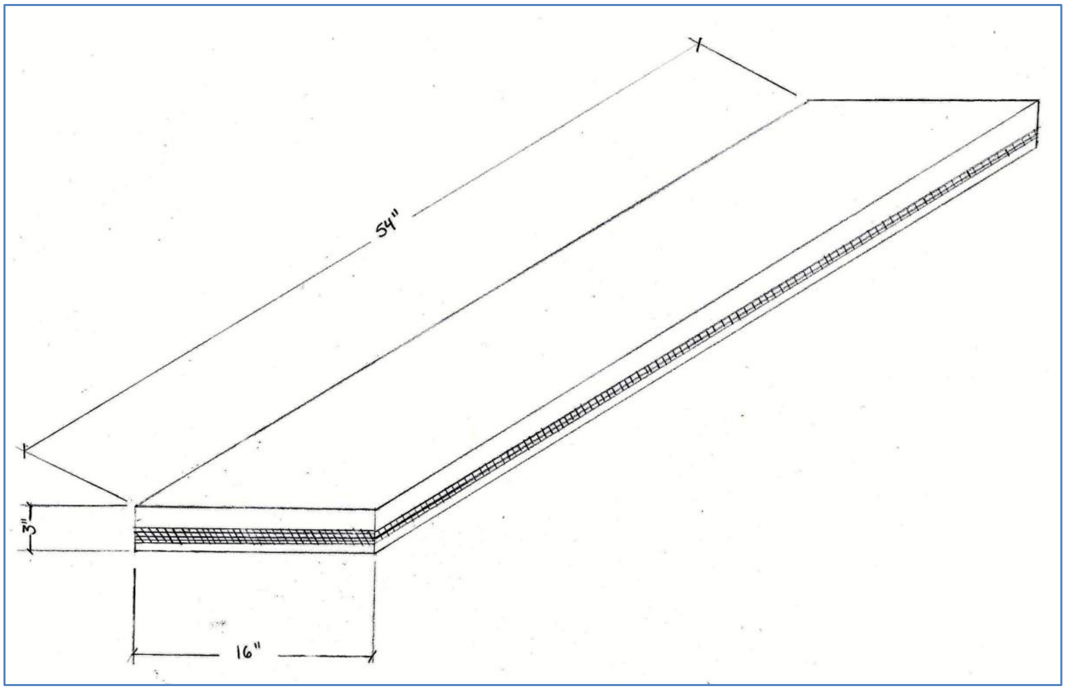


Figure 29 - Fabric Reinforced Elastomeric Bearing Pad (1/8" = 1')

### 4.4.4 Final Design of the Substructure

The substructure design was finalized after the design live and dead loads were established from the superstructure. Ethical considerations were accounted for though the use of ACI and AASHTO specifications during the final design process. The design capacities were calculated as close to the factored loads as possible for economic considerations. These considerations allowed for constructability of the bridge. For example, because the reinforcement was designed for the factored loads acting on the substructure, realistic rebar sizes and spacing were used in the final deep pile substructure design.

All final dimensions and design capacities of the substructure can be viewed in Appendices D and E. Table 40 and 41 below display the final bearing capacities and reinforcement design of the substructure respectively. Figures 30 and 31 display different elevation views of the substructure. This abutment and pile design would be able to withstand the designed superstructure in sandy soil conditions.

Table 40 - Factored Dead Load, Pile Capacity, and Pile Group Capacity of Final Deep Pile Substructure Design

| Element | Description | Value | Unit |
|---|---|---|---|
| DL of Substructure | Load per square foot of the substructure: abutment wall, backwall, and pile cap | 2.33 | ksf |
| DL of Superstructure | Load per square foot of the girders, deck, wearing surface, and parapets | 0.54 | ksf |
| LL of Superstructure | Load due to vehicular traffic | 1.89 | psf |
| Single Pile Bearing Capacity | Bearing capacity of a single pile, given the height of the abutment from the top of the soil, and assumed unit weight of the soil | 3.64 | ksf |
| Group Bearing Capacity | Allowable group bearing capacity of the pile group designed | 30.48 | ksf |

Table 41 - Final Reinforcement Design of the Deep Pile Substructure

| Abutment Element | Flexural R.F. | | Shear R.F. | |
|---|---|---|---|---|
| | *Bars* | *Spacing* | *Bars* | *Spacing* |
| pile cap | 8 #10 bars | 6 in | #14 | 6.5 in |
| top of backwall | 5#4 bars | 4 in | #7 | 18in |
| bridge seat | 4# 5 bars | 3.5 in | #4 | 18in |
| abutment wall - under bridge seat | 5 #5 bars | 18 in | #7 | 18 in |
| abutment wall - backwall | 8 #4 bars | 18in | #4 | 18 in |

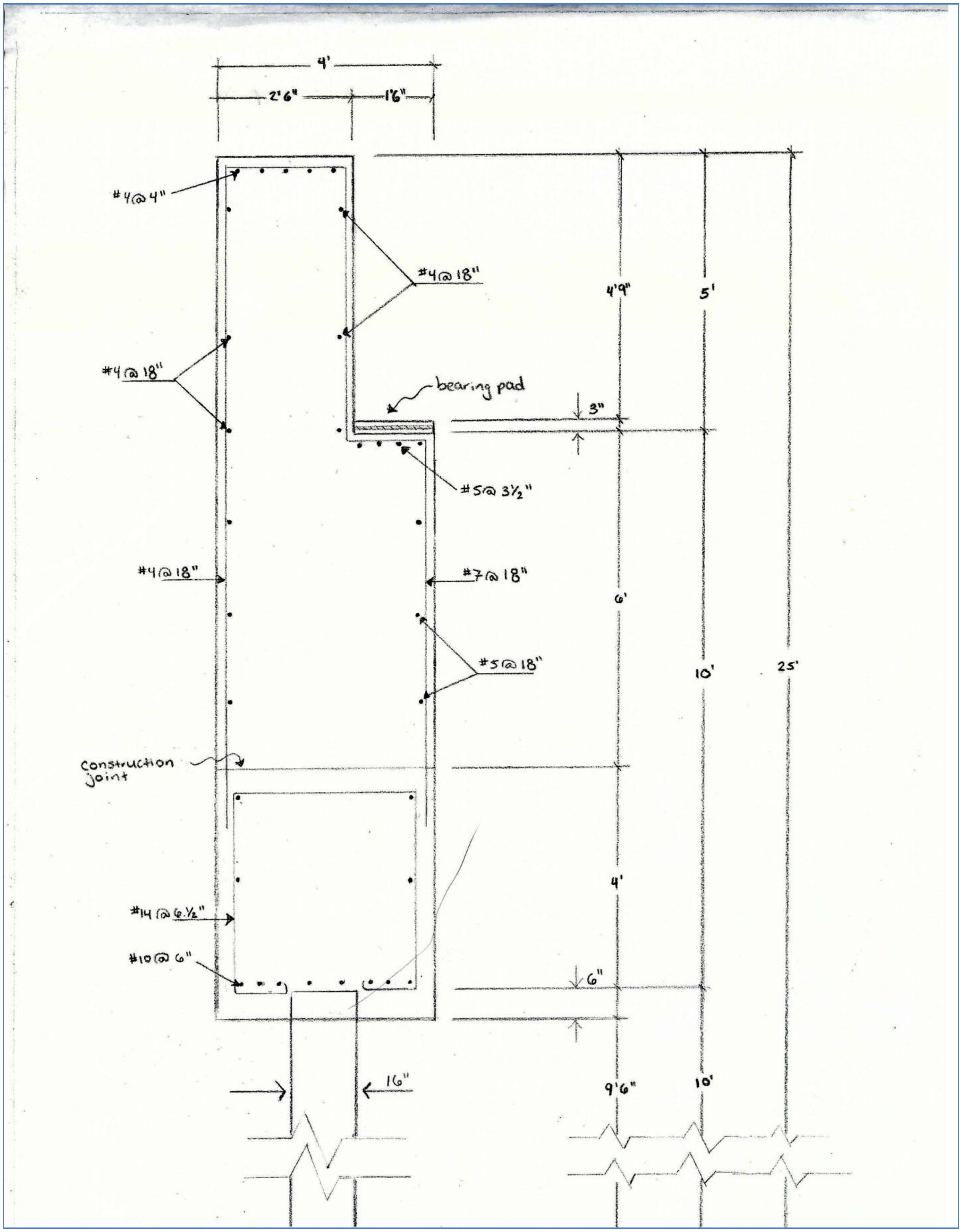


Figure 30 - N-S Elevation View of Final Substructure Design (1/4" = 1')

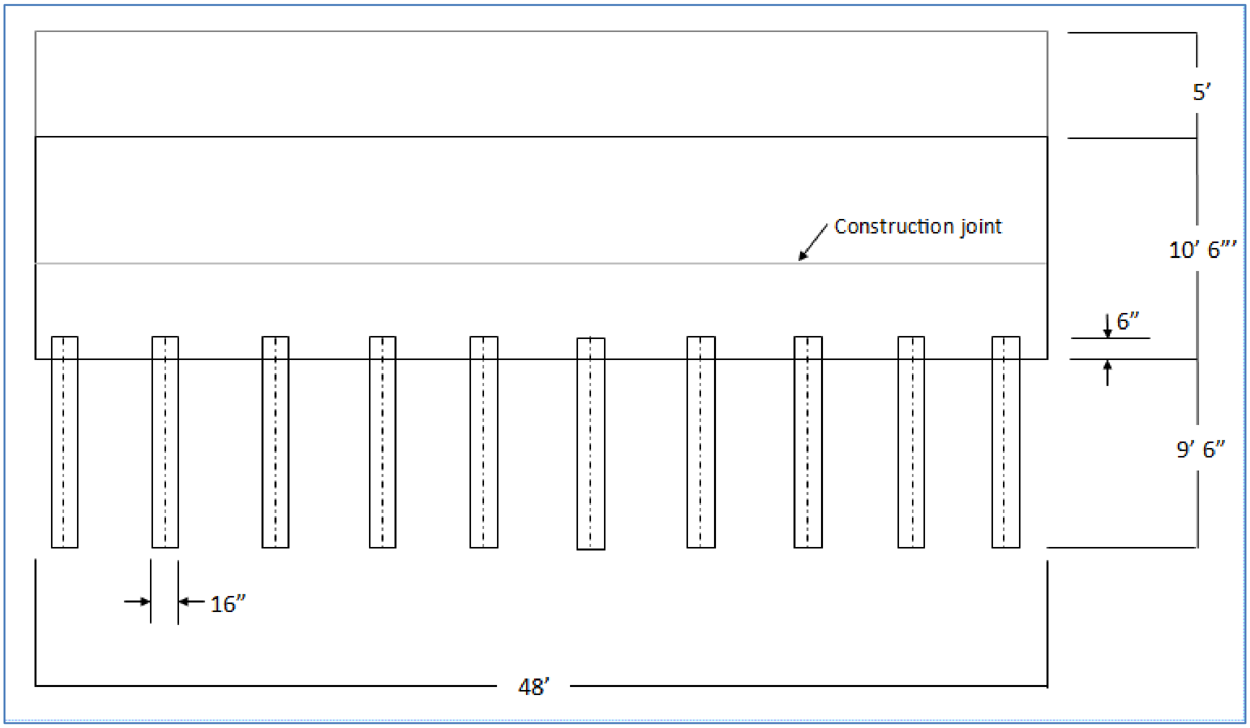


Figure 31 - E-W Elevation View of Final Substructure Design

### 4.4.5 Considerations

As briefly mentioned above, three main considerations were kept in mind during the design of the substructure: ethical, economic, and constructability concerns. Any ethical considerations were addressed by referring to ACI and AASHTO requirements for bridge design. Economic considerations included strategies such as minimizing the amount of material needed in the substructure. Constructability was considered mainly through dimensioning of the substructures. This includes designing the substructure to the nearest 0.5 foot, and using different sizes of rebar with standard requirements.

An example of these considerations was in the design of the abutment wall’s reinforcement. The cantilever wall needed to meet minimum spacing and area reinforcement requirements set forth by ACI and ASHTO discussed in Section 3.4.5.5. Meeting these requirements fulfilled the ethical considerations. The reinforcement was designed to satisfy the factored loads with the smallest amount of reinforcement possible. This satisfied the economic considerations as it was not an over design and constructability considerations since fewer bar sizes simplifies construction. Finally, the abutment wall used realistic rebar sizes making the substructure more constructible. Each element in the substructure went through this process to make the substructure ethical, economical, and realistically constructible.

### 4.4.6 Conclusion

All spread footing, deep pile, and final deep pile equations, assumptions, and calculations for the substructure can be found in Appendix D. One of the most important results drawn from the substructure's final design was that its bearing capacity successfully exceeded the loads from the superstructure. This meant that the final design was proportional to the superstructure's dimensions and would support dead and live loads from the bridge.

## 4.5 3D Model

To synthesize the elements of the final design of the bridge and facilitate visualization, a 3D model was created using *SolidWorks 2012* computer aided design (CAD) software. The model encompasses all of the components of the proposed superstructure and substructure including the piles, pile cap, abutment, elastomeric bearing pad, girders, concrete deck, asphalt wearing surface, sidewalks, and railings. Figures 32 through 34 provide various views of the bridge model. All of the dimensions presented in the previous two sections were used in this model to create an accurate representation of the proposed bridge design.

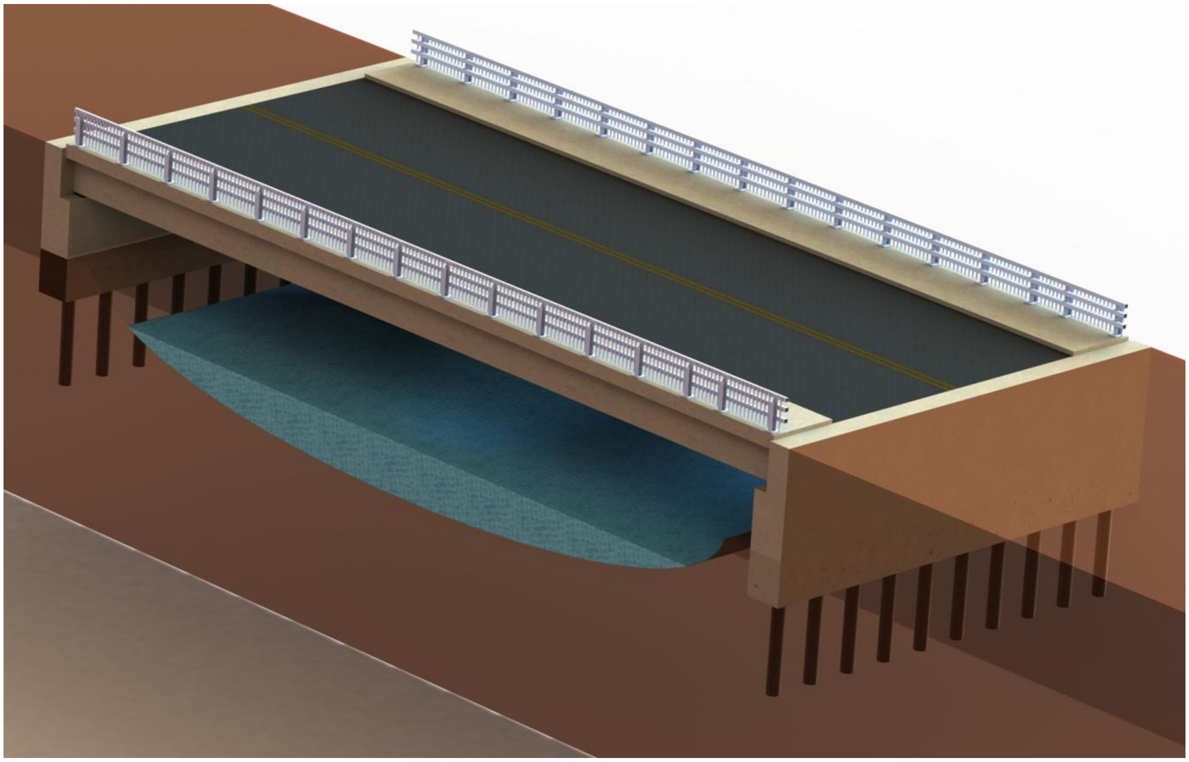

Figure 32 - Full View of Quaboag River Bridge Design

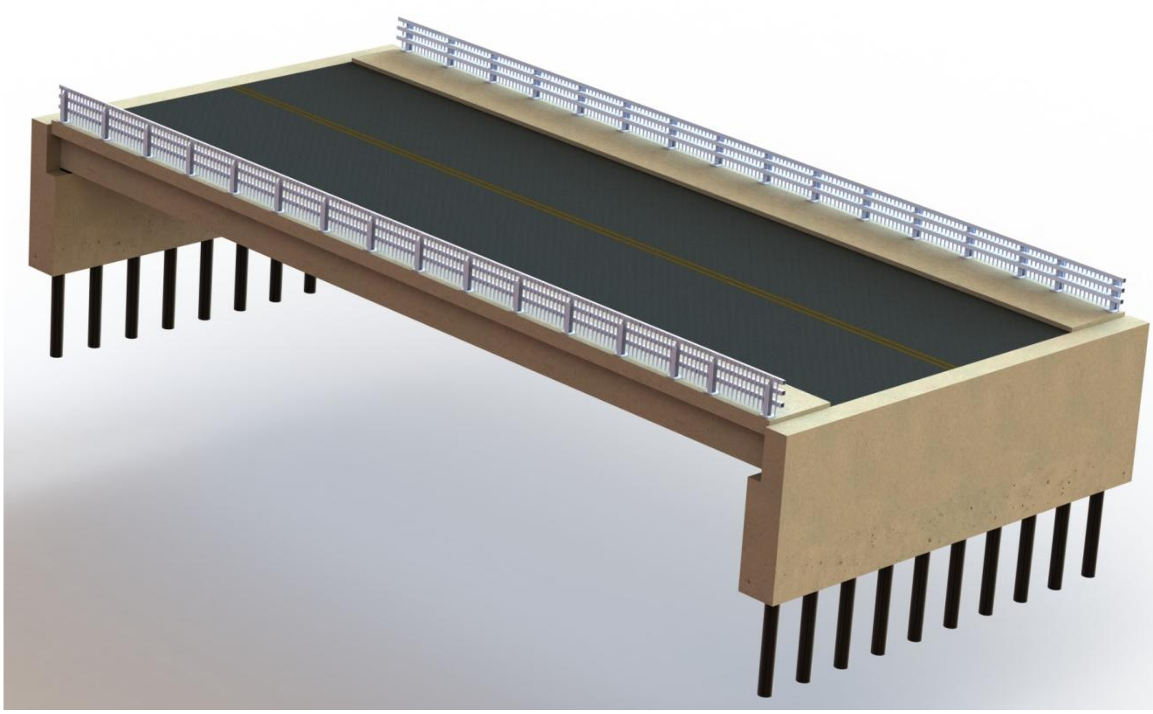

Figure 33 - Bridge Model without Environment

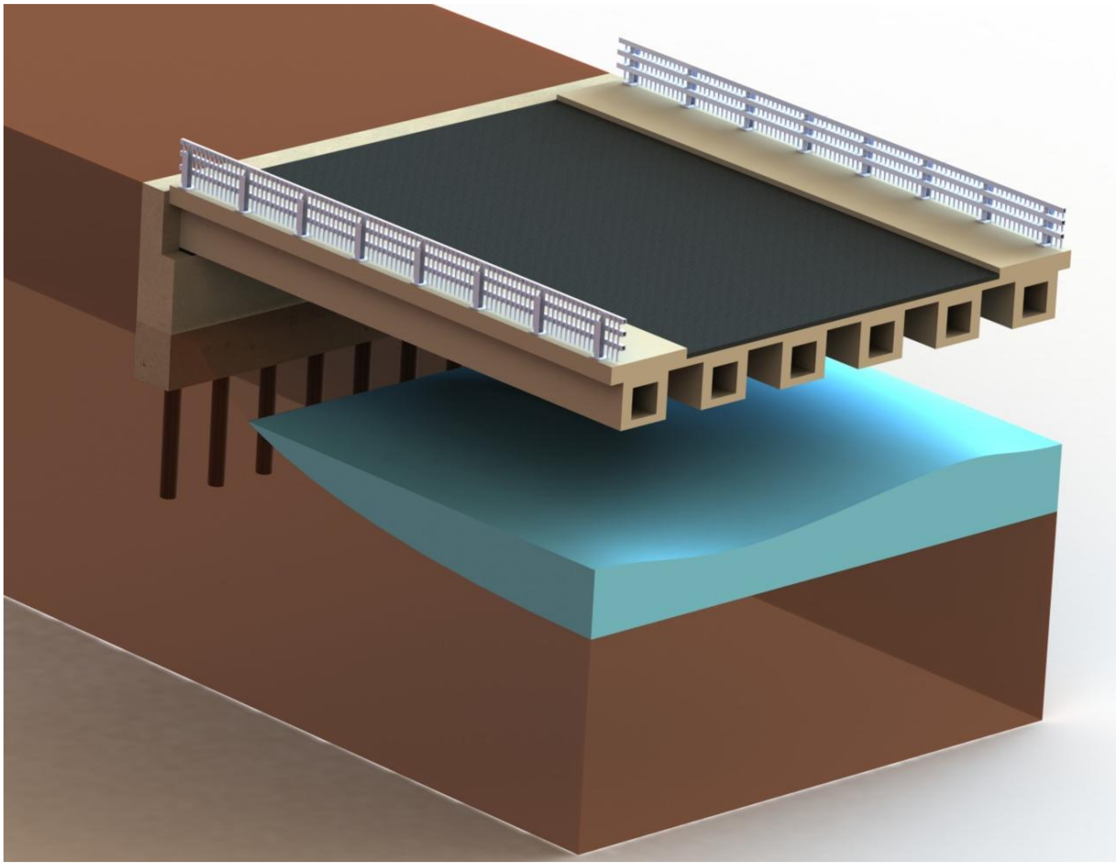

Figure 34 - Cross-Section View of Quaboag River Bridge

## 4.6 *BridgeLCC* Analysis

The *BridgeLCC* program produced a LCCA of the concrete girder and steel girder designs. Assumptions were given values from sources, approximations, or estimations to complete detailed Life-Cycle Cost (LCC) by Bearer, LCC by Period, and Cumulative Cost in Current-Year Dollar graphical depictions. These depictions represented the LCCA of the project.

### 4.6.1 Assumptions

A number of assumptions were necessary to use the NIST *BridgeLCC* program. Table 42 below shows an alphabetical summary of assumptions, sources, and corresponding values.

Table 42 - Assumptions, Respective Sources, and Values Used in the BridgeLCC Analysis

| Assumptions | Source | Value |
|---|---|---|
| 50-Year Flood Event | Estimation based on past Accelerated Bridge Projects (ABPs) | $500,000 |
| Accidents per Million Vehicle-Miles | MassDOT.gov | 1.47 |
| ADT by 2113 | Approximation based on prior knowledge of traffic flow | 20,000 vehicles |
| Agency Cost | MassDOT.gov | $3.71 million (concrete total budget) |
| Average Daily Traffic (ADT) | MassDOT.gov | 6,000 vehicles |
| Concrete Bridge Final Disposal Cost | Estimation based on background research | $1,000,000 |
| Cost per Accident | Approximation based on MassDOT accident report website | $1,000 |
| Cost to Rehabilitate Steel Bridge Design | Estimation based on past ABPs | $1,000,000/25 yrs |
| Disposal Cost | Approximation based on background research | $1,000/10 yrs (concrete); $1,000/25 yrs (steel) |
| Driver Delay Cost | *Traffic and Highway Engineering* N. Garber & L. Hoel | $5.50 |
| Easements and Right of Way Costs | Telegram and Gazette (Telegram and Gazette 2012) | Total: $30,000 (concrete); $5,000 (steel) |
| Inflation Rate | Bureau of Economic Analysis | 1.90% |
| Length of Workzone | Google Maps | 0.5 miles |
| Operations, Maintenance, and Repair Cost | Estimation based on past ABPs | $10,000/10 years (concrete) |
| Real Discount Rate | Office of Management and Budget | 2.3 |
| Service Life | NIST *BridgeLCC* | 10 |
| Speed of Road | Google Maps | 40 mph |
| Total Man-Labor Hours | Approximation based on prior knowledge of construction management | 13,800 hrs |
| User Cost | MassDOT.gov | $3.527 million |
| Worker's Wages | ENR.construction.com | $38.16/hr |
| Workzone Accidents per Million Vehicle-Miles | MassDOT.gov | 1.47 |

Any assumption with a source labeled "Approximation" was a best-guess approximation. For instance, the projected ADT by 2113 had an approximated value of 20,000 vehicles/day. This was a best-guess approximation based on a group member's prior experience in a traffic engineering class. The outcomes from the LCCAs were not sensitive to this approximation. Whether the ADT by 2113 was 6,000 vehicles/day or 20,000 vehicles/day, the total LCC values did not fluctuate by more than 1% in either case. This made the approximation's value non-vital to the resulting LCCAs and conclusions.

The same held true for the other approximations, since the assumption costs were low. For instance, the disposal costs of the concrete ($1,000/10yrs) and steel ($1,000/25yrs) designs are the most differentiating approximations; however, $1,000 is such a small value compared to the upper $3 million initial construction fee, that a disposal fee every 10 or 25 years is hardly noticeable in the LCC. Therefore the purpose of the disposal costs was to show trends in the life time of each bridge design, although the values of the disposals were negligible. This remained true for all approximations: the value was negligible in the LCC and conclusions, however important in the life-cycle depiction of costs for each design.

In contrast to approximations, estimations refer to best-guess values assigned to critical factors in the cost analysis of each bridge design. These mainly affected the steel girder design, as it would need a significant rehabilitation every 25 years. Estimations were necessary to realistically depict the costs that a concrete design versus steel design would incur over 100 years. Estimations were low values, as to not purposely favor the concrete design to the steel design. An instance such as an Act of God, in which a 50-year Flood would be considered, could potentially destroy the bridge, depending on the season. The estimated cost of $500 thousand it would take to rehabilitate or rebuild the bridge after such an event, was a low estimate because the group was fully aware it would cost over $1 million to rebuild a destroyed single span bridge Estimations were important to include, since they had a large effect on the LCC of the steel girder design and a significant impact on the differential cost.

### 4.6.2 Life-cycle Cost by Bearer

As mentioned previously in the Chapter 3, costs by bearer differentiated greatly depending on the girder material used in the bridge design. Overall, the concrete design would be funded by Massachusetts (the Agency) while the steel design would need to be funded by the Town of Brookfield (the User). Figure 35 below shows the approximate differences in LCC by bearer. Landowner (Third Party) costs would be $30,000 and $5,000 for the concrete and steel designs respectively. However, these costs would be supplemented by Brookfield for either design. Figure 35 also represents the costs that landowners would need to sustain if they were not supplemented for easement or right-of-way costs.

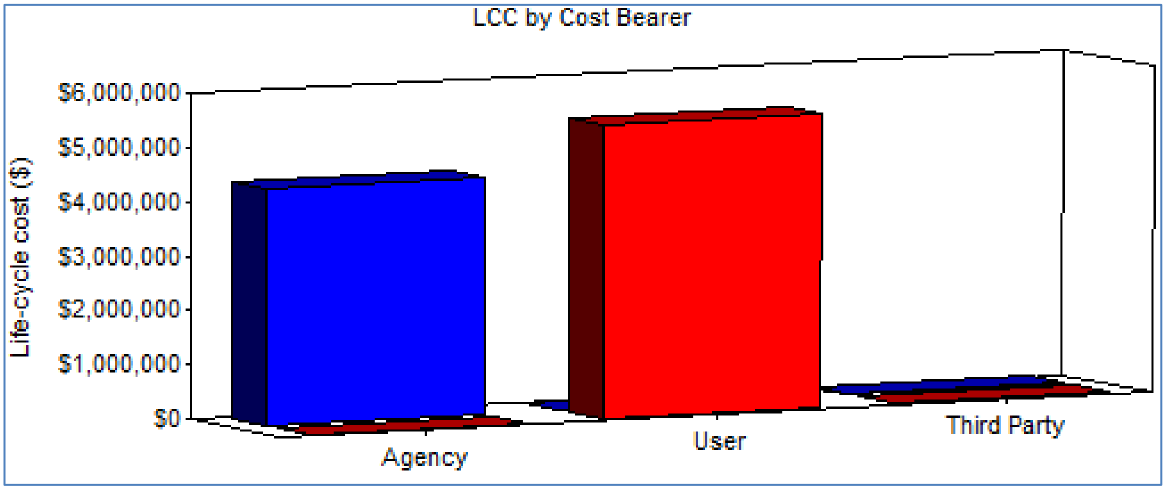


Figure 35 - LCC Cost by Bearer[2]

### 4.6.3 Life-Cycle by Period

The LCC by period (Figure 36) shows the differences in cost for the three major periods of life-cycle costs: initial construction, OM&R, and disposal. Initial construction represents the cost to replace the Quaboag River Bridge. OM&R is the operations, maintenance, and repairs costs over time. Disposal costs included any additional fees of cleaning up the river after initial construction, disposal of bridge materials from OM&R over time, and the final cost of decommissioning the bridge designs. The concrete design has a much lower cost in OM&R than the steel design as shown in Figure 36.

[2] Blue represents the concrete girder design; red represents the steel girder design; held true for all *BridgeLCC* graphical representations

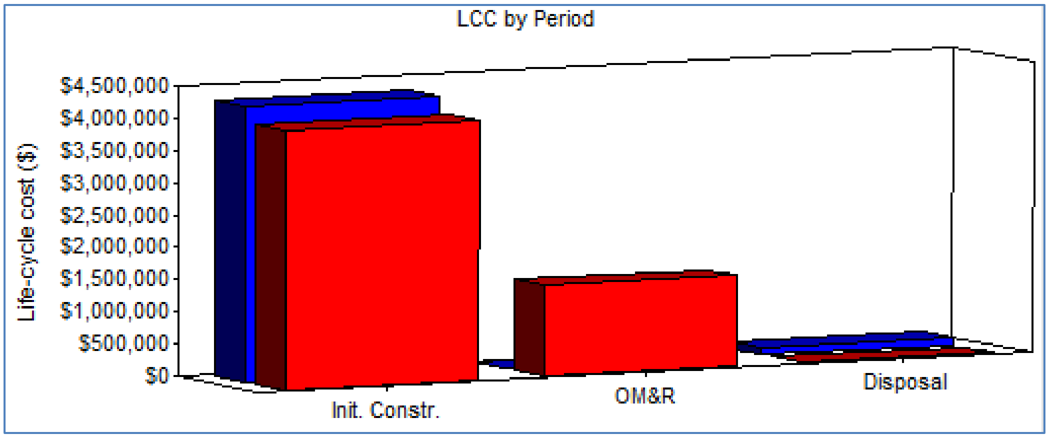


Figure 36 - LCC by Period

### 4.6.4 Cumulative Cost in Current-Year Dollars

The cumulative cost in current-year dollars takes the inflation rate and real discount rate into consideration in the analysis of the LCC. The base year is 2013 whereas the end year is 2113 in this LCCA. The *BridgeLCC* program applied the inflation rate and real discount rate to each cost input into the program to determine the LCC of both designs. Figure 37 below shows the total LCC of each bridge with an inflation rate of 1.90% and discount rate of 2.30%. The total LCC of the concrete design would be approximately $11 million while the total LCC of the steel design would be approximately $21 million over a 100-year service life.

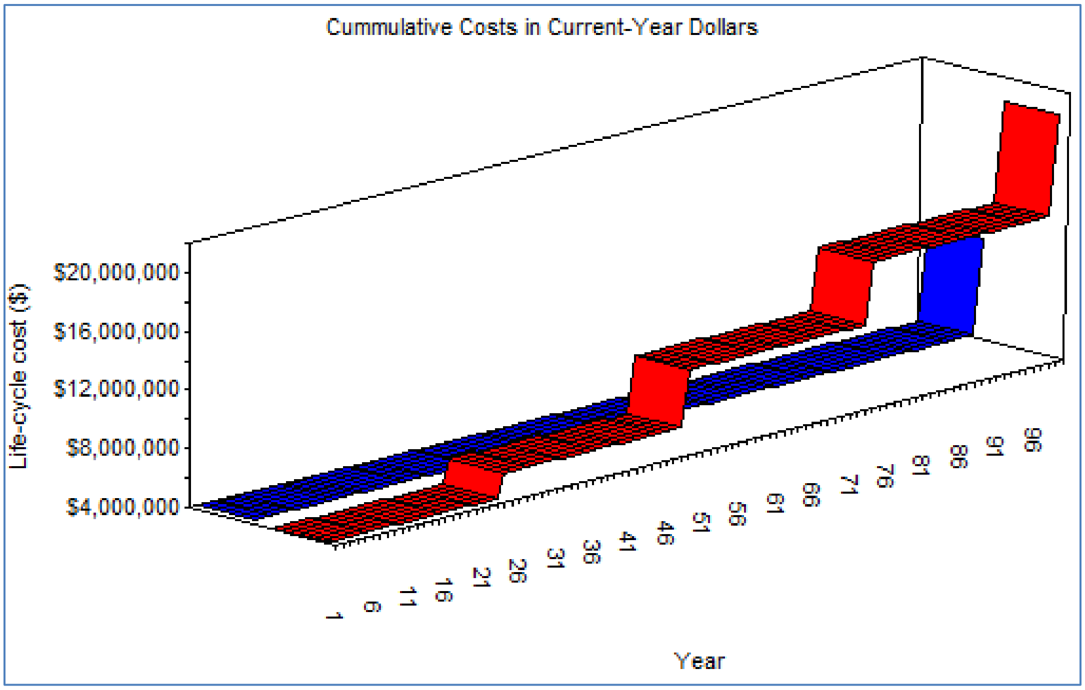


Figure 37 - Cumulative Costs in Current-Year Dollars

The inflation rate and real discount rate were suggested by the Federal Bureau of Economic Analysis and Federal Office of Management and Budget respectively (BEA, 2012 & Lew, 2011). The LCC of each design varied slightly with different real discount rates, as shown in Table 43.

Table 43 - Comparison of Total LCC with Differential Real Discount Rates

| **Control - Inflation Rate (1.90%)**<br>*Real Discount Rate* | **Total LLC of Concrete Design** | **Total LCC of Steel Design** |
|---|---|---|
| 0.00% | $5.319 million | $8.690 million |
| 0.40% | $4.987 million | $7.716 million |
| 0.80% | $4.764 million | $6.980 million |
| 1.30% | $4.585 million | $6.300 million |
| 2.10% | $4.430 million | $5.575 million |
| 2.30% | $4.407 million | $5.442 million |

# 5. Conclusions

## 5.1 Introduction

As explained in Chapter 1, the Quaboag River Bridge in Brookfield, Massachusetts needs to be replaced to satisfy federal 50-year flood plain requirements. Also, the Commonwealth of Massachusetts will only fund this replacement project if concrete girders are used in the new bridge design. As explained in Chapters 3 and 4, a value-based selection process was developed to determine whether a concrete or steel design would be a more ethical, economical, and sustainable design for the Quaboag River Bridge. It was determined that a concrete bridge would be more appropriate for the purposes of this project.

After concrete was chosen as the bridge's girder material, a spread box girder bridge was designed to satisfy the 50-year flood plain requirement. The bridge was also designed to satisfy other dimension requirements, using AASHTO requirements as guidelines throughout the project. This resulted in a concrete bridge designed to withstand AASHTO Strength I and Service I loads.

This project also conducted a life-cycle cost analysis (LCCA) of a concrete versus a steel girder bridge design. Since the concrete design resulted in a lower total life-cycle cost than the steel design, a concrete bridge would be the more economically desirable solution in the case of the Quaboag River Bridge.

The superstructure, substructure, and cost analysis elements of this project each produced conclusions of key findings, limitations during design, and recommendations for future major qualifying projects. These conclusions are explained in the following subsections.

## 5.2 Superstructure

The final design of the superstructure was established to support the design dead loads and live loads per AASHTO LRFD Bridge Design Specifications. The completed superstructure design includes size and spacing recommendations for the concrete box girders as well as the number and location of prestressed steel strands within the girders. The design of the superstructure was accomplished through an iterative process using *Microsoft Excel* spreadsheets to investigate multiple cross-section dimensions. Ultimately the most appropriate cross-section was selected and used to develop the rest of the superstructure design.

### 5.2.1 Key Findings

The key findings of the superstructure include the cross-section dimensions of the concrete girders and the number and location of the prestressed steel strands. The most influential part of the superstructure design was the effect of the live loads on the superstructure. As was discussed in Chapters 3 and 4, distribution factor equations were used for determination of the live load effects on a single girder. However, these equations rely on the spacing and depth of the girders, which was under investigation during the design process. As a result, the distribution factors and live loads were recalculated each time a satisfactory girder size was determined, then cross-section dimensions were re-checked to determine if they were still adequate.

The utilization of *RISA-2D* software greatly helped to facilitate this iterative design process. Once the distribution factors were updated for the selected cross-section design, the applied loads within the software could easily be changed. The moving load simulation was then run to determine the new live load moment on the girders. *RISA-2D* software also facilitated the design of both the interior and exterior girders. Since the distribution factors differed between the interior and exterior girders, models were created for each so that simulations could be run simultaneously. As a result, the design process was streamlined so that a girder size and spacing could be selected quicker and more accurately.

Another key finding during the design of the superstructure was the amount of flexibility that was available in the prestressed steel design. Once the required area of steel was determined, there were two grades of steel and multiple sizes of strands to choose from. The final design for the Quaboag River Bridge utilizes the most sensible design, because it results in fewer required strands. However, even after the number of strands was determined, the location of the strands within the girder could vary a great deal. This flexibility in the prestressed steel design allowed for easy modification if the design of another part of the superstructure changed.

### 5.2.2 Limitations

Limitations in the superstructure design include the design of secondary members, such as the diaphragms and the design of the composite interaction between the girders and deck. Although the scope of the prposed design does not cover these two components, the design could easily be modified to include them. If diaphragms were to be included in the design, the distribution factors for the girders may have to be adjusted because of the effects of the diaphragms on stiffening the girders and distributing the applied loads (Cai, Chandolu & Marcio, 2009). There are a variety of ways to account for diaphragms, and the permissible approach may depend on the jurisdiction that is in charge of constructing the bridge (Cai et al., 2009). In the case of the Quaboag River Bridge, MassDOT would have jurisdiction over the project and as a result would govern how to adjust for diaphragms.

If composite action between the deck and girders were to be included in the design, the required size of the girders and amount of prestressed steel may also change. Through composite action the deck of the superstructure carries some of the load longitudinally and as a result, the load capacity of the girders may be enhanced. However, the effects of composite interaction would need to be fully investigated in order to modify the design.

### 5.2.3 Recommendations

The use of *RISA-2D* software or a similar type of design software to determine the live load effects on the girders is highly recommended to facilitate the design of the superstructure and exploration of alternatives. The use of this software will allow for simplified live load calculations and will also result in easier modification of the design. Future MQP groups are advised to research possible software and learn how to use the software effectively in the beginning stages of the design process. This will allow for a more efficient use of time during the project.

It is also recommended that the superstructure be designed in parallel to the substructure. Although the substructure does rely on loads from the superstructure, an initial design can be created and then adjusted to ensure the substructure provides the appropriate support. Designing these two main bridge components concurrent with one another will also help future groups use time effectively during the design process.

## 5.3 Substructure

The final substructure design was successfully calculated to support the loads exerted by the superstructure through extensive use of *Excel* spreadsheets. With the proper dimensioning, the substructure was proportioned to match the dimensions of the superstructure as well as satisfy the 50-year flood plain that was determined through background research. Key findings and apparent limitations emerged while calculating the bearing capacity of the spread footing and deep pile substructure designs.

### 5.3.1 Key Findings

The most valuable discovery while designing the substructure was the importance of the soil profile. This project assumed a soil profile of entirely sandy soil, which is the worst case scenario since sandy soil is the least stable of soils and has the smallest internal friction angle. Assuming sandy soil resulted in an extremely low bearing capacity of the spread footing design, forcing the group to investigate a deep foundation option for the substructure. Additionally, the 50-year flood plain and depth of the saturated soil created concerns in the effectiveness of the shallow foundation design. Therefore, a pile design was deemed appropriate for this project after calculating the bearing capacity of the original deep pile design.

During the design process for the deep pile substructure, the flexibility in pile design was discovered. Increasing the depth of piles or number of piles added substantial bearing capacity to the substructure. It was also learned that varying the inner and outer diameters of the hollow piles slightly affected the bearing capacity of the piles. Due to the use of *Excel* spreadsheets it was easy to change the number of piles, size, and depth in the design. The piles resulted in being the most flexible element in the substructure design because they did not exert any additional loads on the substructure and only affected the bearing capacity.

Also discovered during substructure design was the difficulty of redesigning the flexural and shear reinforcement. The group aimed for the rebar to be of realistic proportion to the bridge, therefore the altering of abutment dimensions was required. Changes in the dimensions resulted in changes in the substructure's dead load, and the reinforcement had to be redesigned. The best approach in redesign was to work from the top to the bottom of the substructure. Therefore, the reinforcement in the pile cap was calculated last and no dimensions needed to be altered.

The use of the *Excel* spreadsheets was vital in the success of the substructure's design. By using a master dimension and assumptions sheet and using the defined values for all calculations in the other sheets, it was simply a matter of changing one variable to alter the entire substructure's design. *Excel* saved the group a tremendous amount of time by eliminating the need to recalculate by hand all required values for moments, shears, and bearing capacities for each change in design.

### 5.3.2 Limitations

The main limitation in the design of the substructure was the assumption of the Quaboag River area's soil profile. The group was forced to consider the worst case scenario of an entire sandy soil profile, therefore creating a conservative foundation system. Excessive conservatism is essentially not cost effective, and would create higher construction and material fees in the bridge design. However, the reliance on conservative assumptions is considered ethical in this case since the group is confident the foundation will safely support the proposed bridge design.

Another major limitation in the project was conflicting sources of information. Since the Quaboag River Bridge Replacement Project is under much debate by the Town of Brookfield, many news articles about the replacement project have been published to appease the public of the new construction. Conflicting reports of the bridge's size were an obstacle for the group in the beginning of the project. The group made decisions on the bridge dimensions with the help of AASHTO and comparisons to similar MassDOT bridge projects.

Unfortunately, the group was unable to calculate the wing wall design due to time constraints. If the group were to design for a wing wall, the following design options would have been chosen.

- The wing wall would be designed as a straight wing wall, keeping a 90-degree angle between the abutment and the wing wall.
- Although there is a river that flows under to bridge, the wing wall does not need to be splayed because the bridge is not meant to guide the river under the bridge. The wing wall would be for structural stability purposes only.
- The wing wall would also be designed with reinforced concrete since the bridge is designed around a sandy soil profile and will most likely need the added strength but a plain concrete wing wall would be more economical.

### 5.3.3 Recommendations

In general, the group highly recommends future MQPs using *Microsoft Excel* for any lengthy equation and calculation process. Although this is a recommendation based off the substructure's design process, it can be applied to a number of other aspects in future projects.

Specifically it is recommended for this project to look into scour effects and protection. This was discussed in Chapter 2, but due to time constraints it was not addressed. The final deep pile substructure design is assumed to be far enough back and parallel to the river to mitigate the scour potential. One suggestion for scour protection is to research the possibility of steel H piles that would prevent the water from wearing away the concrete abutment walls and pile caps.

## 5.4 Cost Analysis

Like the superstructure and substructure aspects of the project, the cost analysis element also offered key findings, limitations, and recommendations to the project.

### 5.4.1 Key Findings

Life-cycle cost conclusions drawn from the *BridgeLCC* program include: the total LCC of both bridge designs, the bulk of costs lie in the initial construction of both designs, and the effect of the real discount rate on the LCCAs. These three conclusions would help an engineer determine which bridge design has the lower LCC.

The total LCC of the concrete bridge design predicted by *BridgeLCC* was $4.4 million in base year dollars. This is lower than the steel bridge design LCC approximation of $5.4 million. These LCCs were calculated over a 100-year study and did not take into account events that may occur, such as the 50-Year Flood event. Therefore these costs were based off the initial construction and the decommissioning fees of each bridge. Taking events, the inflation rate, and real discount rate into account, *BridgeLCC* approximated the concrete design to be $10 million and the steel design to be $19.5 million. With these LCCs the group concluded that the concrete design is economically better than the steel design over a 100-year period.

The bulk of the cost for each design differed slightly. Initial construction would be the most costly element in the concrete bridge's design, costing Massachusetts $3.7 million to fund. Initial construction of the steel bridge would also be costly, but to the Town of Brookfield at $3.5 million. However, the steel design would also require a $1.0 million rehabilitation project every 25 years, generating another $3.0 million in base year dollars that Brookfield would need to invest in the bridge over a 100-year life-cycle. Therefore, the bulk of expense in the concrete bridge would be in its initial construction, whereas the steel bridge would have nearly an equal amount of cost in initial construction and OM&R.

The real discount rate had an effect on the LCC of both bridges over a 100-year study. As shown in Table 44, in base year dollars the bridges would cost $4.4 and $5.5 million at the end of the study with a real discount rate of 2.3%. When the real discount rate was reduced to 1.3% (which is the commonly used rate for a 10-year study), the LCC of the concrete bridge in base year dollars totaled approximately $4.58 million whereas the steel design totaled $6.30 million. There was little to no change in cumulative costs in current year dollars when the real discount rate was changed as it only effects base year dollar analysis. Therefore, using a low real discount rate adversely heightened the LCCs of both designs over time, while using a higher rate lowered the LCCs over time. This is because the LCC predicts how much money must be set aside today to fund the bridge over its life-cycle. A higher discount rate means money today "appreciates" more quickly for tomorrow. It was concluded that the Office of Management and Budget's suggested 2.3% real discount rate is more beneficial to a LCCA than a 1.3% real discount rate.

**Table 44 - Effect of the Real Discount Rate on the LCC of Both Designs**

| Real Discount Rate | Total LCC of Concrete Design | Total LCC of Steel Design |
|---|---|---|
| 2.30% | $4.407 million | $5.442 million |
| 1.30% | $4.585 million | $6.300 million |

### 5.4.2 Limitations

Limitations of the cost analysis project mainly comprised of discrepancies between sources used on the project. For instance, some MassDOT sites gave the project an estimated $1.5 million value, while others gave $3.71 million. These values were found while running the steel versus concrete design evaluation. The group was forced to do additional research on which number was most accurate, and then make an educated decision on which one to use in the LCCA using *BridgeLCC*.

Other limitations included reliance on assumed values for construction costs and fees, since there were no average values readily available. Although all assumptions were based off of research and educated decisions, the assumptions did not make the cost analysis a completely accurate representation of the two bridge designs. However, since assumptions remained consistent between the designs, they did not have an impact on the outcome of which bridge would be most cost effective, as explained above.

### 5.4.3 Recommendations

The group recommends *BridgeLCC* as an appropriate program for future MQP groups to use. It is user-friendly and easy to learn for a project. Also, *BridgeLCC* provides highly detailed reports using input values for projects to quickly arrive to conclusions. The group also recommends researching construction fees more in depth for *BridgeLCC* to provide a more accurate cost analysis of a bridge project.

# Acknowledgements

We would like to acknowledge Professor Leonardo Albano in for his direction and assistance throughout the course of our project. He provided us with many helpful sources, design methods, equations, and bridge examples to help us complete our project. His help was invaluable and vital in the completion of our MQP. We would also like to thank Ian Jutras for creating our bridge design as a 3D model using *Solidworks*.

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## Appendix A: Proposal

WORCESTER POLYTECHNICE INSTITUTE

# Quaboag River Bridge, Brookfield, MA

A Major Qualifying Project Report

**Lauren D'Angelo, Mariah Seaboldt and Madison Shugrue**

## Abstract

This proposal describes both precast spread-box beam and steel girder designs for a single span bridge. The goal of this project is to evaluate these designs using cost, life expectancy, environmental impact, ethics, and timeline considerations in order to choose which one is the best option for the Accelerated Bridge Program Project No. B-26-002 located in Brookfield, MA. Once a design is determined, a 3D model will be developed with special attentions to cost and constructability.

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# 1. Introduction

This proposal summarizes our background of two possible bridge designs, our anticipated methodology and evaluation analysis of both designs, our expected deliverables, and our hypothesized conclusions. Our project will compare the cost, sustainability, environmental impact, ethics, and estimated timeline of a spread-box beam precast concrete and steel girder design for a single span bridge. This proposal outlines how we plan on choosing from those designs to create a 3D model while addressing cost and constructability concerns.

## 2. Capstone

### Economic
Our project is based on the Accelerated Bridge Project (ABP) B-26-002, located at 148 Fiskdale Rd, Brookfield, Massachusetts. To satisfy the Economic Capstone requirement, we plan to remain below the anticipated cost of this project set forth by the Massachusetts Department of Transportation (MassDOT) while designing our bridge model. We will do this by designing with locally available materials wherever possible.

### Environmental
The bridge our project models, extends over the Quaboag River and is surrounding by wetlands. We will consider any environmental impacts on the surrounding wetlands during our analysis of the decommissioning and construction processes of the bridge. We will also consider the environmental impact associated with the proposed construction materials.

### Sustainability
To design our model for sustainability, we will use certain materials to increase the life expectancy of our design. Thinking in terms of sustainability, lower costing yet durable materials will make for a better investment, as the bridge design will last longer.

### Manufacturability
The bridge construction project we are modeling our project after is part of the Accelerated Bridge Project (ABP) Program. We will use local materials wherever possible to design our model as well as attend to any existing site conditions, such as scour, that may set limits on the manufacturability of our design.

### Ethical
We will design our bridge model to AASHTO LFRD Design Specifications loading standards. This will allow our design to be safe for the bridge's intended use: vehicular and pedestrian travel. We purposely included ethical criteria in any type of evaluation process we use in our methods.

### Health and Safety
To complete the health and safety portion of our Capstone requirement, we will reflect on worker and public safety in our evaluation of the decommissioning of the existing bridge.

### Social
Land owners, with property adjacent to the bridge replacement, are against a new design replacement project. We will address their concerns in the background research of our project. Also, we will design our model to be sustainable, having a long life expectancy. This will lessen future construction inconveniences to the business owners.

### Political
In the background of our project we will bring attention to the issue that if Brookfield does not approve the state's design, the town is then responsible for designing and replacing the bridge itself without federal aid.

# 3. Background

## 3.1 Introduction

This background provides information about the existing and proposed Quaboag River Bridge that our project is based on. It will also explain a precast spread box beam girder design as well as a steel girder design which are possible options for this bridge replacement project. Additionally, it explains the deck components, the substructure, the decommissioning and construction, and the possible evaluation criteria and cost analysis options for a bridge design.

### 3.1.1 Accelerated Bridge Program

Since 2008 the MassDOT and DCR have been working on inspecting and rating over 4,500 bridges in the state of Massachusetts to determine if they meet federal standards. These inspections are part of the Accelerated Bridge Program (ABP) and comprise of examining vital bridge elements such as the deck, superstructure, substructure, and rate of deterioration. In Massachusetts a bridge is considered structurally deficient if it is rated to be in a condition of 4 or less on a scale of 0-9 (0 being the lowest rating, 9 being the highest). “Structurally deficient” does not necessarily mean that a bridge is unsafe, though it does mean that the bridge has the potential of becoming unsafe to transportation if its deficiencies are not repaired or attended to. Currently the ABP had identified over 500 bridges as structurally deficient. Once a bridge is identified as structurally deficient, it is classified as a preservation, rehabilitation, or replacement project. Preservation projects are the least costly and have the shortest design and construction period whereas rehabilitation and replacement projects involve the replacement of major bridge elements and are more design and cost extensive. (“Accelerated Bridges” 2012)

### 3.1.2 Bridge No. B-26-002

The ABP bridge replacement project for Bridge No. B-26-002, is located at 148 Fiskdale Road in Brookfield, Massachusetts (Figure 1). This bridge runs over the Quaboag River and does not meet the 50 Year Flood Plain requirements. It needs to be raised to a minimum height of 6 feet from the Quaboag’s water level to pass federal standards. The proposed Quaboag bridge design provides 6 feet of clearance under the bridge, shifts the bridge easterly on its northern end, and eliminates a dip between itself and an existing CSX Bridge (Figure 2), while aesthetically matching the existing roadway connecting the CSX Bridge. This proposed design improves visibility and vertical geometry of the roadway. A comparison of the existing and proposed bridge design elements of the Quaboag River Bridge can be viewed in Table 1. (Depaola & Broderick 2012)


**Figure 1: Location of Bridge – Brookfield, MA**

**Figure 2: Dip Between Existing Bridge and CSX Bridge**

**Table 1: Existing and Proposed Design Elements**

| Design Element | Existing Quaboag River Bridge | Proposed Bridge |
|---|---|---|
| *# of Girders* | 5W x 4L | N/A |
| *# of I-Beams* | 6 | 6 Type NEBT 1200 Beams |
| *# of Spans* | 2 | 1 of 98' |
| *Abutments* | Concrete Gravity abutments | Full Height Concrete Abutments |
| *Approaching Roadway Shoulder* | N/A | 4' |
| *Approaching Roadway Width* | 24' | 32' |
| *Construction Period* | Completed in 1936 | 2012-2013: 2 construction seasons |
| *Cost* | N/A | $8.35 mil |
| *Drainage System Elements* | N/A | Replaced for the new roadway profile |
| *Height from River's Average Water Height* | 5.5' | 6' |
| *Length* | 79' | 98' |
| *Loading Tolerance* | 18 tons for 2 axle; 22 tons for 3 axle; 34 tons for 5 axle | HL-93 loading in accordance with current AASHTO LRFD and MassDOT standards |
| *Pier Elements* | 1 Center Pier | Center pier will be removed to streambed level |
| *Scouring Elements* | N/A | Old Abutments will remain as a scouring element |
| *Sidewalks* | 6'W E side of bridge; 1'W safety-walk W side of bridge | Two 5.5' W sidewalks on either side of the bridge |
| *Slope Stabilization* | N/A | Modified riprap |
| *Steel Railing* | both sides of bridge | both sides of bridge: S3-TL4 Bridge Rail |
| *Superstructure Material* | N/A | Precast Concrete girders: Boxed Tee or spread box beam stringers with 8in HP 4000psi concrete deck and 3.5in hot mix-asphalt wearing surface |
| *Utilities* | N/A | Will be designed to accommodate a possible future water line |

As shown in Table 1, the major design changes of the bridge are:

- type of superstructure design and material
- extension of the bridge's length
- widening of the bridge
- removal of the center pier
- placement of the new abutments

The details concerning these major design changes and their options are discussed later in the background. Other design elements, such as the type of railings, loading tolerances, and sidewalk design are straight forward and follow Massachusetts state regulations. These changes are due to changes in the utility of the bridge. For example, larger trucks are now using the bridge than when it was constructed in 1936 so the loading tolerances must be changed to accommodate this new traffic. Also, there is more pedestrian traffic on the bridge because in this area the Quaboag River is a popular fishing location. Therefore railings will be type S3-TL4 and the sidewalks are designed to be 5.5’ on both sides of the bridge. These design changes will meet safety regulations for the foot-traffic using the bridge (“Accelerated Bridges” 2012).

The Quaboag Bridge crosses a section of the river that is surrounded by a wide marsh with no clearly defined banks. There is Trout Brook immediately upstream of the river and the river dumps into Quaboag Pond downstream. This area of the river receives a significant amount of recreational river traffic and visitors during the fair to nice weather seasons. Directly north-east of the bridge is White’s Landing, a small Mom and Pop business. White’s Landing can accommodate approximately 5 vehicles in its gravel parking lot and has a small boat launch for its customers east of the bridge. Images of the Quaboag Bridge’s surrounding area can be viewed below (Figures 3 and 4).

**Figure 3: Upstream from the Bridge**

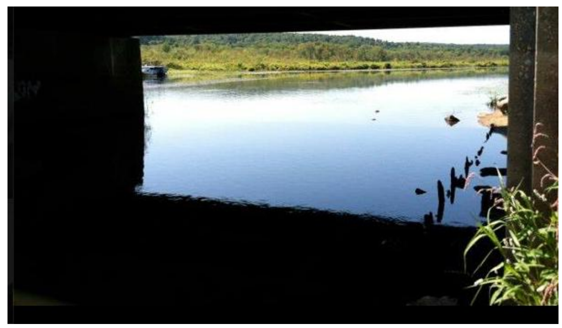


**Figure 4: Downstream from the Bridge**

The marsh and lands adjoining the bridge are owned by the Massachusetts Department of Fish and Game, Division of Conservation and Recreation, the Consolidated Rail Corporation, and the business owners of White’s Landing (Depaola & Broderick 2012). Due to the ownership of the surrounding area and the wetlands, Brookfield needs to obtain the correct documentation to move forward with this project. These document requirements, which can be found on the MassDEP website, include wetland permits, wetland transmittal forms, and water quality certifications (MassDEP 2012). Brookfield also needs to impose permanent and temporary easements on White’s Landing to accommodate the bridge construction (Depaola & Broderick 2012). This has created a debate between the town and White’s Landing owners, significantly slowing the design stage (“Quaboag bridge project to go…” 2012).

White’s Landing owners want an identical bridge superstructure to the existing bridge so it does not affect their business, while the state wants a longer lasting structure erected due to economic concerns. The similar steel superstructure design would last potentially for 25 years before needing attention or repair (“Quaboag bridge project to go…,” 2012). It would also not require permanent easements on White’s Landing, whereas a longer lasting structure would. A public hearing was held on May 15, 2012 to inform the community of the state’s proposed design and its considerations. The design was not passed at a Brookfield town meeting on June 8, 2012 and was scheduled to be reviewed again on September 7, 2012. Brookfield must pass the state’s design at its next town meeting or the state will abandon the project and Brookfield will need to design and repair the bridge without state resources (“Quaboag bridge project to go…” 2012). In a public hearing, the state provided a rough sketch of a proposed design for a precast spread box beam girder bridge that has a life expectancy of 75 years, shown in Figure 5 (Depaola & Broderick 2012).

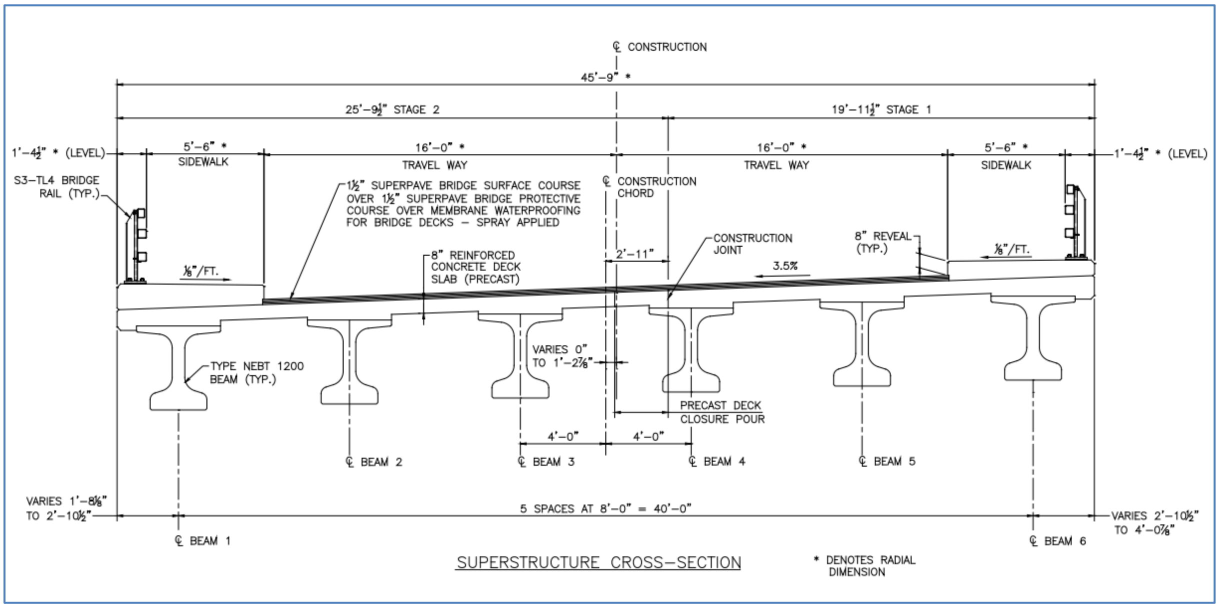



**Figure 5: Sketch of Preliminary Spread Box Beam Design Cross-Section**

## 3.2 Spread Box Beam Bridges

As discussed in the previous sections, the proposed design for the replacement of the Quaboag River Bridge is spread box beam design (Federal Highway Administration 2006). This type of design will be used as one of the comparative designs for the capstone portion of the project. The following subsections will discuss aspects of this type of design and the main components that make up a spread box beam bridge. A later section will elaborate on a steel girder bridge, which will be the other bridge design used for comparative purposes.

### 3.2.1 Beam Design

A spread box beam bridge is made up of prestressed concrete beams known as box beams. The beam has a rectangular cross section with a rectangular void through the center. A typical box beam cross section is depicted in Figure 6 below (Federal Highway Administration 2006). Typical span lengths for box beams vary between 20 and 90 feet. Often the cross-sectional width of the beams is either 36 or 46 inches, the depth can range from 27 to 42 inches and the thickness of the web wall varies between 3 and 6 inches (Federal Highway Administration 2006).

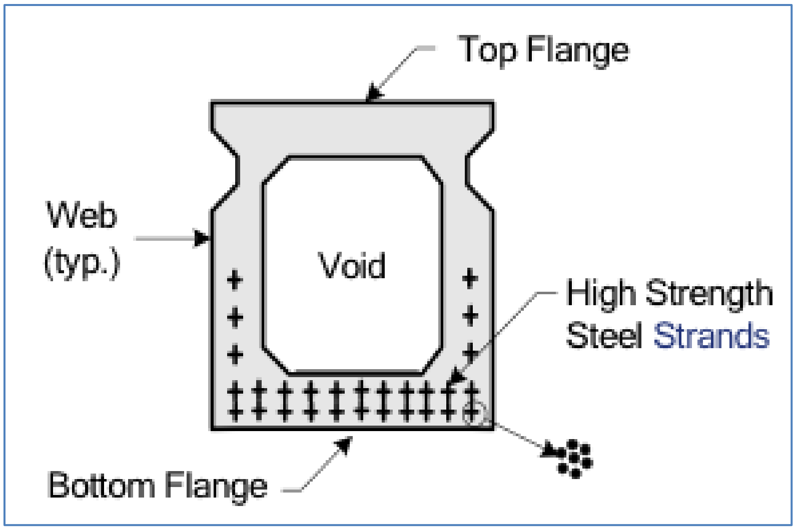



**Figure 6: Box Beam Cross-Section**

Box beam bridges can be designed in one of two ways. The design proposed for the replacement of the Quaboag River Bridge is known as a spread box design (Federal Highway Administration 2006). The design of this bridge uses box beams spaced across the width of the bridge, as shown in Figure 7 below. Box beams can also be used in the design of adjacent box beam bridges. As the name implies, these bridges have box beams placed next to each other with no spacing between them (Federal Highway Administration 2006).

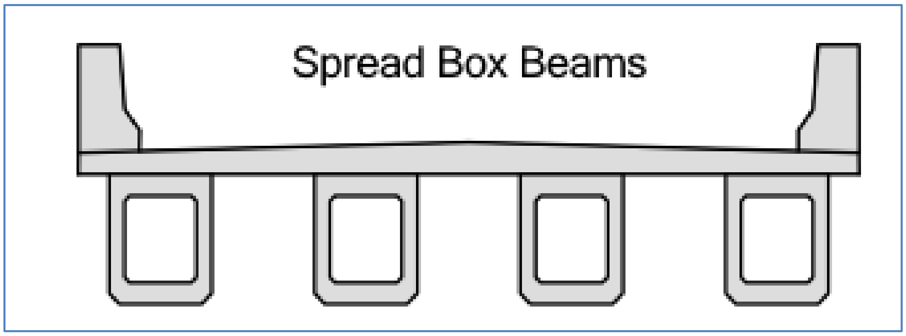



**Figure 7: Spread Box Bridge Layout**

In spread box beam bridges the box beams are usually spaced between 2 to 6 feet apart. These bridges also may involve diaphragms that are placed at the midpoints, endpoints or at distances one third the length of the span. If the diaphragms are placed at the endpoints of the span, they will be located at the abutments or piers. The diaphragms can be made out of cast-in place concrete, precast concrete, or steel (Federal Highway Administration 2006).

### 3.2.2 Steel Reinforcement Placing

Figure 6 on the previous page illustrates typical placing for steel reinforcing bars in box beams. Common strand sizes for reinforcements used in box beams are 1/4, 3/8, 7/16 and 1/2 inch diameter steel rods. Strands are normally spaced every 2 inches, but both strand size and spacing can vary depending on concrete characteristics (Federal Highway Administration 2006).

Other steel placement in box beams can occur if the beams are to be used as composite beams. Composite beams have additional steel stirrups that are placed at the top of the beam and extend out of the concrete. The purpose of these stirrups is for when the cast-in place concrete deck is placed, the steel will reinforce both the deck and the beam. Figure 8 illustrates the placement of the steel stirrups as well as possible placement of secondary reinforcing strands (Federal Highway Administration 2006).

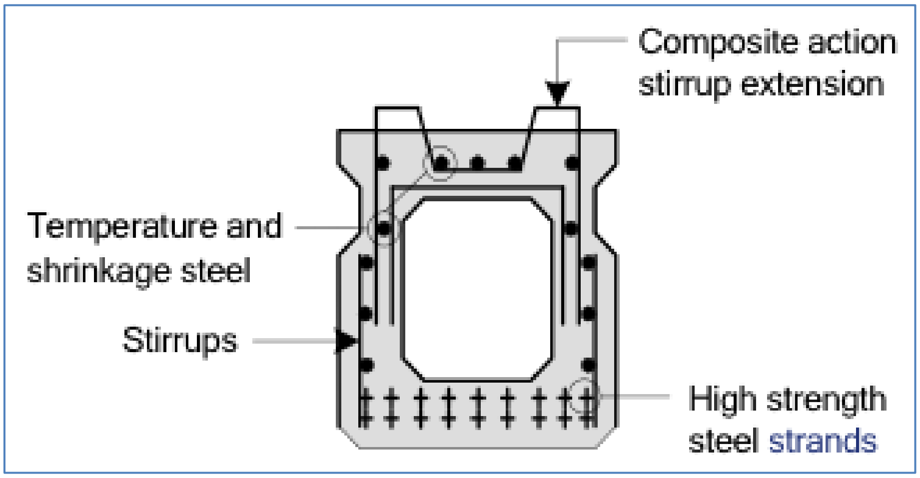



**Figure 8: Composite and Secondary Steel Placement**

### 3.2.3 Advantages and Disadvantages of Box Beams

The use of box beams and other precast bridge components can affect the bridge and construction process in both positive and negative ways. The physical design of the box beam results in a beam that is both strong and light. Since the center of the beam is hollow there is a reduced amount of dead load experienced by the structure. However, the beam still supports design moments and shears (Federal Highway Administration 2006). The depth of the box beam also allows it to be used when there are low clearances between the bridge and whatever it is spanning. Typically these beams have a maximum depth around 42 inches, making them a viable choice for this type of situation (Federal Highway Administration 2006).

Box beams can also save time during the construction process. Since they are precast concrete structures, they can be manufactured off site, which allows other construction or planning processes to take place while the beams are being built. Once the beams are built they can be installed quickly with minimum disruption of traffic (Federal Highway Administration 2006).

Although these beams can be advantageous in certain situations, there are some disadvantages to their use. The concrete beam can experience cracking due either to flexure, shear, or temperature changes. Shrinkage and delamination can also occur between the concrete and reinforcing bars. The concrete used in the box beams is also subject to spalling, efflorescene, collision damage, overload damage, and general wear and abrasion damage. Corrosion of the steel reinforcing strands can also cause problems within the beam, leading to loss of tensile strength and other issues (Federal Highway Administration 2006).

Table 2 below summarizes the advantages and disadvantages discussed above for precast concrete box beams. Much more detail can be provided for box beams and spread box bridges, but the focus of this section is to provide an overview of the design. A similar overview of steel girder bridges will be provided in the next section.

**Table 2: Advantages and Disadvantages of Precast Concrete Box Beams**

| Advantages | Disadvantages |
|---|---|
| Center void reduces dead load | Concrete cracking |
| Holds up to design shear and moments | Delamination |
| Allows for low levels of clearance | Spalling, efflorescene, wear and abrasion damage |
| Save time during construction | Collision and overload damage |
| Can reduce traffic disruption during construction | Corrosion of steel reinforcements |

## 3.3 Steel Box Girder Bridge

A steel box girder bridge design is the second comparative design that will be used for the capstone portion of the project. The following subsections will discuss the configuration options, primary and secondary members, stiffeners, fatigue and fracture critical areas, and the deck interaction of steel box girder bridges.

### 3.3.1 Steel Box Girder Design

Steel box girder bridges are supported by one or more welded steel box girders. Steel box girders can have either rectangular or trapezoidal cross sections (Figure 9). The cross section consists of two or more web plates connected to a single bottom flange plate. There are two span options for box girder bridges: simple spans of 75 feet or more or continuous spans of 100 feet or more (Federal Highway Administration 2006).

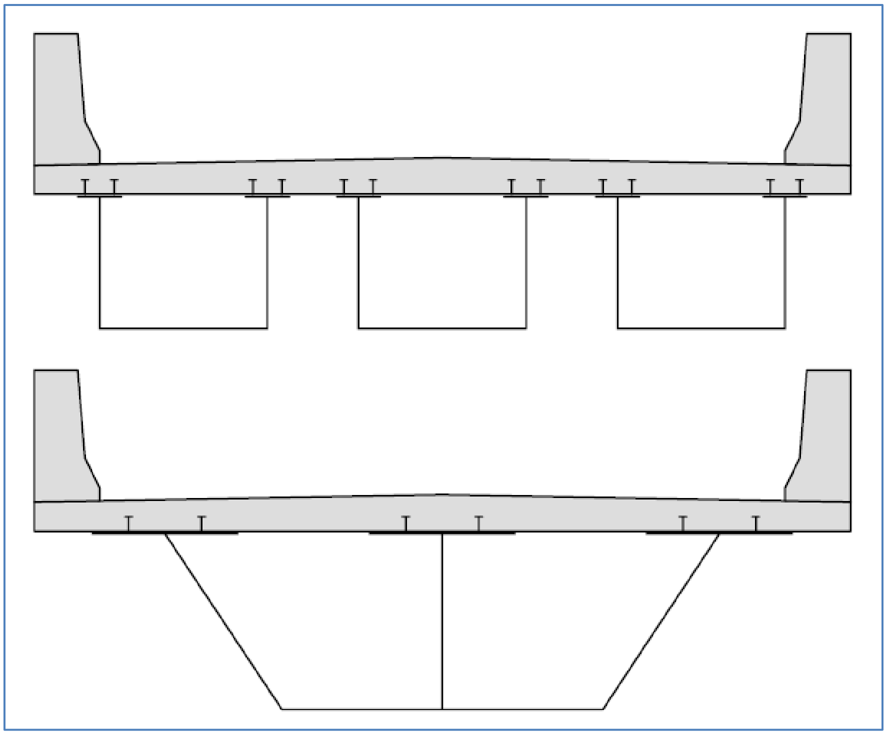


**Figure 9: Steel Box Girder Rectangular and Trapezoidal Cross Sections**

A steel box girder bridge can be designed with a single box configuration or a spread box configuration. Factors such as deck width, span length, terrain, and aesthetics need to be considered when determining the configuration. The primary members of box girder bridges are the box girders and all internal bracings. Diaphragms are primary members for curved box girder bridges and secondary members for straight box girder bridges. Diaphragms can consist of solid plates, rolled shapes, or cross frames constructed with angles, tee shapes, and plates (Federal Highway Administration 2006). They can also be on the interior or exterior of the box. Exterior diaphragms are typically used between box girders on multi box bridges.

Since steel box girder bridges are susceptible to bucking from torsional and shear forces, stiffeners must be used in areas of compression for the webs and bottom flange of large box shapes. The purpose of stiffeners is to increase the stability of the box girder by limiting the unsupported length of the web and bottom flange (Federal Highway Administration 2006).

Fatigue and fracture are two types of failure that need to be accounted for in the design of a steel box girder bridge. Fatigue prone areas are: welded attachments inside the box, attachment welds in the tension zone, butt welds in adjacent longitudinal stiffeners, intersecting welds between webs and flanges, and field splices. Fatigue cracks can also result from web-gap distortion and out-of-plane distortion. The box girders are considered fracture critical members of box girder bridges when a span has two or less box girders making the structure nonredundant (Federal Highway Administration 2006).

Steel box girder bridges have two deck options: a composite deck or an orthotropic steel plate deck. A composite deck consists of the top flange plates incorporating shear connectors and a composite superstructure with a concrete deck. When using a composite deck, the deck and the superstructure work together to carry the live load (Figure 10). On the other hand, an orthotropic steel plate deck is comprised of a top flange consisting of a single plate extending the width of the box and a wearing surface on the top flange (Figure 11). Further detail of bridge decking materials will be provided in the next section.

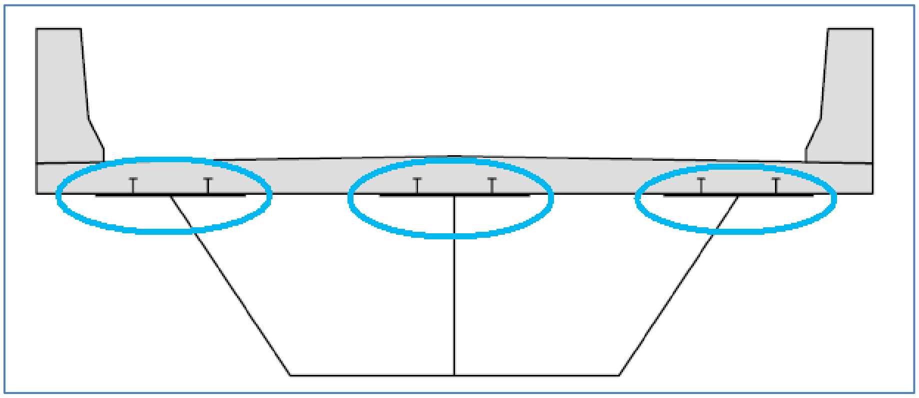


**Figure 10: Box Girder Cross Section with Composite Deck**

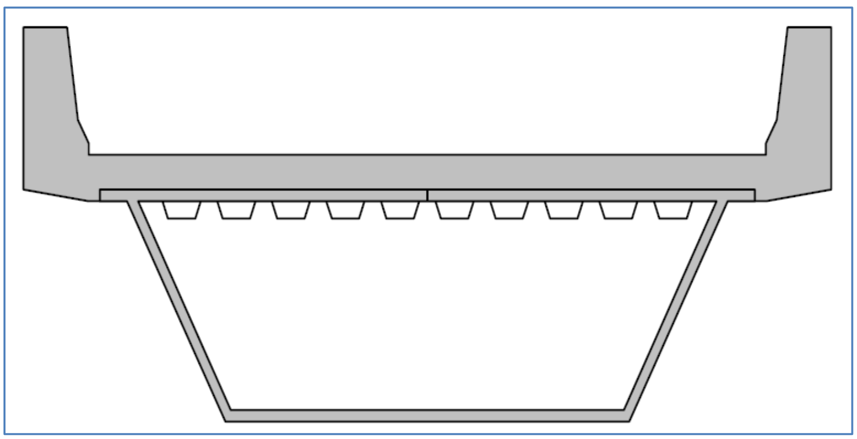


**Figure 11: Box Girder Cross Section with Orthotropic Steel Plate Deck**

## 3.4 Bridge Decks

In this section we will discuss the three types of bridge decks: timber, steel and concrete. First, an overview will be provided of timber and steel decks. Typical designs, wearing surfaces, protective coatings and some advantages and disadvantages will be briefly discussed for the steel and timber decks. However, concrete decks will be discussed in more detail, since they are the most common type of bridge deck material.

### 3.4.1 Timber Decking

Timber is rarely used in bridge structures and decking. About 7% of all bridges in the National Bridge Inventory are timber and of steel bridges, only 7% of these bridges have timber decking (Federal Highway Administration 2006). Even though timber isn't used very often in bridge design, there are still some advantages and disadvantages to its use. This section will give an overview of timber decking and why it may be used in certain situations.

#### 3.4.1.1 Types of Timber Decking

There are multiple different types of timber decking to choose from (Federal Highway Administration 2006). The following are the four main types of timber decking:

- Plank decks
- Nailed laminated decks
- Glued-laminated (Glulam) deck panels
- Stressed-laminated deck panels
- Structural composite lumber decks

These different types of timber decking utilize varying placement of wood along the bridge deck as well as different material make-ups. For example, plank decks and nailed laminated decks are made of timber planks, whereas glue-laminated, stressed-laminated and structural composite lumber are made up of composite wood pieces held together by an adhesive (Federal Highway Administration 2006). Some of the deck types have members running transversely across the bridge while others run longitudinally (Federal Highway Administration 2006). Some types, like Glulam, come in panels as opposed to planks (Federal Highway Administration 2006).

#### 3.4.1.2 Wearing Surfaces

In addition to the different types of timber decking available there are also different wearing surfaces used in conjunction with timber decking. In some instances the wearing surface is another layer of timber that is placed along the projected wheelpath of traffic (Federal Highway Administration 2006). Other wearing surfaces include bituminous (asphalt) mixtures and concrete. However, bituminous mixtures and concrete are not used for certain types of timber decking because the deck can crack when the decking members deflect (Federal Highway Administration 2006).

#### 3.4.1.3 Protective Coatings

Wearing surfaces provide some protection to timber decks, but additional protective systems must be used to prevent decay (Federal Highway Administration 2006). Water repellants, preservatives, fumigants, fire retardants and paint are all protective coatings that can be used on timber decking. Most of these coatings help prevent decay of the wood, but fire retardants have the important function of slowing the spread of fire through the decking. Steel decks and concrete decks do not utilize fire retardant coatings, but they do also need protection from decay and corrosion (Federal Highway Administration 2006).

#### 3.4.1.4 Advantages and Disadvantages

Even though timber decks are not widely used, they do have certain advantages. Although timber does decay, it is resistant to deicing chemicals, which will harm concrete and steel decks (Federal Highway Administration 2006). Timber is also a renewable material, easy to fabricate and lightweight, which helps reduce dead load (Federal Highway Administration 2006).

However, there are also some disadvantages to timber, which may be why it is not as commonly used. As mentioned above the fire hazard presented by timber is more than that of steel or concrete. Timber is also susceptible to insect, fungus and parasite damage and may deflect or split (Federal Highway Administration 2006). Other possible disadvantages include:

- Checks
- Shakes
- Loose connections
- Surface depressions

- Chemical attack

Timber decking is not very widely used in bridge construction, however in certain situations it may be the right choice for design and material.

### 3.4.2 Steel Decking

Steel decking is used more frequently than timber decking, but is still not as widely used as concrete decking. Steel decks are often found on older bridges and may be used for rehabilitation of bridge decks or for bridges with very long spans (Federal Highway Administration 2006). Steel decks are also often used if dead load is a concern in a bridge design because steel decks weigh less than concrete decks (Federal Highway Administration 2006).

#### *3.4.2.1 Types of Steel Decking*

Each type of steel decking utilizes different designs. Some designs leave the steel members of the deck and superstructure exposed, while others can be partially or fully filled with concrete. The four main types of steel bridge decks are (Federal Highway Administration 2006):

- Orthotropic decks
- Buckle plate decks
- Corrugated steel flooring
- Grid decks.

Grid decks can be subcategorized as: welded grid decks, exodermic grid decks, and riveted grid decks (Federal Highway Administration 2006).

#### *3.4.2.2 Wearing Surfaces*

Like timber decks, steel decks can have different wearing surfaces. If grid decks are open (not filled with concrete) then the serrated edges of the grating act as the wearing surface. If a steel deck is filled with concrete then the concrete acts as the wearing surface. Otherwise asphalt may be used, especially in orthotropic decks (Federal Highway Administration 2006).

#### *3.4.2.3 Means of Protection*

Steel decks must be protected from wear and corrosion because they are often more exposed to weather and loading conditions than other types of decking and sometimes leave the superstructure exposed as well (Federal Highway Administration 2006). A variety of paints can be used to protect steel from moisture, oxidation, and chlorides. Paint is usually applied as a primer, intermediate coat, and then topcoat. The steel can also be protected through a galvanization process, which coats the steel in zinc. Galvanized steel will not corrode as fast because the outer coating of zinc will corrode instead of the steel. Some steel decks are also protected through an epoxy coating that will help protect them from corrosive elements. However, epoxy coatings on steel decks are rare (Federal Highway Administration 2006).

#### *3.4.2.4 Advantages and Disadvantages*

As with timber and any other type of design, there are disadvantages and advantages to the use of steel decks. As mentioned above, steel is lighter in weight than concrete decks and as a result the superstructure can withstand more of live load. However, since a steel deck often leaves the superstructure exposed, it can result in more corrosion and a shorter lifespan of both the deck and bridge. In addition to corrosion, the other main structural disadvantages to steel are fatigue cracks and other stress cracks (Federal Highway Administration 2006).

### 3.4.3 Concrete Decks

The most common type of bridge decking is a concrete deck. Concrete can be formed into multiple different shapes and as a result, it can be used effectively in many different types of bridge construction (Federal Highway Administration 2006). Concrete decks can also be composite or non-composite. If composite, the deck is physically joined to the superstructure, creating a stronger structure. Non-composite decks are not joined to the superstructure resulting in a weaker structure (Federal Highway Administration 2006). The following subsections will discuss the types of concrete decks currently in use as well as wearing surfaces, protective systems and other aspects of the decks.

#### *3.4.3.1 Concrete Deck Types*

Concrete decks can be broken up into two main types: reinforced cast-in-place and precast. Precast concrete decks include normal precast panels, prestressed precast panels and prestressed precast panels with cast-in-place top (Federal Highway Administration 2006).

Cast-in-place concrete decks are made onsite by pouring wet concrete into either permanent or temporary forms. Temporary forms are usually made of wood, while permanent, or stay-in-place, forms are made of corrugated metal. Before being poured into the forms, steel reinforcing bar (rebar) is laid. The concrete is then poured over these bars and when it cures, the deck is in place (Federal Highway Administration 2006). The purpose of rebar is to help increase the tensile strength of the concrete. Without this rebar concrete would have very weak tensile strength and would not be a good choice for bridge decking. The primary reinforcing bars are laid so that they will be at the top and bottom of the concrete deck. Primary bars are what will carry the main tensile stress developed within the concrete. Secondary bars are placed perpendicular to the primary reinforcement and will mainly be carrying stresses developed as a result of temperature changes and shrinkage (Federal Highway Administration 2006).

Precast deck panels are also made of reinforced concrete, but instead of being formed onsite, they are formed and cured offsite. The panels are then brought to the construction site and put in place when necessary. They are attached to the superstructure with either mechanical clips or shear connectors. Prior to being attached though, the panels are leveled using leveling bolts or grout (Federal Highway Administration 2006).

Prestressed precast panels are constructed in the same way as regular precast panels. However, prestressed panels also have prestressed steel reinforcement. Tension is applied to the bars before the panels are formed and are held in tension until the concrete has cured. As a result, once the panels are formed, the tensioned bars are exerting compression forces on the concrete itself. This helps to reduce the amount of cracking experienced by the concrete (Federal Highway Administration 2006).

Prestressed precast panels with a cast-in-place top are simply prestressed panels that have been put in place as the bridge deck and are then overlaid with a cast-in-place top. The panels act as forms and the cast-in-place overlay becomes composite with both the deck and the superstructure (Federal Highway Administration 2006).

#### *3.4.3.2 Other Similar Deck Materials*

Two newer deck materials are being used in similar ways as concrete decks. Fiber reinforced polymer (FRP) uses glass fibers as reinforcement in polyester or vinyl ester resins. Similar to precast panels FRP decks are usually formed in panels at a factory offsite and are then shipped to the construction site. The panels are then put together using adhesives and then attached to the superstructure. FRP decks can be compositely attached to the superstructure through the use of grout (Federal Highway Administration 2006).

Fiber reinforced concrete (FRC) is also another new type of bridge deck material. In FRC decks, Portland cement is combined with polypropylene fibers. The addition of these fibers helps reduce cracking of the concrete due to shrinkage and increases the impact strength of the concrete once it's cured. Steel reinforcing bar may or may not be used when using FRC to create a bridge deck (Federal Highway Administration 2006).

#### *3.4.3.3 Wearing Surfaces*

As with steel and timber decking, concrete decks also utilize wearing surfaces. Either a concrete or asphalt wearing surface is normally used for concrete decks. Concrete wearing surfaces can either be integral or overlay. An integral concrete wearing surface is cast with the deck. Once this integral surface has worn down it is replaced with an overlay (Federal Highway Administration 2006). Overlay concrete surfaces are cast after the concrete deck is in place. Some overlay types are:

- Low slump dense concrete (LSDC)
- Latex modified concrete (LMC)
- Lightweight concrete (LWC)
- Fiber reinforced concrete (FRC)

Each concrete overlay has different characteristics and is used for different reasons (Federal Highway Administration 2006). FRC was discussed in the previous subsection, but is often used as a deck surface in order to prevent cracking. LSDC has a low water to cement ratio and as a result cures very quickly. LSDC is so dense that it doesn't allow penetration by chlorides and can be effective in areas where deicing products are used. However, it is subject to cracking so a LSDC overlay must be resurfaced after about 25 years (Federal Highway Administration 2006).

LMC is a mixture of Portland cement and latex solids. LMC is more expensive to make than LSDC, but is easier to lie (Federal Highway Administration 2006). The addition of latex into a cement concrete mixture reduces the amount of water needed in the mixture. With less water necessary, the resulting concrete has a high compressive strength, meaning it will experience less cracking and be more resilient against corrosive agents like water and chlorides (BASF Corporation 2011).

LWC incorporates lighter aggregates within its mixture and has a higher amount of entrained air. As the name suggests this makes for a significantly lighter product, reducing the dead load experienced by the structure. LWC is not only used in overlays, but in precast and cast-in-place bridge decks as well (Federal Highway Administration 2006).

In addition to using concrete surfaces, asphalt is often used atop concrete bridge decks. Asphalt layers can be between 1 and 2.5 inches thick and may be placed after a waterproof membrane is laid on the deck. This membrane helps prevent the penetration of corrosive agents into the concrete (Federal Highway Administration 2006).

#### *3.4.3.4 Means of Protection*

The main protection for concrete decks involves preventing the steel reinforcing bars from corroding. Sealants can be placed atop concrete decks to help stop chlorides from penetrating the deck and corroding the steel. Common sealants include boiled linseed oil and elastomeric membranes (Federal Highway Administration 2006).

Some steel reinforcing bars are made to help prevent corrosion and therefore deterioration of concrete decks. Bars with an epoxy coating will resist corrosion from chemicals and moisture and as discussed in steel decking, some steel bars undergo a galvanization process. Stainless steel bars and fiber reinforced polymer bars are also sometimes used for reinforcement because they do not corrode. Fiberglass reinforced polymer bars are significantly lighter than steel bars and as a result, may also be used in concrete decks (Federal Highway Administration 2006).

Waterproof membranes can be used to protect concrete decks and prevent corrosion of steel reinforcing bars. Two types of membranes are self-adhering membranes and liquid waterproofing membranes. These membranes will help reduce cracking of the concrete and penetration of water into the deck (Federal Highway Administration 2006).

#### *3.4.3.5 Advantages and Disadvantages*

Concrete decks are the most commonly used bridge decks for a reason. They can be molded to fit many different shapes and sizes and sometimes can be cast offsite, saving construction time. There are also many different types of concrete to choose from, allowing a designer to pick and choose the option that best works for a given project (Federal Highway Administration 2006).

However, there are some disadvantages to the use concrete that must be taken into consideration. The main disadvantage comes from the corrosion of the steel reinforcing bars that are used within the deck. The corrosion of these bars can result in a loss of tensile strength within the deck. The deck is also subject to cracking, scaling, spalling and other problems associated with environmental exposure of the concrete (Federal Highway Administration 2006).

#### *3.4.3.6 Deck Conclusions*

Regardless of the deck a designer chooses, there will be advantages and disadvantages involved. Although concrete decks are most commonly used in bridge design, there may be instances where steel or timber decks are desired. However, concrete decks have proven to be the most versatile of designs, even though there are certain disadvantages that come along with their use.

## 3.5 Substructure

The substructure of a bridge supports all the elements of the superstructure. Its purpose is to transfer the loads from the superstructure to the foundation, soil, or rock (Rossow 2007). The main elements of the substructure are the foundations, abutments, retaining structures, scour protection, piers, and bearings.

### 3.5.1 Foundations

All foundation designs must meet three requirements:

4) Provide adequate safety against any structural failures
5) Provide adequate bearing capacity of the soil beneath the foundation with a specific factor of safety design
6) Achieve acceptable total or differential settlements under the working load.

There are two types of foundations, shallow and deep. Shallow foundation classifications include spread footings, strap footings, combined footings and mat or raft footings. These types of foundations provide support entirely from their bases. Deep foundations classifications include piles, shafts, caissons, anchors, and spread footings. These foundations occupy relatively smaller surface ground areas and can usually take larger loads than shallow foundations (Chen & Duan 2000).

### 3.5.2 Abutments

Abutments are located at the end of a bridge, to provide end support of the superstructure and to retain the approaching roadway embankment. They are classified according to their locations with respect to the approaching roadway embankment. Common abutment types are full height, stub, open, and integral abutments. Figures 12-15 display these different types of abutments. Abutments are typically constructed with one or more of the following materials: plain cement concrete, reinforced concrete, stone masonry, steel, or timber (Rossow 2007).

Full and stub abutments are used for bridges with shorter spans or if there are issues with the surrounding terrain. Stub abutments are may be used to keep abutments away from the roadway or waterway. They also reduce the cost of the substructure, but increase the cost of the superstructure (Rossow 2007).

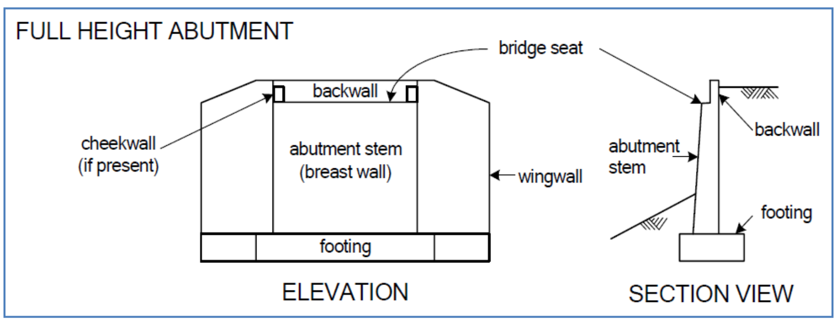



**Figure 12: Full Height Abutment Elevation and Section Views**

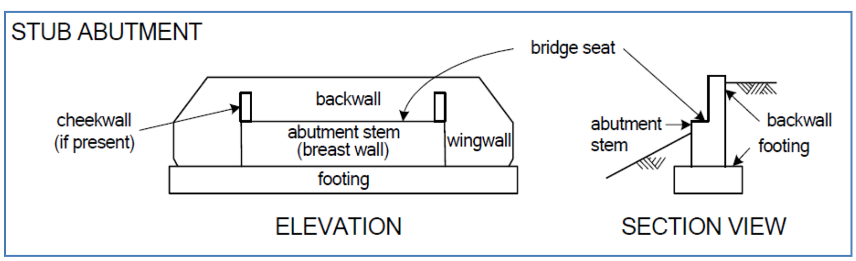



**Figure 13: Stub Abutment Elevation and Section Views**

Open abutments, also referred to as spill-through abutments, have the approaching roadway embankment extend on a slope between the bridge seat and "through" the supporting columns. The topmost part of the embankment is actually retained by the abutment cap (Rossow 2007). Open abutments generally have lower cost since most of the massive construction and heavy reinforcement of the substructure is eliminated. An additional advantage to open abutments is that they have the potential to be converted into a pier if more spans needed to be added to the bridge over time (Rossow 2007). However, excessive erosion or scour may occur over the time around the abutment, creating the abutment to eventually fill (Rossow 2007). Open abutments are discouraged near stream and river beds as they are susceptible to erosion (Rossow 2007).

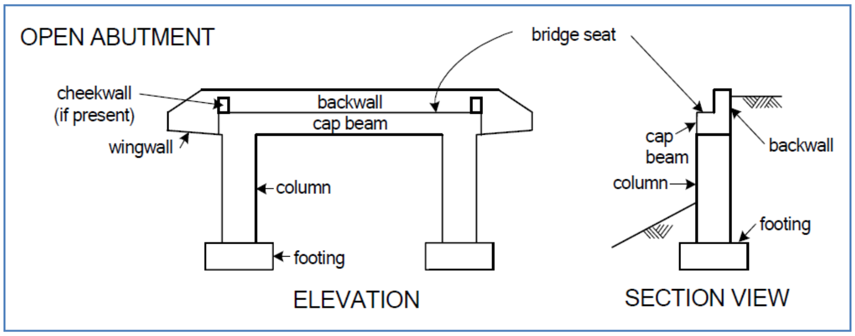



**Figure 14: Open Abutment Elevation and Section Views**

In some cases bridge design fails because the superstructure and substructure's expansion devices work improperly. Integral abutments are becoming more popular to eliminate this expansion failure (Rossow 2007). The superstructure and substructure are integral components and act as one unit without the use of an expansion joint. Relative movement is instead accommodated by the pavement joints and approaching roadway slabs (Rossow 2007). Although an advantage of integral abutments is that they lack bearing devices and joints that require maintenance, they have the disadvantage of frequent cracking due to settling and over compaction of backfill (Rossow 2007).

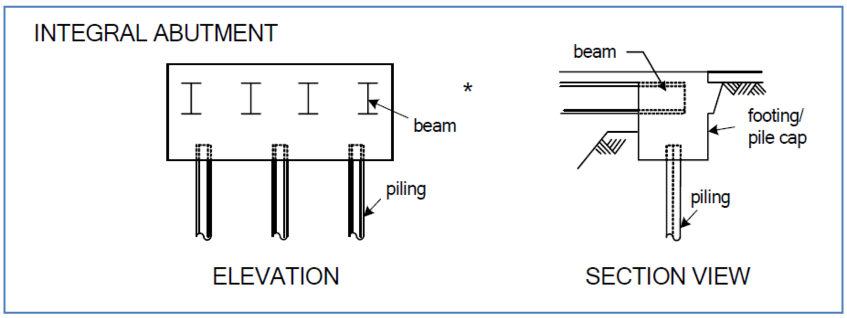



**Figure 15: Integral Abutment Elevation and Section Views**

Abutment elements are shown in Figures 12-15. Below, Table 3 details what the primary elements' roles are in abutment design.

**Table 3: Abutment Elements (Rossow 2007)**

| Element | Description |
|---|---|
| Bridge Seat | provides a bearing area that supports the superstructure |
| Backwall | retains the approaching roadway; provides support for the approach slab |
| Footing/Pile Cap | transmits weight of the abutment to the supporting soil/rock |
| Cheek Wall | mostly aesthetics; can provide bearings protection for elements |
| Abutment Stem | supports the bridge seat; retains soil |
| Deep Foundations | transmits weight of the abutment to the supporting soil/rock |

### 3.5.3 Retaining Structures

A bridge's retaining structure is designed to resist lateral forces on the bridge such as soil pressure (Chen & Duan 2000). Common types of retaining structures include a cantilever wall, tieback wall, soil nail wall, and mechanically stabilized embankment wall (Chen & Duan 2000). Often the bridge's abutments are used as retaining structures, especially if they are full height or stub abutments (Chen & Duan 2000). All retaining structures must withstand a bridge's bearing capacity and structural components as well as be resistant to overturning and sliding (Chen & Duan 2000).

### 3.5.4 Scour Protection

Bridge abutments and piers that are adjacent to streams, flood plains, or water are susceptible to structural failures due to scouring action (Barker & Puckett 1997). Scour is a site design consideration while designing the substructure of the bridge. One type of scour in a river is due to the lateral shifting of the channel. This is most common at the outside of each bend in a meandering river due to the higher velocity of the stream (Barker & Puckett 1997). Another type of scour occurs due to the erosion of the river bed during periods of high flow. The maximum depth of scour can be predicted by observation of the river bed during periods of high flow. A third type of scour comes from areas of high velocity in the river due to obstructions such as piers (Barker & Puckett 1997).

The most common type of scour protection is the use of riprap. Riprap is a sustaining wall of stones or chunks of concrete that are used to prevent slope erosion. Other types of sour protection include gabions, articulated concrete blocks, and grout filled mattresses. Placement of abutments and foundations can also serve as scour protection methods (Barker & Puckett 1997).

### 3.5.5 Piers

A pier is located between the ends of a bridge. It is designed to support the bridge at intermediate intervals with minimal interference to road or water traffic passing under the bridge. A pier is generally constructed with only one column and supported by one footing (Rossow 2007).

The existing Quaboag River Bridge is supported by one solid shaft pier. This pier is to be removed to streambed level in the state's proposed replacement bridge design. Since the proposed design is of a single span, there will be no piers in the substructure of the bridge.

### 3.5.6 Bearings

Bridge bearings provide an interface between and superstructure and substructure of a bridge (Rossow 2007). Their primary functions are to transmit loads from the superstructure to the substructure, allow rotation caused by loading, and permit horizontal movement of the superstructure due to thermal expansion and contraction (Rossow 2007). The main forces on a bridge bearing are its self-weight, traffic loads, wind loads, and earthquake loads (Chen & Duan 2000). A bridge bearing consists of four basic elements: the sole plate, the bearing or bearing surface, the masonry plate, and the anchorage. Table 4 describes each of these elements. All bearings have at least the bearing or bearing surface, though not all bearing have all four of these elements.

**Table 4: Primary Bearing Elements (Rossow 2007)**

| Element | Description |
|---|---|
| Sole Plate | Steel plate attached to bottom of girders or beams |
| Bearing or Bearing Surface | Secured to the sole plate; provides transmittal of forces from sole plate to masonry plate |
| Masonry Plate | steel plate attached to the bearing seat; distributes vertical forces to substructure unit |
| Anchorage | bolts that connect the bearing to the superstructure; restrains masonry plate from horizontal movement; can be used to resist transverse movement |

Bearings can be fixed, which restrict translational movements and allow rotational movements, or expansionary, which allows both translational and rotational movements (Chen & Duan 2000). Different types of bearing can be classified as: sliding plate, roller, rocker, pin and link, elastomeric, or pot bearings (Rossow 2007). Sliding plate bearings typically provide longitudinal movement on bridges with spans of 15 meters or less (Chen & Duan, 2000). Roller bearings are composed of one or more cylindrical rollers between two parallel steel plates. Singular rollers accommodate both translational and rotational movements whereas multiple rollers work only with translational movements (Chen & Duan 2000). Rocker bearings come in a variety of designs. Most consist of a pin at its top to allow rotational movement and a curved surface at its bottom to allow translational movement of the bridge (Chen & Duan 2000). Pin and link bearings are usually found in steel bridges and are used to accommodate rotational movements (Chen & Duan 2000). Elastomeric bearings consist of both plain and laminated neoprene pads to transmit both types of movement (Chen & Duan 2000). Lastly, pot bearings comprise of a plain elastomeric disk that is confined in a steel "pot" ring (Chen & Duan 2000). These bearings are able to transmit translational loadings (Chen & Duan 2000).

These different types of bearings have comparatively different loading capacities and costs as shown in Table 5. Bearings need to be routinely inspected to ensure they still work for their intended purpose – translational and/or rotational movement (Rossow 2007). Neglecting bearing inspection can lead to bridge failures and ethical questions in the bridge's maintenance (Rossow 2007).

Table 5: Bearing Type Capacities and Cost (Chen & Duan 2000)

| Bearing Type | Load | | Translation | | Rotational Max (rad.) | Costs | |
|---|---|---|---|---|---|---|---|
| | *Min (kN)* | *Max (kN)* | *Min (mm)* | *Max (mm)* | | *Initial* | *Maintenance* |
| Sliding Plate | 0 | >10,000 | 25 | >10 | 0 | Low | Moderate |
| Single Roller | 0 | 450 | 25 | >10 | >0.04 | Moderate | High |
| Multiple Roller | 500 | 10,000 | 100 | >10 | >0.04 | High | High |
| Pin and Link | 1,200 | 4,500 | 0 | 0 | >0.04 | Moderate | High |
| Elastomeric | 0 | 450 | 0 | 15 | 0.01 | Low | Low |
| Pot | 1,200 | 10,000 | 0 | 0 | 0.02 | Moderate | High |

## 3.6 Decommissioning of the Bridge

The decommissioning, or the removal of an existing bridge, is a necessity in the case of our project since the Quaboag River Bridge is a bridge replacement project. Things to consider when removing a bridge include type of the demolition, traffic management, and environmental impact of the surrounding area (Gedeon 1995). One specific consideration, the removal of the existing center pier, is also discussed later in this section.

### 3.6.1 Types of Bridge Demolition

The type of demolition a project uses depends on the type of bridge being demolished and the circumstances surrounding it. The most common types of demolition methods are detailed in Table 6. Each method is unique, with different applications, advantages, and disadvantages (Abudayyeh, Sawhney & Buchanan 1998).

**Table 6: Demolition Methods (Abudayyeh, Sawhney & Buchanan 1998)**

| Method | Applications | Advantages | Disadvantages |
|---|---|---|---|
| Hydraulic Hammers | Demo of bridge decks, peers, slabs, and pavements | High production rate, greater mobility, operable in all weather | Noise, dust, vibrations |
| Whiphammers | Bridge deck removal | High production rate | High energy input |
| Crushers | Full and partial bridge removal | No dust, low noise, no vibrations, great mobility, operable in all weather, rapid and safe cutting of rebar | |
| Water Jet Cutting | Partial removal of deteriorated concrete in bridge decks | Minimum labor, low noise, no dust, high production rate, no vibration, remaining concrete surface irregular allowing good bonding to new concrete | Rebar shadow problems, cost, needs large quantities of water, and disposal of the water that is mixed with debris |
| Blasting and mini blasting | Full and partial bridge removal | Speed, short durations of noise and dust | dust, noise, vibrations, flying debris, and dangerous |
| Sawing and Cutting | Partial removal of deteriorated concrete in bridge decks | No dust, no vibration, and produces clean edges | Difficulties arise around rebar, cost |
| Mechanical Splitters | Full and partial bridge removal | No vibration, inexpensive, little dust, remaining concrete undamaged, and can be used underwater | Time consuming and requires the use of breakers to expose rebar |
| Chemical Splitters | Full and partial bridge removal | No vibration, no noise, safe, and non-explosive | More expensive than mechanical splitting, requires more time, not operable in cold weather |
| Jackhammers | Partial removal of deteriorated concrete in bridge decks | Easy to use | Slow, noise, dust, and remaining concrete and rebar may be damaged |

A combination of methods can be used in bridge demolition. The types of methods used depend on the financial, site, structural, existing concrete, environmental, worker and public safety, recycling, and disposal limitations of a bridge (Abudayyeh, Sawhney & Buchanan 1998).

#### *3.6.1.1 Demolition of Center Pier and Abutments*

Since the proposed design of the Quaboag River Bridge consists of a single span, the existing center pier under the bridge needs to be removed. The pier is eyeballed at 5 feet in width, and made entirely of concrete. The state proposes removing the pier down to the stream level (Depaola & Broderick 2012). This would allow river traffic to move more freely under the bridge. The existing abutments are also proposed to be partially removed to provide additional clearance under the bridge. Remnants of the abutments will act as scour protection to the new bridge. The removal of the center pier as well as the partial removal of the abutments may consist of one of the methods listed in Table 6. As both parts of the substructure are in or close to water, environmental and worker safety concerns are of the most importance in the removal of this pier.

### 3.6.2 Traffic Management

Whenever bridge construction is performed, drivers are faced with unexpected traffic conditions. These changes can be hazardous, therefore making traffic management important during bridge construction (Rossow 2005). Worker and traffic safety, public relations, and cost are the three most important factors associated with traffic management ("Work Zone…" 2002).

#### *3.6.2.1 Worker and Traffic Safety*

To ensure worker and traffic safety, traffic control procedures are set by a work-site manager and are used to (Rossow 2005):

- Warn drivers and pedestrians of any hazards
- Advise traffic of the proper way to travel through construction
- Inform roadway users of the changes in traffic regulations of the surrounding area
- Guide traffic through/around the work-site
- Define areas where traffic should not operate

Additional onsite safety procedures include: short safety meetings each morning prior to the beginning of daily construction, proper use of tools and equipment, and the following of OSHA's safety regulations for construction workers (Rossow 2005).

#### *3.6.2.2 Public Relations*

During road and bridge construction it is important the public remains informed of the changing traffic conditions of the area ("Work Zone…" 2002). The public can remain informed through: the local media, the MassDOT website, town meetings and informal hearings, surveys and brochures, as well as contact to the surrounding land owners ("Work Zone…" 2002). Keeping the public informed of changing road conditions maintains adequate traffic flow through the roadway under construction and can avoid driver aggravation.

#### *3.6.2.3 Cost*

Cost associated with traffic management include the building and removal of a temporary runaround, using a local route detour and structurally upgrading its roadway, paying for an accelerated construction progress, or providing stage construction that, may result in increased unit costs ("Work Zone…" 2002). When determining the cost for onsite options, a designer considers:

- Right of way costs
- Additional construction costs
- Environmental effects
- Vehicular delay
- User costs
- Crash potential

A designer also considers the effects of unofficial detours when designing a traffic management system. However, in the Quaboag River Bridge project, there is no detour route available, so the traffic manager will not need to consider this factor ("Work Zone…" 2002).

#### *3.6.2.4 Quaboag River Bridge Traffic Management Plan*

The Quaboag River Bridge's traffic management plan is to build the bridge in two stages. Stage one will maintain vehicular and pedestrian traffic on the western side of the existing structure, while the eastern side is removed and that side of the proposed bridge is constructed. Stage two will shift traffic to the newly constructed section of the bridge, while the remainder of the existing bridge is removed and the western portion of the proposed bridge is constructed (Depaola & Broderick 2012).

### 3.6.3 Environmental Impact

Before a contract to decommission a bridge is awarded, an evaluation to assess the impact of the concrete removal by the river, stream, or waterway needs to be performed (Gedeon 1995). The assessment varies from project to project and greatly depends on the size and condition of the waterway and size of the bridge to be decommissioned (Gedeon 1995). In some cases the aggregate used in the bridge's concrete is the same size and type as that found in the waterway, creating little environmental impact on the area (Gedeon 1995). In other cases, where debris fragments are larger, they are transported and placed in open water to serve as a fish attractor reef (Gedeon 1995). Overall, recycling is highly encouraged in concrete removal when a bridge is approved for decommissioning.

In the case of the Quaboag Bridge, the shallow streambed and tight site are environmental considerations that the project managers need to consider when demolishing the existing bridge. There could be adverse effects to the fish and water life populations and soil quality if too much bridge debris falls into the stream. To avoid any adverse effects on the environment, a tray like receptacle or container, called a catch basin, can be used to catch falling debris from the bridge deck demolition (LaBounty 2011). The debris that falls into the catch basin is supported by a crane system and placed into a dump truck to be disposed of (LaBounty 2011). This catch system allows crushed concrete pieces and rebar of the existing bridge to fall conveniently below, without an undesirable and hazardous cleanup process. Catch basins satisfy environmental concerns and would help keep the Quaboag River debris free during decommissioning.

## 3.7 Constructability

"Constructability" is to obtain broader knowledge of building methods early in the design process to have construction of the project run smoothly (Rowings, Harmelink & Buttler 1991). Bridges are often designed to be of high quality and safety standards but with little attention to the construction methods and details (Rowings, Harmelink & Buttler 1991). Construction issues that are encountered in the field can be avoided by considering efficient building strategies before site work begins (Rowings, Harmelink & Buttler 1991). By incorporating construction knowledge into design, costly change orders, budget overruns, scope growth, and litigations can be avoided (Rowings, Harmelink & Buttler 1991). The areas of construction knowledge that are listed below prove beneficial in the design process of a project (Rowings, Harmelink & Buttler 1991):

- Availability and cost of materials
- Availability and cost of skilled labor
- Constraints and costs of transportation
- Understanding of various construction methods

If these areas of knowledge are considered, construction of a project is more efficient. This is due to alternatives considered for construction strategies before the project is built, saving time in the field if a problem were to arise. Overall, the development and application of constructability concepts has the potential for creating better designs (Rowings, Harmelink & Buttler 1991).

## 3.8 Building Information Modeling

Building information modeling (BIM) is a digital representation of physical and functional characteristics of a facility. BIM shares knowledge and displays information that allows invested parties to work together in a timely and costly manner. BIM programs are considered to be interoperable. Interoperability is the ability of two or more programs to operate in a reciprocal manner. It is important to the success of the use of BIM because it allows individuals and systems to access, identify, and integrate information across many systems (Salazar 2012). Examples of BIM programs are Revit, AutoCAD Civil 3D, eSPAN140, Primavera, and Microsoft Project.

### 3.8.1 Revit

Revit is a 3D BIM software. It is considered a 3D software because it produces design, analysis, and documentation of a building model (Salazar 2012). Revit concepts are parametric objects, families, and categories. Parametric objects are objects that can change size, material, and graphic look while remaining the same object. Every parametric object is belongs to a family. Revit has model categories and annotation categories. Model categories cover all physical objects found in buildings. Revit can also be used in collaboration with other programs to create 4D (includes time) or 5D (includes time and cost) models.

### 3.8.2 AutoCAD Civil 3D

AutoCAD Civil 3D is an object oriented interface. Objects represent survey, design, and construction elements. The objects it uses include points, surfaces, alignments, profiles, pipe networks, and corridor models. Civil 3D also allows for easy labeling of points, surfaces, parcels, and alignments (Salazar 2012). These objects visually show the project and their labels verbalize the information. Civil 3D can be used to create site and transportation designs. It can also be used in collaboration with other programs to create a 4D or 5D model.

### 3.8.3 eSPAN140

eSPAN140 is a web-based design tool for short-span steel bridges up to 140 feet. eSPAN140 features steel fabrication and errection details including rolled beam, plate girder, corrugated steel pipe and structural plate (American Institute of Steel Construction July 2012).

### 3.8.4 Primavera

Primavera can be used with AutoCAD Civil 3D, Revit, or any other 3D BIM to create a 5D model because it adds time and cost to a project. It can create presentations, time schedules, cost control, site development plan, plan diagrams, and underlays (Salazar 2012).

### 3.8.5 Microsoft Project

Microsoft Project can also be used with a 3D BIM to create a 5D model. It integrates time and cost in an organized manner. It is compatible with all other Microsoft software such as Excel, Share Point, and Visual Studio. It features schedules, timelines, cost analysis, visual planners and management resources, single entry mode (Microsoft 2010).

## 3.9 Evaluation Criteria for Bridge Designs

There are different ways of evaluating engineering designs. Whether that design is a bridge, building, roadway, etc., different alternative designs must be evaluated using set criteria. These criteria can be developed by the engineer, a federal or state government, developer or another party associated with the design. All of these different parties may have different criteria they wish to evaluate a design by, or they may weight one piece of criteria more heavily than others. Often one of the most valuable criteria is the cost of the proposed project, but there are other aspects that may be evaluated as well. Whether it’s aesthetics, environmental impact, sustainability or some other criterion, each proposed design will be evaluated and a single design will be chosen that best suits the project’s needs.

### 2.9.1 Overview of Evaluation Criteria

As a result of the large array of criteria from which to choose, our team has developed a system through which to evaluate the two alternative bridge designs. We will use the following categories to conduct our evaluation:

- Cost
- Life Expectancy (sustainability)
- Environmental impact
- Ethics
- Timeline

The cost of the bridge will be an estimate of the total cost for each design. Life expectancy of the two bridges includes the time until the bridge needs to be replaced as well as the time to any major maintenance that will need to be conducted on the bridge. As discussed previously, the environmental impact of each design will include the impact of the methods used to remove the center pier. However, it will also include a measure of the carbon dioxide emissions and energy consumption for each design. The ethics of each bridge design will be evaluated by the magnitude of the design loads for each design. Finally, we will evaluate how long the design and construction process of each design would take.

This section is meant to give a brief overview of the evaluation criteria our team has chosen to use for the two proposed designs. We will elaborate further on the evaluation process in the next chapter of our proposal.

### 2.9.2 Life-Cycle Cost Analysis

Since the cost of a construction project is often the determining factor in the evaluation of alternative designs, a more in-depth analysis will be conducted for this evaluation criterion. The cost will consist of the overall material cost for the two different designs, the labor and construction costs as well as the cost of the bridge during its lifetime. A life-cycle cost analysis (LCCA) will be conducted to determine the bridge maintenance and repair costs for each design.

LCCA is a process by which the future costs of a proposed design are evaluated and compared to other alternative designs. The steps to conducting this type of cost analysis are as follows (US Department of Transportation 2002):

- Establish design alternatives
- Determine design activities
- Estimate both agency and user costs
- Determine life-cycle costs for each alternative design
- Analyze results

Once the alternative designs are established, the activities associated with each design must be determined. These activities include both the initial design and construction activities as well as activities that will need to occur in the future after the project has been completed and been in use (US Department of Transportation 2002). The cost of both the initial and future activities must then be determined as well as any specific costs experienced by users. Once all of the initial and future costs are determined for each design, they are converted into present-day costs and compared (US Department of Transportation 2002).

Software is often used to aid in LCCA. There are many different types of software available, but one example is BridgeLCC produced by the National Institute of Standards and Technology (NIST) (NIST 2011). BridgeLCC was developed to help engineers assess bridge designs that use new-age construction materials. However, the software also serves to compare alternative designs that use conventional materials, which is how the software will be utilized for the purposes of this project (NIST 2011).

Through the use of this software and other resources the cost of each design can be estimated and compared. The methodology for how this will be done is discussed in the next chapter.

# 4. Methodology

## 4.1 Introduction

The purpose of this methodology is to provide an overview of how our project will be developed. It will also serve as a basis for constructing the project schedule. We will design a precast concrete girder model and steel girder model in 2D with the specified design loads from the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications 2012 Manual based on the Quaboag Bridge's intended use. Then we will compare the two models using weighted evaluation criteria to determine which model is "best" to make into a 3D model. We will identify cost and construction considerations with the chosen model.

## 4.2 Evaluation Methods

The goal of our project is to determine which of the two bridge designs is a better choice for the proposed replacement project. In order to make that determination, our team developed a list of criteria to evaluate different aspects of the designs. This list of criteria was briefly outlined in our background, but in this section we will discuss the criteria in more detail. We will focus on how we intend to use each criterion to evaluate the bridges.

### 4.2.1 Criteria

As we discussed in our background, our team will evaluate each bridge design on the basis of cost, life expectancy, environmental impact, ethics and timeline. The process by which we will use these criteria to ultimately determine the best design will be discussed in the next subsection. The following paragraphs will be dedicated to how our team will evaluate the designs in each individual category.

#### *4.2.1.1 Cost*

Through the use of cost indices, software and other resources, our group will estimate the cost of each design. The cost estimates will incorporate all aspects of the project including labor (both design and construction work) and materials. The cost analysis of each design will also include any major maintenance costs the bridges will experience during their lifetime. Some cost indices that will be beneficial to our team's analysis are the Construction Cost Index and Material Cost Index published by Engineering News Record (ENR 2012). ENR also publishes a Building Cost Index, but since we are focused on bridge designs and not building designs, this index may not be as much of a use for our project.

As discussed in the previous chapter, our team will also utilize software to compare the costs of the two bridge designs over each designs lifetime. BridgeLCC, developed by the National Institute of Standards and Technology (NIST) is an example of a program that can be used by our team to conduct life-cycle cost analysis (NIST 2011). The software will allow our team to evaluate the cost of the two designs over the lifecycle of each design, not just the immediate construction and design costs.

#### *4.2.1.2 Life Expectancy*

Our team considers life expectancy to be how long (in years) each of the bridge designs will last before needing to be replaced. The concept main be referred to as the "sustainability" of each bridge throughout the project as well. When ranking each of the bridges in terms of life expectancy, our team will also take into consideration any major maintenance that would need to take place on the bridges as well. For example, replacing the bridge's deck would be considered a major maintenance project.

In order to determine the lifecycle of each bridge our group will conduct further research on past bridge projects as well as construction material resources. These resources will help us determine how long different materials last and when different types of bridges are often replaced.

#### *4.2.1.3 Environmental Impact*

Another aspect our team will evaluate is the environmental impact of each design. We will consider both the environmental impact of construction process, such as the decommissioning of the bridge, as well as the impact resulting from the materials used in the bridge design. Since the bridge is located in a wetland area we will research special considerations that must be taken when constructing in this type of area.

The environmental impact of the bridge materials will be evaluated based on the amount of carbon dioxide ($CO_2$) emissions produced and the energy consumed through the production of the materials used. Our previous research indicates that the amount of $CO_2$ produced and the energy consumed by construction equipment/processes accounts for a very small percentage of the total emissions and energy consumed (Itoh, Sunuwar, Hirano, Hammad & Nishido 2000). As a result our team will most likely evaluate construction processes through other aspects besides $CO_2$ emissions and energy consumption. Such aspects may include damage to, or disruption of, the surrounding wetlands.

#### *4.2.1.4 Ethics*

The two bridge designs will be ethically evaluated based on the magnitude of loading that each bridge can sustain. These design loads will be determined through the use of American Association of State Highway and Transportation Officials (AASHTO) and Precast/Prestressed Concrete Institute design specifications. These specifications will provide us with the tools necessary to design the steel and concrete parts of our bridges and will ultimately determine the design loads for each of our two designs. We will provide more detail about how we will use these standards in our sections dedicated to the superstructure and substructure.

#### *4.2.1.5 Timeline*

Finally the two designs will be evaluated based on the timeline of each theoretical project. Our team will estimate the amount of design time and construction time necessary to complete the project. In order to develop an estimate for each of the designs we will research other similar projects as well as typical durations of different construction processes. The use of software will help our team develop construction schedules for each of the designs. A shorter estimated timeline will rank better in our final evaluation than a longer timeline.

### 4.2.2 Evaluation Process

After our team evaluates each of the designs individually in the categories discussed above, we will then conduct an overall evaluation of the designs. In our final report we will include the rankings of each design in the individual categories so that they may be compared in specific criteria. However, to determine which design is better suited for the given site, we will compare the importance of each of the criteria we have listed and apply weights to each category. After conducting our analysis through the process described below, our team will be able to indicate which design is more favorable.

The process our team intends to use will first evaluate the designs independent of cost, so that it can be determined which design will provide more benefit for the given price (Hunter & Steward 2002). As mentioned above the criteria that our group has outlined will be compared and the importance of each category will be determined in relation to the other categories. Developing this type of hierarchy will allow our team to apply weight percentages to each category based on level of importance (Hunter & Stewart 2002).

Each design will then be given a numerical ranking from 1 to 10 in each of the given categories. In our final report we will discuss how we determined the rankings. By multiplying each of these rankings by the determine weights we will obtain a score, or level of performance, for each of the designs in the given categories. For comparison purposes we will also rank the original conditions (or no build scenario) so that our team has a baseline to work from (Hunter & Stewart 2002).

The levels of performance for each category will be tallied given the designs an overall score. By comparing these scores to the original conditions we can determine if there was a positive or negative change by applying a given design. Finally, each total score will be divided by the estimated cost of the designs. This value, called the value index, will allow our team to evaluate how much benefit was obtained for a given cost. Through comparing the value indices of each design we will then determine the most beneficial design (Hunter & Stewart 2002).

## 4.3 Applying Loads

We will be using AASHTO LFRD Design Specifications (2012) loading traffic scheme HL-93 from Section 3 of the ASSHTO manual. This loading allows 2-axle vehicles to be 18 tons maximum, 3-axle vehicles to be 22 tons max, and 5-axle vehicles to be 34 tons max. Any other loading considerations that we will need to use will be taken from the same section in AASHTO Manual.

## 4.4 Superstructure

We will be designing two superstructure designs, a steel model and a precast concrete model. Once we have determined which model we will use for the girder design, we will design the concrete deck of the bridge.

### 4.4.1. Steel Box Girder Design

We will use the AISC Steel Construction Manual to help us design the steel box girder bridge. We will also design the bridge to AASHTO LRFD Bridge Design Specifications (2012). We will evaluate this design with the evaluation criteria previously mentioned in this proposal.

### 4.4.2 Precast Spread Box Beam Design

The design of the precast spread box beam bridge will be conducted through the use of AASHTO LRFD Bridge Design Specifications (2012). These specifications will guide our team in designing the box beams themselves as well as the reinforcing steel to be used in the beams. Another resource that will be utilized by our team during the design process is the Precast/Prestressed Concrete Institute's (PCI) Bridge Design Manual (2003). The Federal Highway Administration (FHWA) also has two design examples, concrete and steel, that will help guide our team through the design process (FHWA 2011). As with the steel girder design, the precast superstructure will be evaluated based on the evaluation criteria discussed above.

### 4.4.3 Bridge Deck Design

The bridge deck will also be designed using AASHTO and PCI specifications (AASHTO 2012; PCI 2003). The FHWA design examples will also be utilized by our team for the purposes of designing the bridge deck (2011). Both of the designs chosen by our team will consist of a concrete deck. Through our background research we have determined that this is the most practical decking material to use when compared to steel or timber decks. We will determine the type of concrete used and the wearing surface later in the design process.

## 4.5 Substructure

The substructure will be designed only once, after the type of superstructure is chosen. We will design the foundations, abutments, scour protection, and bearings of the substructure using the AASHTO LRFD Bridge Design Specifications 2012 Manual. Evaluation criteria of cost, life expectancy, environmental impact, ethical considerations, and timeline will be taken into account when designing these elements.

### 4.5.1 Foundations

We will first choose what type of foundation our bridge design will be by determining the soil type of the site. The foundations will be designed using AASHTO LRFD Bridge Design Specifications 2012 Manual. Foundations will be proportioned and designed so that the supporting soil provides adequate nominal resistance under all applicable load limits. They will also be proportioned and located to maintain stability under all applicable loading states (AASHTO 2012).

### 4.5.2 Abutments

Abutments will be full height abutments, as show in Section 3.5.2 in Figure 12. This way they will act as both supports to the superstructure and approaching roadway, as well as retaining structures. We will design the abutments using the following steps (Barker & Puckett 1997):

1) Select preliminary proportions of the wall
2) Determine loads and earth pressures
3) Calculate magnitude of reaction forces on base
4) Check stability and safety criteria
    a. Check the location of the normal component of reactions
    b. Check the adequacy of bearing pressure
    c. Check the safety against sliding
5) Revise proportions of the wall and repeat Steps 2-4 until stability criteria are satisfied then check:
    a. Settlement within tolerable limits
    b. Safety against deep seated foundation failure
6) Evaluate economic, ethic, environmental, and time line considerations

#### *4.5.2.1 Preliminary Proportions*

These proportions will consider scour concerns on the abutments. They will be determined by the proposed design of the bridge, exhibited to Brookfield at the Design Public Hearing Meeting. These preliminary proportions will be the baseline to the design of the abutments and will change as calculations on the design are performed (Barker & Puckett 1997).

#### *4.5.2.2 Loads and Earth Pressure*

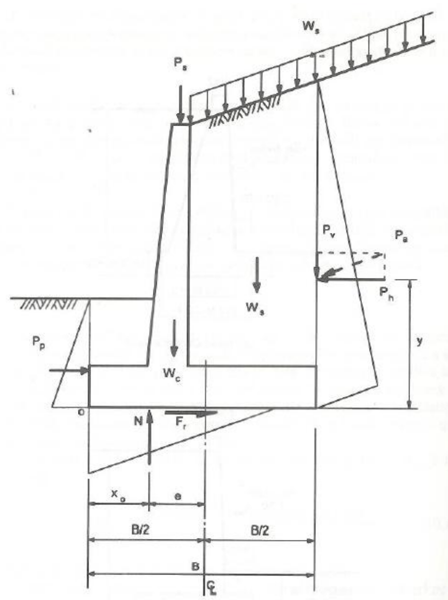
The loads that we will use will follow the AASHTO LRFD Bridge Design Specifications 2012 Manual, and be designed for the intended use of the bridge. Earth pressures will be determined by either on-site analysis of the bridge's surrounding area or information given by the Replacement Bridge Project No. B-26-002 Project Manager.

### *4.5.2.3 Reactions Forces*

Reaction forces will include gravity, static pressure from the superstructure, dynamic pressure from the loading, and earth pressures. Figure 16 on the following page shows the typical forces on a cantilever abutment.

### *4.5.2.4 Stability and Safety Criteria*

Performance factors and factors of safety will be considered in the abutment design as safety criteria. Overturning moment, bearing failure, and lateral pressures will be checked for safety in the design process.




$P_a$ = active earth pressure

$P_p$ = passive pressure

$P_v$ = vertical pressure

$P_h$ = lateral pressure

$P_s$ = structure pressure

y = 0.4 x height

B = abutment length

N = vertical resultant force

$F_r$ = friction

$x_o$ = (summation of moments about point o)/N

e = (B/2) – $x_o$

$w_{c=}$ = weight of concrete

$w_s$ = weight of soil

**Figure 16: Typical Forces on an Abutment (Barker & Puckett 1997)**

### *4.5.2.5 Considerations*

The detailed evaluation criteria explained in sections 3.9 and 4.2 will be used in the final determination of the abutment design.

### 4.5.3 Scour Protection

Where the potential for scour exists on the foundation and abutments, we will attempt to locate these elements away from the river bank when practical. Specifically, we will design the foundations below the scour depths as per the AASHTO Design Specifications (AASHTO 2012).

### 4.5.4 Bearings

First we will choose the type of bearing from Table 5 by keeping the evaluation criteria describe in section 3.9 and 4.2 in mind. Depending on the type of bearing, we will develop a procedure to design the chosen bearing. The loads of the bridge's superstructure will be used to design the bearing's characteristics of material and size.

## 4.6 Decommissioning

Since our project takes into account the existing bridge, the decommissioning process needs to be considered. Two factors we will consider are the type of demolition and how to control the environmental impact of the demolition.

### 4.6.1 Demolition

The demolition of the old bridge includes the removal of the superstructure and the partial removal of the substructure. To choose the type of bridge demolition of the deck and wearing coarse, we will compare the methods listed in Table 6: Bridge Demolition Methods in Section 3.6.1. The factors we will consider when choosing a demolition method are: cost, environmental impact, life-cycle, ethical considerations, and time line. We will assume the bridge to be in continuous use during the demolition, as described in Section 3.6.2.4, and that the community wants minimal impact on the Quaboag River and the local business, White's Landing.

We will use the same evaluation criteria and considerations to choose the type of demolition method to remove the center pier and abutments.

### 4.6.2 Environmental Impact

We will determine the type of catch basin used in the demolition of the bridge once the demolition method is chosen. This will be done by researching different catch basin methods to determine what type is best suited for our project. An example of one catch basin system is described in Section 3.6.3.

## 4.7 Constructability

Our project will take into account the constructability of our 2D and 3D models while we are in our design process. Factors that we will consider are:

1) Availability and cost of materials
2) Constraints and costs of transportation
3) Various construction methods

We will assume that there is normal availability of skilled labor. Materials and their transportation to the site that are used in our models will be researched for accessibility in the Massachusetts area. We will also research available demolition and construction methods surrounding the Brookfield area. Considering factors 1-3 will create a more realistic final design of the Quaboag River Bridge No. B-26-002 replacement project.

## 4.8 Building Information Model

For our project we will need a 3D BIM that will include design, analysis, and documentation of a building model. Revit Structures and AutoCAD Civil 3D both offer 3D BIM models. We have chosen to work with both programs since they can be integrated with each other. We will use Civil 3D for the topography of the area and some road and bridge elements, then transfer the data into Revit Structures (IMAGINiT Technologies, 2011). In Revit Structures we will complete the bridge design and add loads. We will use Revit Structures for the design and analysis of both the steel box girder and precast concrete bridge options. We will also use eSPAN140 for the load analysis of the steel box girder bridge design. However, we have not found an equivalent program to eSPAN140 for the load analysis of a precast concrete bridge design but will continue to research such programs.

## 4.9 MQP Project Tasks and Schedule

As a team, we will work collaboratively on each part of the project. This includes designing the 2D precast and steel models, the evaluation criteria process to choose a design, choosing and creating a 3D model, calculating the superstructure and substructure designs, researching cost and construction possibilities, and working with different design software to create our deliverables. Although we will each contribute to these different parts of the project, we will each head different aspects. Listed below are our delegated focuses through the continuation of our project:

1. Developing and Analyzing Evaluation Criteria and Methods – *Mariah*
2. Superstructure Calculations for Precast and Steel 2D Models – *Madison and Mariah*
3. Substructure Calculations for Chosen Model – *Lauren, Madison, and Mariah*
4. Design Software Programs –*Lauren and Madison*
5. Cost and Construction Considerations – *Lauren*

These tasks were decided upon as a group based on our Civil Engineering concentrations and experience. Madison and Mariah both have a concentration in structures while Lauren has a completed concentration in Project Management. Tasks were also decided upon classes that we have each taken and the background research we did individually for this proposal.

The milestones for our project are listed below. We aim to have the specified work for each milestone completed before the date.

1. Nov.22 (*Thanksgiving*)
    - Make a decision between the steel girder and precast concrete girder superstructure designs
2. Dec. 13 (*End of B term*)
    - Complete and submit writing for Milestone 1
    - Complete all major calculations on the superstructure and substructure
    - Complete majority of smaller calculations
3. Jan. 24 (*2 Weeks into C term*)
    - Complete all smaller calculations
    - Complete and submit majority of writing for Milestone 2
4. Feb. 7 (*4 Weeks into C term*)
    - Complete all calculations
    - Complete and submit writing for Milestone 3
5. Feb. 21 (*6 Weeks into C term*)
    - Complete and submit all cost/construction analysis writing
6. March 1 (*End of C term*)
    - Final report completed

## 5. Deliverables

With the completion of this project we will have proposed a bridge design that best serves the Brookfield, MA community. We will have analyzed each superstructure design using the evaluation criteria we have created. This analysis will include a life-cycle cost analysis for each design. We will also have completed all design calculations by hand and checked all calculations by the software mentioned previously in the methods section. We will also have designed a 3D BIM model with the help of 3D BIM software. The 3D model will provide a structural drawing of the bridge design and bridge components, structural analysis of the bridge, and documentation of the design and analysis. Lastly, we will have produced graphs, charts, and models for easy display and understanding of important information.

# 6. Conclusions

## 6.1 Predicted Chosen Design

We expect our chosen design will be the precast spread box beam design due to our background research. Based on our background research, we know that a precast concrete design is more economically desirable than a steel design because of its lower cost and longer life expectancy. Looking at our evaluation criteria, we predict that the precast concrete design will also be more environmentally friendly and constructible than a steel design based on the Quaboag Bridge's existing site conditions. Finally, we predict that the precast concrete and steel designs will be of equal ethical value, because both models will be designed with the same AASHTO loading schemes.

## 6.2 Possible Obstacles

Obstacles we see our project encountering are:

1) Learning the design software
2) Calculating for all the possible forces in our designs
3) Finding information about local materials
4) Contacting the MassDOT project manager for information about the Quaboag Bridge

We believe that learning the design software will be time consuming and are therefore setting ample time aside to learn these programs. To ensure calculation of all possible forces on our design, we plan on submitting our design calculations multiple times for our advisor to review before our final MQP Report Submittal. This way there will be a professional set of eyes on our design and calculations, helping us with anything we might overlook. To find information about local construction materials, we will directly call companies when we have any questions that are not answered on their websites. Finally, we are most concerned with contacting the Project Manager with questions concerning the Bridge Replacement No. B-26-002 Project. All of our A term attempts have been unsuccessful. If we do not receive needed information from him that affects the continuation of our design, we will assume any existing conditions we need, making note of them in our final methodology.

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## Appendix B: Evaluation Criteria Analysis

Table 45 - Criteria Hierarchy Comparison

| Criteria Comparison | Higher Importance | Reasons |
|---|---|---|
| LE vs. EI | LE | - if the bridge lasts for 75+ years, another bridge will not need to be built, resulting in less construction in the future<br>- it is in Brookfield's best interests to have a longer lasting bridge because it's cost effective |
| LE vs. PE | PE | - safety is more important than the life expectancy of a bridge<br>- if the bridge is ethical, it means that it has a longer life expectancy because it was designed for the loads on the bridge |
| EI vs. PE | PE | - human safety is more important than environmental concerns |
| LE vs. T | LE | - if the road is a well-traveled road, a designer should be more concerned with the bridges life expectancy than construction process so that future construction can be avoided |
| EI vs. T | EI | - it is important to protect the marsh<br>- Fiskdale road is not too busy with traffic flow<br>- Brookfield may have to spend a lot of money later on cleaning up the marsh if they are not careful during the construction process |
| PE vs. T | PE | - prioritize no one getting hurt on the job site<br>- focus on building a safe bridge and not speed through the construction process<br>- Fiskdale road is not too busy with traffic flow |

### Table 46 - Rating Rubric

| Category | Exceptional (10-8) | | Neutral (7-4) | | Poor (3-1) | |
|---|---|---|---|---|---|---|
| | ***Steel*** | ***Concrete*** | ***Steel*** | ***Concrete*** | ***Steel*** | ***Concrete*** |
| **Life Expectancy** | *Scale:*<br>- Greater than 75 years<br>- paint every 5 years or less | *Scale:*<br>- Greater than 75 years | *Scale:*<br>- 75 years<br>- paint every 8 years | *Scale:*<br>- 75 years | *Scale:*<br>- Less than 75 years<br>- paint every 10 years or more | *Scale:*<br>- Less than 75 years |
| **Environmental Impact (Average of the following)**<br>**Energy Consumption** | *Scale: (Energy Consumption)*<br>- very little energy is consumed during the manufacuring<br>- very little electricty is needed<br>- hardly consumes any resources | | *Scale: (Energy Consumption)*<br>- a moderate amount of fossil fuels are consumed | | *Scale: (Energy Consumption)*<br>- large consumption of fossil fuels<br>- consumes a large quantity and variety of resources<br>- large amounts of electricty needed for production | |
| **CO2 Emissions** | *Scale: (CO2 Emissions)*<br>- very little CO2 is produced | | *Scale: (CO2 Emissions)*<br>- some CO2 is produced | | *Scale: (CO2 Emissions)*<br>- large amounts of CO2 produced | |
| **Other Emissions** | *Scale: (Other Emissions)*<br>- little or no other emissions are produced | | *Scale: (Other Emissions)*<br>- some pollution from other chemicals occurs | | *Scale: (Other Emissions)*<br>- large emissions from other types of toxic | |
| **Recyclability** | *Scale: (Recyclability)*<br>- most or all of the material can be recycled<br>- a large amoung of other recycled materials can be used | | *Scale: (Recyclability)*<br>- some amount of the material can be recycled<br>- some other recycled materials can be used in | | *Scale: (Recyclability)*<br>- no part of the material can be recycled<br>- other recycled materials cannot be used in | |
| **Personal Ethics** | Scale:<br>- Use of traffic barriers for co<br>- Designed with Strengths I-V, Extreme Events I&II, Service I-IV and Fatigue I&II<br>- include scour protection | *Scale:*<br>- proper use of shoring/rip rap during construction<br>- Designed with Strengths I-V, Extreme Events I&II, Service I-IV and Fatigue I&II<br>- include scour protection | *Scale:*<br>- Some traffic protection during construction<br>- Designed with Stength I, II, & III and Service I & II<br>- include some scour protecti | *Scale:*<br>- Some Use of shoring/rip rap<br>- Designed with Stength I, II, & III and Service I & III<br>- include some scour protection | *Scale:*<br>- No scaffolding used<br>-Designed with Strength I<br>- no scour protection considered | *Scale:*<br>- no shoring/rip rap used<br>-Designed with Strength I<br>- no scour protection considered |
| **Timeline** | *Scale:*<br>- 8 month construction period | *Scale:*<br>- 8 month construction period | *Scale:*<br>- 1 year constructin period | *Scale:*<br>- 1 year construction period | *Scale:*<br>- over 1 year contruction period | *Scale:*<br>- over 1 year constructino period |

Table 47 - Performance Rating Matrix

| Criteria | Unit Of Measure | Criteria Weight | Concept | Performance Rating | | | | | | | | | | Total Performance |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | |
| Life Expectancy | Years | 28.6 | No Build | | 2 | | | | | | | | | 57.2 |
| | | | Steel | | | | | | | 7 | | | | 200.2 |
| | | | Concrete | | | | | | | 7 | | | | 200.2 |
| Environmental Impact | Qualitative | 14.3 | No Build | | | | | | | | | 9 | | 128.7 |
| | | | Steel | | | | 4 | | | | | | | 57.2 |
| | | | Concrete | | | | | 5 | | | | | | 71.5 |
| Personal Ethics | Qualitative | 42.9 | No Build | 1 | | | | | | | | | | 42.9 |
| | | | Steel | | | | 4 | | | | | | | 171.6 |
| | | | Concrete | | | | | | | 7 | | | | 300.3 |
| Timeline | Months | 14.3 | No Build | | | | | | | | | | 10 | 143 |
| | | | Steel | | | | | | | | | 9 | | 128.7 |
| | | | Concrete | | | | | | | | | 9 | | 128.7 |

| Criteria | Unit Of Measure | low | | | | Rating | | | | | high |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Life Expectancy | Years | | | 50 | | 75 | | | 100 | | |
| Timeline | Months | 16 | 15 | 14 | 13 | 12 | 11 | 10 | 9 | 8 | 7 |

**Table 48 - Environmental Impact Subcategories Rating Matrix**

| Criteria | Concept | Performance Rating | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| Energy Consumption | No Build | | | | | | | | | | 10 |
| | Steel | | 2 | | | | | | | | |
| | Concrete | | | | | | 6 | | | | |
| CO2 Emissions | No Build | | | | | | | | | 9 | |
| | Steel | | 2 | | | | | | | | |
| | Concrete | | | | | | 6 | | | | |
| Other Emissions | No Build | | | | | | | | | | 10 |
| | Steel | | | 3 | | | | | | | |
| | Concrete | | | | 4 | | | | | | |
| Recyclability | No Build | | | | | | | | 8 | | |
| | Steel | | | | | | | | | 9 | |
| | Concrete | | | | 4 | | | | | | |

| | | |
|---|---|---|
| **Average** | No Build | 9 |
| | Steel | 4 |
| | Concrete | 5 |

**Table 49 - Performance Rating Matrix**

| Concept | Total Performance | Total Cost (millions of $) | Value Index (P/C) | % Value Improvement |
|---|---|---|---|---|
| No Build | 371.8 | - | - | |
| Steel | 557.7 | 3.527481 | 158.10 | |
| Concrete | 700.7 | 3.710339 | 188.85 | **19%** |

(Concrete is a better choice than steel by 19%)

# Appendix C: Superstructure Design

I. Pickets: 1.72 pickets @ 2.04 lbs each ∴ weight = 3.51 lbs ≅ 4 lbs to be conservat

$\frac{4 \text{ pickets}}{2.33 \text{ ft}} = \frac{x \text{ pickets}}{1 \text{ ft}}$ $x = 1.72$ pickets

1.5"

0.063" → 1.5"

$A_x = (1.5)(1.5) - (1.5-0.063)(1.5-0.063) = 0.185031$

$A_x = 0.185031 \text{ in}^2$

3.20833' = 38.5"

1.5" 1.5"

$\forall = (1.5)(1.5)(38.5) - (1.5-0.063)(1.5-0.063)(38.5)$

$\forall = 86.625 - 79.50$

** $\forall = 7.124 \text{ in}^3 = 0.004123 \text{ ft}^3$

Volume for 1.72 pickets = $0.004123 \text{ ft}^3 (1.72 \text{ pickets}) = 0.007091 \text{ ft}^3$

$= 12.25 \text{ in}^3$

unit weight of steel: $495 \text{ lb/ft}^3$

1 picket weight = $0.004123 \text{ ft}^3 (495 \text{ lb/ft}^3) = 2.041 \text{ lbs}$

∴ pickets weight / 1 ft = $\forall \gamma_s = 0.00709 \text{ ft}^3 (495) = 3.51 \text{ lbs}$

II. 1 x HSS 5 x 4 x 1/4 = 13.9 lb/ft

2 x HSS 5 x 5 x 1/4 = 15.6 lb/ft

$13.9 + 2(15.6) = 45.1 \text{ lbs}$

III. Post = 13.94 lb

W6 x 25 → 25 lb/ft

$\frac{1 \text{ post}}{6.5 \text{ ft}} = \frac{x \text{ post}}{1 \text{ ft}}$ ∴ 0.153846 post

3.625'

$3.625 \text{ ft} (25 \text{ lb/ft}) = 90.625 \text{ lbs}$

IV. Total:

I + II + III = 4 lbs + 45.1 lbs + 13.9 lbs = 63 lbs

→ because there are two railings, we multiply it by 2 to get:

$\boxed{126 \text{ lb/ft}}$

Figure 38 - Parapet Hand Calculations

| | A | B | C | D | E | F | G | H | I |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Girder spacing | 8 | ft | | | | | | |
| 2 | Top cover | 2.5 | in | Assumed initial value | | | | | |
| 3 | Bottom cover | 1 | in | Assumed initial value | | | | | |
| 4 | Slab compressive strength | 4 | ksi | | | | | | |
| 5 | Concrete density | 150 | pcf | | | | 87.5 | lb/ft | |
| 6 | Future wearing surface density | 140 | pcf | AASHTO Table 3.5.1-1 | | | 52.5 | lb/ft | |
| 7 | Deck width | 47 | ft | | | | | | |
| 8 | Deck depth (w/o wearing surface) | 7 | in | | | | | | |
| 9 | Roadway width | 32 | ft | | | | | | |
| 10 | Depth of asphalt | 4.5 | in | | | | | | |
| 11 | | | | | | | | | |
| 12 | Dead load of deck (w/o reinf. Steel) | 4112.5 | lb/ft | | 5918.5 | unfactored dead load w/o beam wt | | | |
| 13 | Dead load for asphalt | 1680 | lb/ft | | 986 | one sixth dead load w/o beam wt | | | |
| 14 | Dead load of railings | 126 | lb/ft | | 1875 | beam wt | | | |
| 15 | | | | | 17169 | Total unfactored load w/ beam wt (lb/ft) | | | |
| 16 | One-sixth dead load | 986 | lb/ft | | 1683 | Total wt of superstructure (k) | | | |
| 17 | | | | | | | | | |
| 18 | Dead load of girder sw (one girder) | 1875 | lb/ft | | | | | | |
| 19 | | | | | | | | | |
| 20 | Total dead load (unfactored) (one girder) | 2861 | lb/ft | | | | | | |
| 21 | Total dead load (factor = 1.2) | 3434 | lb/ft | | | | | | |

Figure 39 - Dead Load Calculation Spreadsheet

Distribution Factors:

Interior beams:

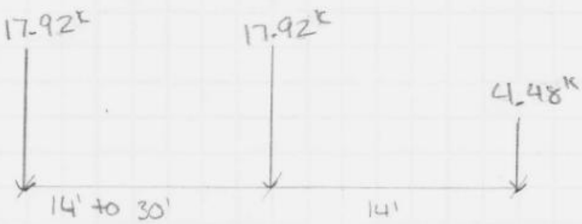
$$\left(\frac{S}{3.0}\right)^{0.35}\left(\frac{Sd}{12L^2}\right)^{0.25} = \left(\frac{8'}{3.0}\right)^{0.35}\left(\frac{(8')(45'')}{12(98')^2}\right)^{0.25} = 0.33$$

↓ one lane loaded

$$\left(\frac{S}{6.3}\right)^{0.6}\left(\frac{Sd}{12L^2}\right)^{0.125} = \left(\frac{8'}{6.3}\right)^{0.6}\left(\frac{(8')(45'')}{12(98')^2}\right)^{0.125} = 0.56$$

↓ two lanes loaded

Use a distribution factor of 0.56 to be conservative.




Design Lane Load:

$(0.56)(0.64\ k/ft) = 0.358^{k}/ft$

Live-Load Moment from Software: $M_u = 1261^{'k}$

Figure 40 - Interior Beam Distribution Factor Calculations

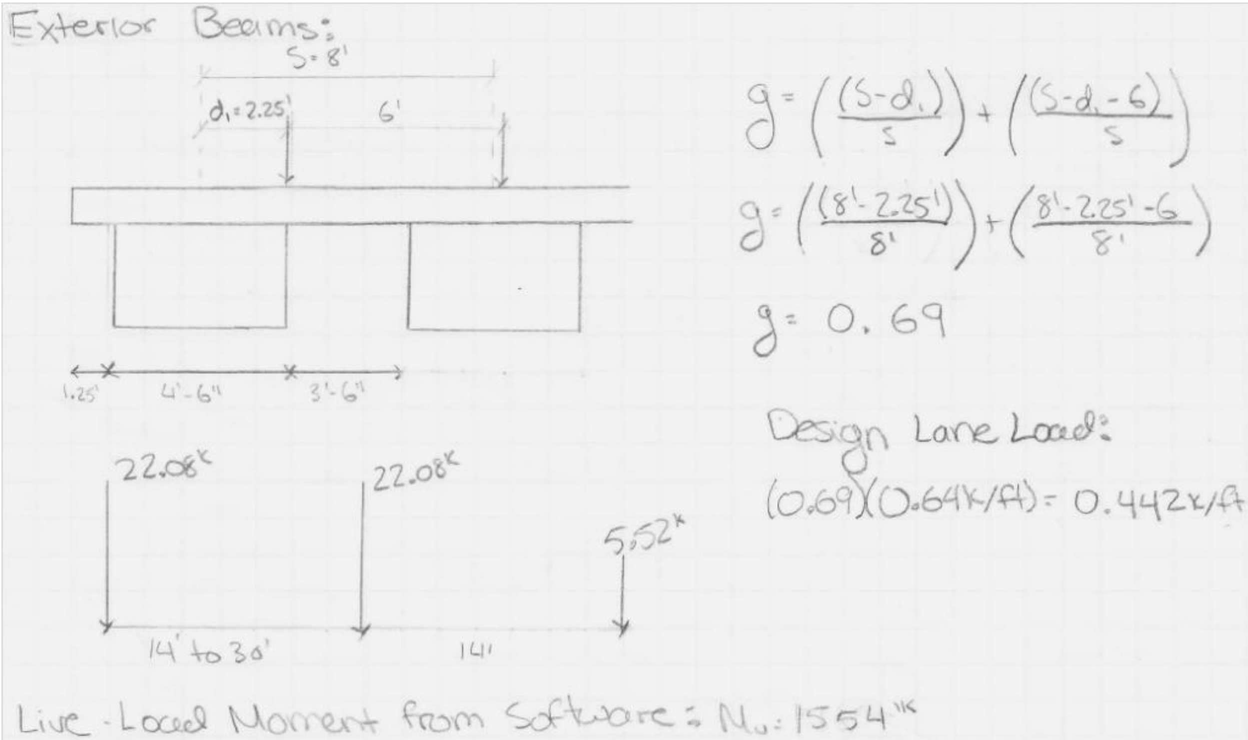



**Figure 41 - Exterior Beam Distribution Factor Calculations**

| | A | B | C |
|---|---|---|---|
| 1 | | | |
| 2 | Ml | 1261 | k-ft |
| 3 | f'c | 8000 | psi |
| 4 | f'ci | 6000 | psi |
| 5 | fci | -3600 | psi |
| 6 | fti | 232 | psi |
| 7 | fcs | -3600 | psi |
| 8 | fts | 537 | psi |
| 9 | R | 0.85 | |
| 10 | thickness | 12 | in |

**Figure 42 - Input Values for Interior Girders**

| E | F | G | H | I | J | K | L | M | N | O |
|---|---|---|---|---|---|---|---|---|---|---|
| b (in) | h (in) | I (void) (in^4) | c (void) (in) | S(void) (in^3) | beam wt (void) (lb/ft) | dead wt | M beam (void ) (k-ft) | M dead (k-ft) | S1 Limit (void) | S2 Limit (void) |
| 54 | 40 | 277760 | 20 | 13888 | 1750 | 986 | 2101 | 1184 | 14365 | 15168 |
| 54 | 41 | 297862 | 20.5 | 14530 | 1775 | 986 | 2131 | 1184 | 14460 | 15268 |
| 54 | 42 | 318816 | 21 | 15182 | 1800 | 986 | 2161 | 1184 | 14555 | 15368 |
| 54 | 43 | 340634 | 21.5 | 15843 | 1825 | 986 | 2191 | 1184 | 14650 | 15468 |
| 54 | 44 | 363328 | 22 | 16515 | 1850 | 986 | 2221 | 1184 | 14745 | 15568 |
| 54 | 45 | 386910 | 22.5 | 17196 | 1875 | 986 | 2251 | 1184 | 14840 | 15668 |
| 54 | 46 | 411392 | 23 | 17887 | 1900 | 986 | 2281 | 1184 | 14934 | 15768 |
| 54 | 47 | 436786 | 23.5 | 18587 | 1925 | 986 | 2311 | 1184 | 15029 | 15869 |
| 54 | 48 | 463104 | 24 | 19296 | 1950 | 986 | 2341 | 1184 | 15124 | 15969 |
| 54 | 49 | 490358 | 24.5 | 20015 | 1975 | 986 | 2371 | 1184 | 15219 | 16069 |
| 54 | 50 | 518560 | 25 | 20742 | 2000 | 986 | 2401 | 1184 | 15314 | 16169 |
| 54 | 51 | 547722 | 25.5 | 21479 | 2025 | 986 | 2431 | 1184 | 15409 | 16269 |
| 54 | 52 | 577856 | 26 | 22225 | 2050 | 986 | 2461 | 1184 | 15503 | 16369 |
| 54 | 53 | 608974 | 26.5 | 22980 | 2075 | 986 | 2491 | 1184 | 15598 | 16469 |
| 54 | 54 | 641088 | 27 | 23744 | 2100 | 986 | 2521 | 1184 | 15693 | 16570 |
| 54 | 55 | 674210 | 27.5 | 24517 | 2125 | 986 | 2551 | 1184 | 15788 | 16670 |
| 54 | 56 | 708352 | 28 | 25298 | 2150 | 986 | 2581 | 1184 | 15883 | 16770 |
| 54 | 57 | 743526 | 28.5 | 26089 | 2175 | 986 | 2611 | 1184 | 15978 | 16870 |
| 54 | 58 | 779744 | 29 | 26888 | 2200 | 986 | 2641 | 1184 | 16072 | 16970 |
| 54 | 59 | 817018 | 29.5 | 27696 | 2225 | 986 | 2671 | 1184 | 16167 | 17070 |
| 54 | 60 | 855360 | 30 | 28512 | 2250 | 986 | 2701 | 1184 | 16262 | 17170 |
| 54 | 61 | 894782 | 30.5 | 29337 | 2275 | 986 | 2731 | 1184 | 16357 | 17270 |
| 54 | 62 | 935296 | 31 | 30171 | 2300 | 986 | 2761 | 1184 | 16452 | 17371 |
| 54 | 63 | 976914 | 31.5 | 31013 | 2325 | 986 | 2791 | 1184 | 16547 | 17471 |
| 54 | 64 | 1019648 | 32 | 31864 | 2350 | 986 | 2821 | 1184 | 16641 | 17571 |
| 54 | 65 | 1063510 | 32.5 | 32723 | 2375 | 986 | 2851 | 1184 | 16736 | 17671 |
| 54 | 66 | 1108512 | 33 | 33591 | 2400 | 986 | 2881 | 1184 | 16831 | 17771 |
| 54 | 67 | 1154666 | 33.5 | 34468 | 2425 | 986 | 2911 | 1184 | 16926 | 17871 |
| 54 | 68 | 1201984 | 34 | 35352 | 2450 | 986 | 2941 | 1184 | 17021 | 17971 |
| 54 | 69 | 1250478 | 34.5 | 36246 | 2475 | 986 | 2971 | 1184 | 17116 | 18072 |

**Figure 43 - Calculation Steps for Multiple Cross-Section Dimensions for Interior Girders**

| | A | B | C | D | E | F | G | H | I |
|---|---|---|---|---|---|---|---|---|---|
| 35 | | | | | b (in) | h (in) | S1 (void) | S2 (void) | Governing Limit (void) |
| 36 | | | | | 54 | 40 | Bad | Bad | S_2 |
| 37 | | | | | 54 | 41 | Good | Bad | S_2 |
| 38 | | | | | 54 | 42 | Good | Bad | S_2 |
| 39 | | | | | 54 | 43 | Good | Good | S_2 |
| 40 | | | | | 54 | 44 | Good | Good | S_2 |
| 41 | Choose 54" x 45" because of | | | | 54 | 45 | Good | Good | S_2 |
| 42 | | | | | 54 | 46 | Good | Good | S_2 |
| 43 | | | | | 54 | 47 | Good | Good | S_2 |
| 44 | | | | | 54 | 48 | Good | Good | S_2 |
| 45 | | | | | 54 | 49 | Good | Good | S_2 |
| 46 | | | | | 54 | 50 | Good | Good | S_2 |
| 47 | | | | | 54 | 51 | Good | Good | S_2 |
| 48 | | | | | 54 | 52 | Good | Good | S_2 |
| 49 | | | | | 54 | 53 | Good | Good | S_2 |
| 50 | | | | | 54 | 54 | Good | Good | S_2 |
| 51 | | | | | 54 | 55 | Good | Good | S_2 |
| 52 | | | | | 54 | 56 | Good | Good | S_2 |
| 53 | | | | | 54 | 57 | Good | Good | S_2 |
| 54 | | | | | 54 | 58 | Good | Good | S_2 |
| 55 | | | | | 54 | 59 | Good | Good | S_2 |
| 56 | | | | | 54 | 60 | Good | Good | S_2 |
| 57 | | | | | 54 | 61 | Good | Good | S_2 |
| 58 | | | | | 54 | 62 | Good | Good | S_2 |
| 59 | | | | | 54 | 63 | Good | Good | S_2 |
| 60 | | | | | 54 | 64 | Good | Good | S_2 |
| 61 | | | | | 54 | 65 | Good | Good | S_2 |
| 62 | | | | | 54 | 66 | Good | Good | S_2 |
| 63 | | | | | 54 | 67 | Good | Good | S_2 |
| 64 | | | | | 54 | 68 | Good | Good | S_2 |
| 65 | | | | | 54 | 69 | Good | Good | S_2 |

**Figure 44 - Output Information for Various Cross-Sections for Interior Girders**

| | A | B | C |
|---|---|---|---|
| 1 | | | |
| 2 | MI | 1554 | k-ft |
| 3 | f'c | 8000 | psi |
| 4 | f'ci | 6000 | psi |
| 5 | fci | -3600 | psi |
| 6 | fti | 232 | psi |
| 7 | fcs | -3600 | psi |
| 8 | fts | 537 | psi |
| 9 | R | 0.85 | |
| 10 | thickness | 12 | in |

**Figure 45 - Input Values for Exterior Girders**

| E | F | G | H | I | J | K | L | M | N | O |
|---|---|---|---|---|---|---|---|---|---|---|
| b (in) | h (in) | I (void) (in^4) | c (void) (in) | S(void) (in^3) | beam wt (void) (lb/ft) | dead wt | M beam (void ) (k-ft) | M dead (k-ft) | S1 Limit (void) | S2 Limit (void) |
| 54 | 40 | 277760 | 20 | 13888 | 1750 | 986 | 2101 | 1184 | 15291 | 16145 |
| 54 | 41 | 297862 | 20.5 | 14530 | 1775 | 986 | 2131 | 1184 | 15386 | 16245 |
| 54 | 42 | 318816 | 21 | 15182 | 1800 | 986 | 2161 | 1184 | 15481 | 16345 |
| 54 | 43 | 340634 | 21.5 | 15843 | 1825 | 986 | 2191 | 1184 | 15576 | 16446 |
| 54 | 44 | 363328 | 22 | 16515 | 1850 | 986 | 2221 | 1184 | 15671 | 16546 |
| 54 | 45 | 386910 | 22.5 | 17196 | 1875 | 986 | 2251 | 1184 | 15765 | 16646 |
| 54 | 46 | 411392 | 23 | 17887 | 1900 | 986 | 2281 | 1184 | 15860 | 16746 |
| 54 | 47 | 436786 | 23.5 | 18587 | 1925 | 986 | 2311 | 1184 | 15955 | 16846 |
| 54 | 48 | 463104 | 24 | 19296 | 1950 | 986 | 2341 | 1184 | 16050 | 16946 |
| 54 | 49 | 490358 | 24.5 | 20015 | 1975 | 986 | 2371 | 1184 | 16145 | 17046 |
| 54 | 50 | 518560 | 25 | 20742 | 2000 | 986 | 2401 | 1184 | 16240 | 17147 |
| 54 | 51 | 547722 | 25.5 | 21479 | 2025 | 986 | 2431 | 1184 | 16334 | 17247 |
| 54 | 52 | 577856 | 26 | 22225 | 2050 | 986 | 2461 | 1184 | 16429 | 17347 |
| 54 | 53 | 608974 | 26.5 | 22980 | 2075 | 986 | 2491 | 1184 | 16524 | 17447 |
| 54 | 54 | 641088 | 27 | 23744 | 2100 | 986 | 2521 | 1184 | 16619 | 17547 |
| 54 | 55 | 674210 | 27.5 | 24517 | 2125 | 986 | 2551 | 1184 | 16714 | 17647 |
| 54 | 56 | 708352 | 28 | 25298 | 2150 | 986 | 2581 | 1184 | 16809 | 17747 |
| 54 | 57 | 743526 | 28.5 | 26089 | 2175 | 986 | 2611 | 1184 | 16903 | 17848 |
| 54 | 58 | 779744 | 29 | 26888 | 2200 | 986 | 2641 | 1184 | 16998 | 17948 |
| 54 | 59 | 817018 | 29.5 | 27696 | 2225 | 986 | 2671 | 1184 | 17093 | 18048 |
| 54 | 60 | 855360 | 30 | 28512 | 2250 | 986 | 2701 | 1184 | 17188 | 18148 |
| 54 | 61 | 894782 | 30.5 | 29337 | 2275 | 986 | 2731 | 1184 | 17283 | 18248 |
| 54 | 62 | 935296 | 31 | 30171 | 2300 | 986 | 2761 | 1184 | 17378 | 18348 |
| 54 | 63 | 976914 | 31.5 | 31013 | 2325 | 986 | 2791 | 1184 | 17473 | 18448 |
| 54 | 64 | 1019648 | 32 | 31864 | 2350 | 986 | 2821 | 1184 | 17567 | 18548 |
| 54 | 65 | 1063510 | 32.5 | 32723 | 2375 | 986 | 2851 | 1184 | 17662 | 18649 |
| 54 | 66 | 1108512 | 33 | 33591 | 2400 | 986 | 2881 | 1184 | 17757 | 18749 |
| 54 | 67 | 1154666 | 33.5 | 34468 | 2425 | 986 | 2911 | 1184 | 17852 | 18849 |
| 54 | 68 | 1201984 | 34 | 35352 | 2450 | 986 | 2941 | 1184 | 17947 | 18949 |
| 54 | 69 | 1250478 | 34.5 | 36246 | 2475 | 986 | 2971 | 1184 | 18042 | 19049 |

Figure 46 - Calculation Steps for Multiple Cross-Section Dimensions for Exterior Girders

| | E | F | G | H | I |
|---|---|---|---|---|---|
| 35 | b (in) | h (in) | S1 (void) | S2 (void) | Governing Limit (void) |
| 36 | 54 | 40 | Bad | Bad | S_2 |
| 37 | 54 | 41 | Bad | Bad | S_2 |
| 38 | 54 | 42 | Bad | Bad | S_2 |
| 39 | 54 | 43 | Good | Bad | S_2 |
| 40 | 54 | 44 | Good | Bad | S_2 |
| 41 | 54 | 45 | Good | Good | S_2 |
| 42 | 54 | 46 | Good | Good | S_2 |
| 43 | 54 | 47 | Good | Good | S_2 |
| 44 | 54 | 48 | Good | Good | S_2 |
| 45 | 54 | 49 | Good | Good | S_2 |
| 46 | 54 | 50 | Good | Good | S_2 |
| 47 | 54 | 51 | Good | Good | S_2 |
| 48 | 54 | 52 | Good | Good | S_2 |
| 49 | 54 | 53 | Good | Good | S_2 |
| 50 | 54 | 54 | Good | Good | S_2 |
| 51 | 54 | 55 | Good | Good | S_2 |
| 52 | 54 | 56 | Good | Good | S_2 |
| 53 | 54 | 57 | Good | Good | S_2 |
| 54 | 54 | 58 | Good | Good | S_2 |
| 55 | 54 | 59 | Good | Good | S_2 |
| 56 | 54 | 60 | Good | Good | S_2 |
| 57 | 54 | 61 | Good | Good | S_2 |
| 58 | 54 | 62 | Good | Good | S_2 |
| 59 | 54 | 63 | Good | Good | S_2 |
| 60 | 54 | 64 | Good | Good | S_2 |
| 61 | 54 | 65 | Good | Good | S_2 |
| 62 | 54 | 66 | Good | Good | S_2 |
| 63 | 54 | 67 | Good | Good | S_2 |
| 64 | 54 | 68 | Good | Good | S_2 |
| 65 | 54 | 69 | Good | Good | S_2 |

**Figure 47 - Output Information for Various Cross-Sections for Exterior Girders**

| | A | B | C |
|---|---|---|---|
| 12 | c1 | 22.5 | in |
| 13 | c2 | 22.5 | in |
| 14 | fcci | -1684 | psi |
| 15 | | | |
| 16 | Ac | 1800 | in2 |
| 17 | Pi | 3030859 | lb |
| 18 | | | |
| 19 | e | 16.5 | in |
| 20 | | | |
| 21 | Ap | 16.03629 | in2 |
| 22 | N | 73.89996 | strands |

Figure 48 - Prestressed Steel Output Values from Spreadsheet

Using 270 ksi steel strands :

$F_p = 0.7(270\text{ksi}) = 189\text{ ksi}$

$P_i = 3031$ kips

$A_p = \frac{3031^k}{189\text{ksi}} = 16.0\text{ in}^2$

Using 0.6" strands ($A_s = 0.217\text{ in}^2$)

$N = \frac{16.0\text{ in}^2}{0.217\text{ in}^2} = 74$ strands

Use 15 strands in 5 post-tensioning ducts for a total of 75 strands. (3 Tendons made of 5 strands each)

$A_d = 2(15\text{ strands} \times 0.217\text{ in}^2/\text{strand}) = 6.51\text{ in}^2$

$D = \sqrt{\frac{4A_d}{\pi}} = \sqrt{\frac{4(6.51\text{ in}^2)}{\pi}} = 2.88\text{ in}$ ← use a diameter of 3"

$A_d = \frac{\pi}{4}(3'')^2 = 7.07\text{ in}^2$

Figure 49 - Prestressed Steel Calculations

Section modulus limits:

$$S_1 \geq \frac{M_0 + M_d + M_l}{Rf_{ti} - f_{cs}}$$

$$S_2 \geq \frac{M_0 + M_d + M_l}{f_{ts} - Rf_{ci}}$$

$M_0$ = Moment due to the self weight of the girder (k-ft)

$M_d$ = Moment due to the service dead load of the structure (k-ft)

$M_l$ = Moment due to the service live load of the structure (k-ft)

R = effectiveness ratio (assumed to be 0.85 for this design)

$f_{ti}$ = Allowable tensile stress immediately after transfer (ksi)

$f_{cs}$ = Allowable compressive stress at service load (ksi)

$f_{ts}$ = Allowable tensile stress at service load (ksi)

$f_{ci}$ = Allowable compressive stress immediately after transfer (ksi)

Concrete centroidal stress:

$$f_{cci} = f_{ti} - \frac{c_1}{h}(f_{ti} - f_{ci})$$

$$h = c_1 + c_2$$

$f_{cci}$ = Concrete centroidal stress (ksi)

$c_1$ = Distance from centroid to top flange (in)

$c_2$ = Distance from centroid to bottom flange (same as $c_1$ for this design) (in)

Initial prestressed force:

$$P_i = A_c f_{cci}$$

$P_i$ = Initial prestressed force (k)

$A_c$ = Cross-sectional area of member ($in^2$)

Eccentricity:

$$e = (f_{ti} - f_{cci})\frac{S}{P_i}$$

e = Eccentricity of prestressed steel

S = Section modulus of member

Area of prestressed steel:

$$A_p = \frac{P_i}{\emptyset f_t}$$

$A_p$ = Required area of prestressed steel ($in^2$)

ϕ = Reduction factor (0.70 for this design)

$f_t$ = Available tensile strength from prestressed strands (ksi)

Number of prestressed strands:

$$N = \frac{A_p}{A_s}$$

N = Required number of steel strands

$A_s$ = Cross-sectional area of a single steel strand

# Appendix D: Substructure Design Equations

## *Substructure Equation Process*

A) Site and Material Assumption Symbols

**Table 50 - Site and Material Assumption Symbols**

| Assumption | Symbol | Unit |
|---|---|---|
| Specific Gravity soil | $G_S$ | - |
| unit weight of concrete | $\gamma_c$ | pcf |
| Unit weight water | $\gamma_w$ | pcf |
| Unit weight dry soil | $\gamma_d$ | pcf |
| unit weight | $\gamma$ | pcf |
| effective friction method | $\phi'$ | o |
| soil foundation interface friction angle | $\phi_f$ | o |
| Poisson's Ratio | $v$ | - |
| Soil modulus of elasticity | $E_S$ | psi |
| coefficient of lateral earth pressure | $K$ | - |
| effective cohesion | $c'=c_T=S_u$ | psf |
| inclination of ground surface above the wall | $\beta$ | o |
| coefficient of lateral earth pressure at rest | $K_o$ | - |
| active coefficient of lateral earth pressure | $K_a$ | - |
| passive coefficient of lateral earth pressure | $K_p$ | - |
| over consolidation ratio | OCR | - |
| strength reduction/resistance factor | $\phi$ | - |
| strength of steel | $f_y$ | psi |
| strength of concrete | $f'_c$ | psi |
| unit weight of asphalt | $\gamma_{asphalt}$ | pcf |
| modulus of rigidity of concrete | $G_c$ | psi |

B) Dimension Assumption Symbols

Table 51 - Dimension Assumption Symbols

| Dimension | Symbol | Unit |
|---|---|---|
| pile cap base width | b | ft |
| pile cap height | $d_s$ | ft |
| abutment wall height | H | ft |
| stem wall height (to top of bridge seat) | $h_s$ | ft |
| backwall height | $h_b$ | ft |
| height of top of footing to water surface | d | ft |
| stem width | w | ft |
| bridge seat width | s | ft |
| backwall width | $w_b$ | ft |
| pile cap length | l | ft |
| length of abutment wall | L | ft |
| pile depth | D | ft |
| pile diameter | B | ft |
| pile area of face | $A_F$ | ft |
| diameter of pile | $d_{pile}$ | in |
| number of rows of piles | m | - |
| number of piles per row | n | - |
| pile spacing – edge spacing | $s_e$ | ft |
| pile spacing - interior spacing | $s_i$ | ft |
| length of girders | $L_g$ | ft |
| area of flexure reinforcement | $A_s$ | $in^2$ |
| diameter of reinforcement | $d_{bar}$ | in |
| roadway width | $w_{road}$ | feet |
| depth of roadway asphalt | $d_{asph}$ | feet |
| girder spacing | $l_g$ | ft |
| area of shear reinforcement | $A_v$ | $in^2$ |
| number of girders | $N_{girder}$ | - |

C) Preliminary design equation process to find the ultimate bearing capacity:

a. Unit weight prime

$\gamma' = \gamma - \gamma_w$

b. Void ratio

$$e = \frac{G_s \gamma_w}{\gamma_d}$$

c. Pore water pressure at depth *d*

$u = \gamma_w d$

d. Vertical effective stress at d

$\sigma'_{zd} = \gamma d - u$

e. Ultimate bearing capacity

$Q_{ult.} = c' N_c + \sigma'_{zd} N_q + 0.5 \gamma' b N_\gamma$

D) Original deep pile and final pile design equation process to find the allowable bearing capacity:

a. Pore water pressure at depth D

$$u_D = \gamma_w D$$

b. Vertical effective stress at depth D

$$\sigma'_{zD} = \gamma D - u$$

c. Coefficient of lateral earth pressure

$$K_o = (1 - sin\phi')OCR^{sin\phi'}$$

d. Bearing capacity factor $N_\sigma$*

$$N_\sigma^* = \frac{3}{3 - sin\phi'} e^{\frac{(90-\phi')\pi}{180}}$$

e. Bearing capacity factor $N_{q*}$

$$N_q^* = \frac{(1 + 2K_o)N_\sigma^*}{3}$$

f. Bearing capacity factor $N_\gamma$*

$$N_\gamma^* = 0.6\left(N_q^* - 1\right)tan\phi'$$

g. Unit side friction resistance

$$f_s = \frac{K_o \sigma'_{zD} K \phi_f}{K_o \phi'}$$

h. Toe load resistance

$$P_t = (B\gamma' N_\gamma^* + \sigma'_{zD} N_q^*)A_F$$

i. Side friction

$$P_s = d_{pile} D f_s$$

j. Weight of pile cap

$$w_{pc} = whl\gamma_c$$

k. Weight of pile cap per pile

$$w_{\frac{pc}{N}} = \frac{w_{pc}}{mn}$$

l. Allowable pile load capacity

$$P_a = \frac{P_t + P_s - w_{\frac{pc}{N}}}{F}$$

m. Group efficiency factor

$$\eta = 1 - \tan^{-1}(\tfrac{B}{s})\frac{(n-1)m + (m-1)n}{90mn}$$

n. Allowable group pile load capacity

$$P_{ag} = \eta P_a mn$$

o. Passive normal force acting between soil and unit length of abutment wall

$$P_o = \gamma' \frac{(H+h)^2 K_o}{2}$$

p. Active normal force acting between soil and unit length of abutment wall

$$P_{act} = \gamma' \frac{(H+h)^2 K_a cos\beta}{2}$$

q. Total moment about abutment wall

$$M_{aw} = P_0 - P_a$$

r. Axial reaction force in piles resulting from moment

$$P_i = \frac{M_{aw}}{2s_i}$$

s. Load capacity per pile:

   i. Pile 1

   $$P_1 = P_a + P_i$$

   ii. Pile 2

   $$P_2 = P_a$$

   iii. Pile 3

   $$P_3 = P_a - P_i$$

t. End bearing capacity per pile

$$q_t = B\gamma' N_\gamma^* + \sigma'_{zD} N_q^*$$

u. Dead Load (unfactored) of pile cap

$$DL_{pc} = \frac{bhl\gamma_c}{bl} = h\gamma_c$$

v. Allowable pile bearing capacity

$$Q_{allowable} = q_t + f_s - DL_{pc}$$

w. Ultimate group allowable bearing capacity

$$Q_{ult} = \eta(Q_{allowable})mn$$

E) Pile cap reinforcing steel design equation process:

a. Maximum factored load of superstructure from an interior girder
$P_u = 1.2\text{DL}_{girder} + 1.6LL_{girder}$

b. Factored load due to self-weight of pile cap and diaphragm
$\omega_u = 1.2DL_{sub}$

c. Positive moment about the pile cap due to the superstructure and substructure
$M_u = 0.8(\frac{P_u s_i}{4} + \frac{\omega_u s_i^2}{8})$

d. Neutral axis in pile cap
$a = \frac{A_s f_y}{f_c' b}$

e. Effective depth of pile cap
$d_e = d_s - 6 - (d_s - d') - \frac{d_{bar}}{2}$

f. Nominal moment about pile cap for reinforcement design
$M_n = A_s f_y (d_e - {}^{a}/_{2})$

g. Design moment capacity
$M_r = \phi M_n$

h. Check that the design moment is greater than the positive moment around the pile cap for the steel reinforcement design to be acceptable
$M_r > 4/3 M_u$

i. Maximum factored shear
$V_u = P_u + \frac{\omega_u s_i}{2}$

j. Effective depth for shear calculations
$d_v = d_e - \frac{a}{2}$

k. Nominal shear capacity
$V_c = 2\sqrt{f_c'} b d_v$

l. Design shear capacity
$V_r = \phi V_c$

m. Check if steel reinforcement is not necessary for the assumed depth of the pile cap by the maximum factored shear being less than half of the design shear capacity
$V_u < 0.5V_r$

n. Check if the effective depth is adequate for shear design
$V_u < 6\sqrt{f_c'}bd_v$

o. Capacity of stirrups
$V_s = A_v f_y \dfrac{d_v}{s_s}$

p. Design capacity of stirrups
$V_n = \phi V_s$

q. Check if the design capacity of the stirrups and concrete shear capacity exceeds the maximum factored shear
$V_n \geq V_u - V_r$

F) Backwall reinforcing steel design equation process:

a. Maximum factored girder reaction

$$P_{str-I} = 1.2\left(DL_{super}\right) + 1.6\left(LL_{super}\right)$$

b. Backwall weight

$$\omega_b = w_b h_b \gamma_c$$

c. Approach slab weight

$$\omega_{slab} = w_{road} d_{asph} \gamma_{asph}$$

d. Future wearing surface

$$FWS = \frac{w_{road} d_{asph} \gamma_{asph}}{2}$$

e. Approach slab lane load (ASSHTO standard)

$$\omega_{lane} = 640\ lb/ft$$

f. Maximum factored load due to dead load of self-weight of diaphragm

$$\omega_{str.-I} = 1.25(\omega_b + \omega_{slab}) + 1.5FWS + 1.75\frac{\omega_{lane} N_{lanes}}{L}$$

g. Ultimate maximum positive moment

$$M_u = 0.8\left(\frac{P_{str.-I} s_i}{4} + \frac{\omega_{str.-I} s_i^2}{8}\right)$$

h. Neutral axis in backwall

$$a = \frac{A_s f_y}{f_c' b}$$

i. Effective depth of backwall

$$d_e = d_s - (d_s - d') - \frac{d_{bar}}{2}$$

j. Nominal moment

$$M_n = A_s f_y\left(d_e - {}^{a}/_{2}\right)$$

k. Design moment capacity

$$M_r = \phi M_n$$

l. Check that the design moment is greater than the positive moment around the backwall for the steel reinforcement design to be acceptable

$$M_r > 4/3 M_u$$

m. Maximum factored shear

$$V_u = P_u + \frac{\omega_u s_i}{2}$$

n. Effective depth for shear calculations

$$d_v = d_e - \frac{a}{2}$$

o. Nominal shear capacity

$$V_c = 2\sqrt{f_c'}bd_v$$

p. Design shear capacity

$$V_r = \phi V_c$$

q. Check if steel reinforcement is not necessary for the assumed depth of the pile cap by the maximum factored shear being less than half of the design shear capacity

$$V_u < 0.5V_r$$

r. Check if the effective depth is adequate for shear design

$$V_u < 6\sqrt{f_c'}bd_v$$

s. Capacity of stirrups

$$V_s = A_v f_y \frac{d_v}{s_s}$$

t. Design capacity of stirrups

$$V_n = \phi V_s$$

G) Bridge seat reinforcing steel design equation process:

a. Maximum factored girder reaction
$P_u = 1.2P$

b. Maximum factored load due to dead load of self-weight of diaphragm
$\omega_u = 1.6LL_{girder}$

c. Ultimate maximum positive moment
$$M_u = 0.8\left(\frac{P_u s_i}{4} + \frac{\omega_u s_i^2}{8}\right)$$

d. Neutral axis in backwall
$$a = \frac{A_s f_y}{f_c' b}$$

e. Effective depth of backwall
$$d_e = d_s - (d_s - d') - \frac{d_{bar}}{2}$$

f. Nominal moment
$M_n = A_s f_y\left(d_e - {}^{a}/_{2}\right)$

g. Design moment capacity
$M_r = \phi M_n$

h. Check that the design moment is greater than the positive moment around the backwall for the steel reinforcement design to be acceptable
$M_r > 4/3M_u$

i. Maximum factored shear
$$V_u = P_u + \frac{\omega_u s_i}{2}$$

j. Effective depth for shear calculations
$$d_v = d_e - \frac{a}{2}$$

k. Nominal shear capacity
$V_c = 2\sqrt{f_c'}bd_v$

l. Design shear capacity
$V_r = \phi V_c$

m. Check if steel reinforcement is not necessary for the assumed depth of the pile cap by the maximum factored shear being less than half of the design shear capacity
$V_u < 0.5V_r$

n. Check if the effective depth is adequate for shear design
$V_u < 6\sqrt{f_c'}bd_v$

o. Capacity of stirrups
$$V_s = A_v f_y \frac{d_v}{s_s}$$

p. Design capacity of stirrups
$V_n = \phi V_s$

H) Elastomeric bearing design equation process:

a. Temperature movement

$$\Delta_t = 1.5\alpha_c LT$$

b. Minimum thickness of elastomeric bearing pad

$$t_{min} = 2\Delta_T$$

c. Length of trial pad

$$L_p = \frac{DL_{girder} + LL_{girder}}{12w_{bf}}$$

d. Assumed pad length

$$L_a$$

e. Trial pad dimensions

$$L_a \times w_{bf} \times t_{min}$$

f. Check maximum thickness of pad

$$\frac{1}{3}L_a > t_{min}$$

g. Check compressive stress is less than pad capacity

$$P_{\max\ load} < \frac{DL}{w_{bf}L_a}$$

h. Shape factor

$$\xi = \frac{w_{bf}L_a}{w_{bf} + L_a}$$

i. Calculate for final thickness of pad (round to the nearest 1/4”)

$$t_f = \left| t_{min} - \frac{(t_{min}\xi)}{100} \right| - \frac{t_{min}\xi}{400}$$

j. Maximum shear calculation

$$V_{max} = \frac{DL_{super}}{6}$$

k. Design shear calculation

$$V_D = \frac{GL_a w_{bf}\Delta_T}{t_f}$$

l. Check if the design shear exceeds the maximum shear
$V_D > V_{max}$

m. If the design shear did not exceed the maximum shear, choose a greater thickness than determined in step *i* and repeat steps *j-l*

n. Repeat step *m* until
$V_D > V_{max}$

# Appendix F: Substructure Spreadsheet Calculations and Drawings

## *Substructure Results*

1. Preliminary Design

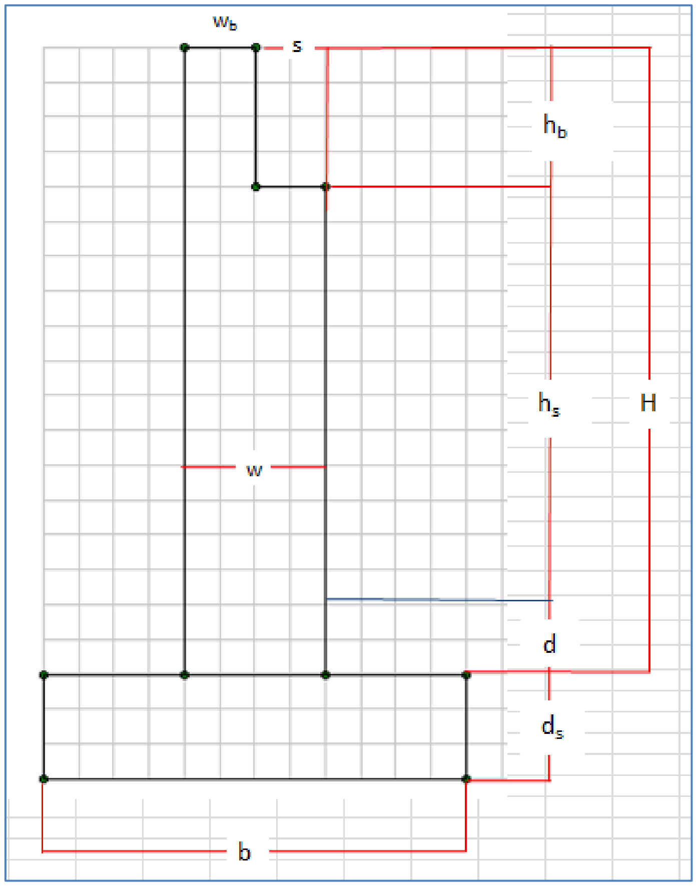



Figure 50 - N-S Elevation View of Preliminary Design

Table 52 - Dimensions, Load and Bearing Capacity of Preliminary Design

| Variable | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| pile cap base | $b$ | 12 | ft | assumption |
| pile cap height | $d_s$ | 3 | ft | assumption |
| abutment wall height | $H$ | 18 | ft | assumption |
| stem wall height (to bridge seat) | $h_s$ | 14 | ft | assumption |
| backwall height | $h_b$ | 4 | ft | assumption |
| height of top of footing to water surface | $d$ | 4 | ft | assumption |
| stem width | $w$ | 4 | ft | assumption |
| bridge seat | $s$ | 2.5 | ft | assumption |
| backwall width | $w_b$ | 1.5 | ft | assumption |
| length of pile | $l$ | 52 | ft | assumption |
| length of abutment wall | $L$ | 48 | ft | assumption |
| weight of substructure | $W_{sub}$ | 727.20 | kips | calculation |
| DL of substructure | $DL_{sub}$ | 3.15 | ksf | calculation |
| ultimate bearing capacity of footing | $q_{ult}$ | 1.71 | ksf | calculation |

Conclusion:

$DL \gg q_{ult}$ $\therefore$ preliminary design is not adequate

2. Original Deep Pile Substructure Design

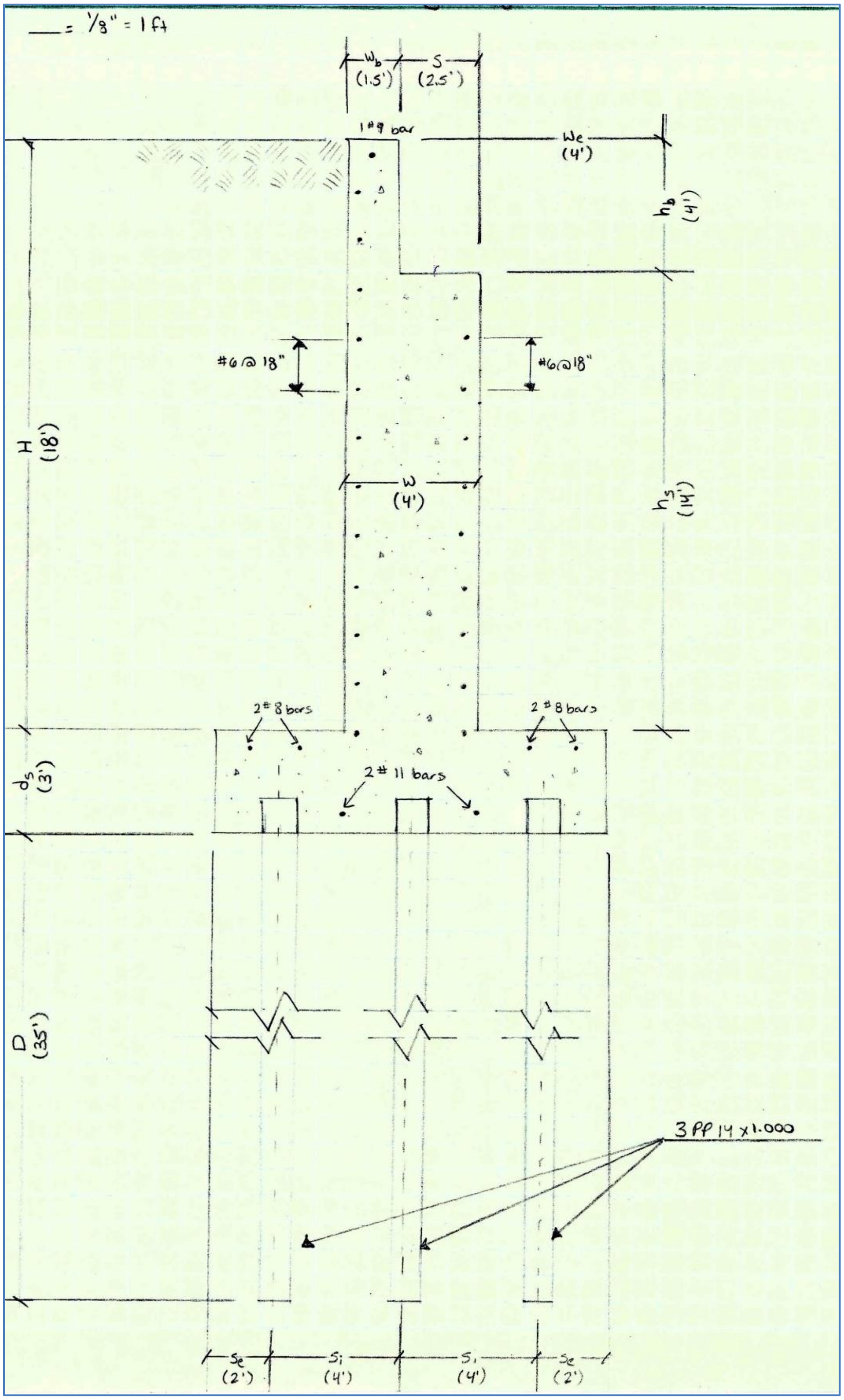



Figure 51 - N-S Elevation of Substructure Original Deep Pile Substructure Design (1/8" = 1')

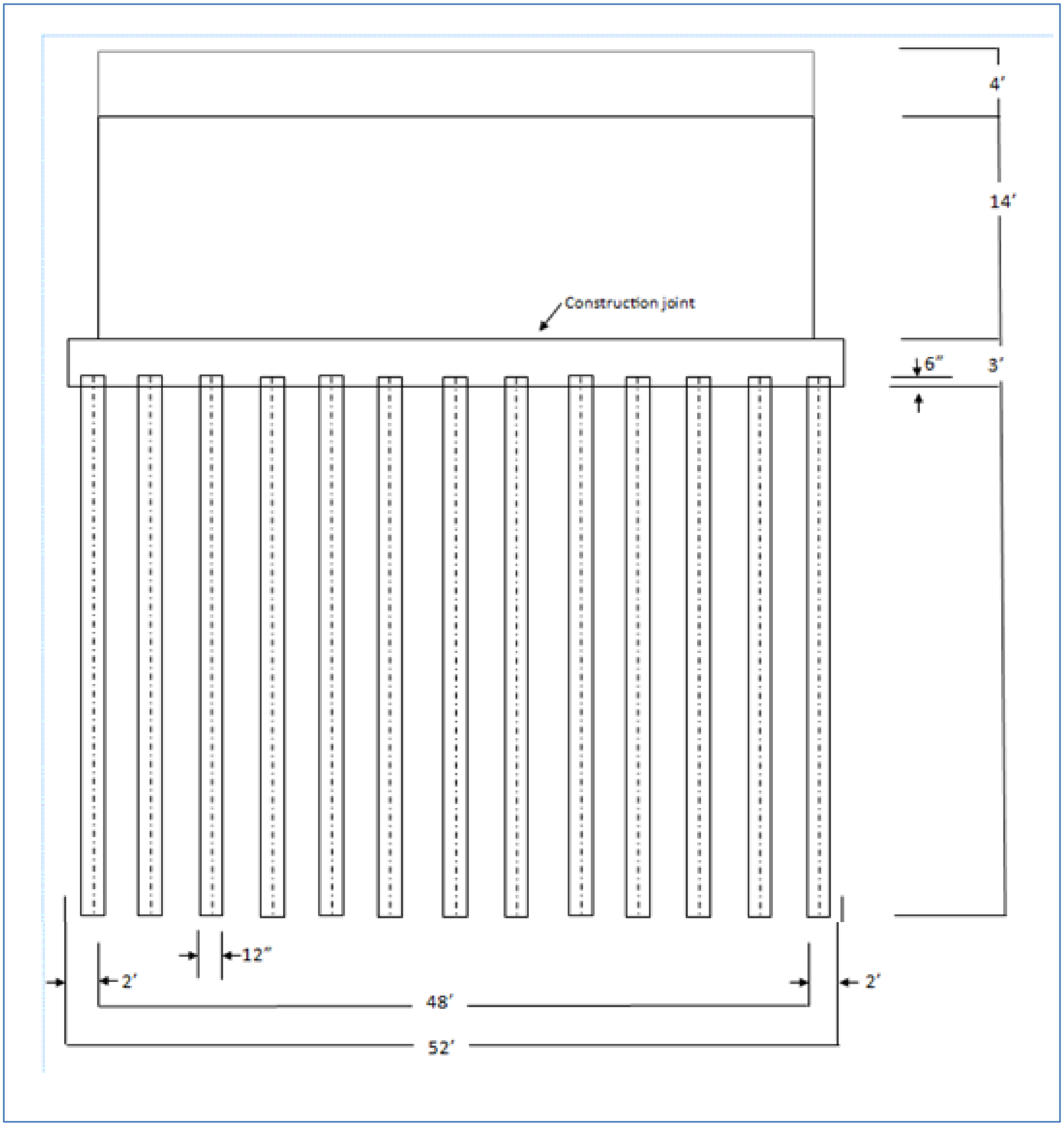



Figure 52 - E-W Elevation View of Substructure Original Deep Pile Substructure Design

a. Pile group assumptions and calculations

Table 53 - Pile Group Dimensions, Load and Bearing Capacity of Original Deep Pile Substructure Design

| Variable | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| Specific Gravity soil | $G_S$ | 2.7 | - | assumption |
| unit weight of concrete | $\gamma_c$ | 150 | pcf | assumption |
| Unit weight water | $\gamma_w$ | 62.4 | pcf | standard |
| Unit weight dry soil | $\gamma_d$ | 120 | pcf | assumption |
| unit weight | $\gamma$ | 133 | pcf | assumption |
| unit weight prime | $\gamma'$ | 70.6 | pcf | calculation |
| void ratio | e | 1.404 | - | calculation |
| pore water pressure | u | 2184 | psf | calculation |
| effective friction method | $\phi'$ | 30 | o | assumption |
| soil foundation interface friction angle | $\phi_f$ | 21 | o | assumption |
| - | $\phi_f/\phi'$ | 0.7 | - | calculation |
| Poisson's Ratio | v | 0.3 | - | assumption |
| Soil modulus of elasticity | $E_S$ | 7300 | psi | assumption |
| | $\tan(\phi')$ | 0.57735 | - | calculation |
| | $\sin(\phi')$ | 0.5 | - | calculation |
| | $\tan^2(45+\phi'/2)$ | 3 | - | calculation |
| Rigidity Index | $I_R$ | 1.968056 | psi | calculation |
| coefficient of lateral earth pressure at rest | $K_o$ | 0.5 | - | calculation |
| coefficient of lateral earth pressure | K | 0.5 | - | assumption |
| bearing capacity factor $N^*_y$ | $N^*_y$ | 2.8592 | - | Terzaghi Method |
| bearing capacity factor $N^*_q$ | $N^*_q$ | 9.253798 | - | Terzaghi Method |
| bearing capacity factor $N^*_\sigma$ | $N^*_\sigma$ | 13.8807 | - | Terzaghi Method |
| effective cohesion | $c'=c_T=S_u$ | 250 | psf | assumption |
| unit side friction resistance | $f_s$ | 864.85 | psf | calculation |
| pile shape factor | $K_{shape}$ | 1 | - | assumption |
| average shear stress | $\tau'_{average}$ | 1.081985 | kips | calculation |
| pile cap width | w | 12 | ft | assumption |
| pile cap height | h | 3 | ft | assumption |

| | | | | |
|---|---|---|---|---|
| pile cap length | l | 52 | ft | assumption |
| volume of pile cap | V | 1872 | $ft^3$ | calculation |
| weight of pile cap | W | 280800 | lb | calculation |
| group efficiency factor | η | 0.71279 | - | calculation |
| number of piles in group | N | 39 | - | calculation |
| number of rows of piles | m | 3 | - | assumption |
| number of piles per row | n | 13 | - | assumption |
| $\tan^{-1}(B/s)$ | θ | 16.26 | o | calculation |
| center to center pile spacing | $s_i$ | 4 | ft | assumption |
| pile length | D | 35 | ft | assumption |
| height from top of footing to ground | $z_w$ | 4 | ft | assumption |
| pile diameter | B | 1.16667 | ft | assumption |
| steel pile area (PP12.75x0.375) | $A_S$ | 0.28361 | $ft^2$ | assumption |
| diameter of pile | $d_{pile}$ | 14 | in | assumption |
| surface area of pile | SA | 128.46 | $ft^2$ | calculation |
| net unit toe bearing resistance | $q_t$ | 23.10 | ksf | calculation |
| toe load resistance | $P_t$ | 24.73 | kips | calculation |
| side friction | $P_s$ | 111.10 | kips | calculation |
| weight of pile cap | $w_c$ | 280.80 | kips | calculation |
| number of piles | η | 39.00 | - | calculation |
| weight of pile cap on each pile | $w_{pc/N}$ | 7.200 | kips | calculation |
| factor of safety | F | 3.50 | - | Coduto book assumption |
| allowable load capacity | $P_a$ | 36.75 | kips | calculation |
| bearing capacity due to friction | $f_s$ | 0.86 | ksf | calculation |
| weight of pile cap | $DL_{PC}$ | 0.45 | ksf | calculation |
| allowable bearing capacity per pile | $Q_{allowable}$ | 23.52 | ksf | calculation |
| allowable group load capacity | $P_{ag}$ | 1021.63 | kips | calculation |
| ultimate group bearing capacity | $Q_{ult}$ | 653.73 | ksf | calculation |

Conclusion:

$DL_{PC} \ll Q_{ult}$ ∴ a pile design is adequate for the design

b. Pile cap reinforcement assumptions and calculations

**Table 54 - Pile Cap Reinforcement Dimensions, Load and Bearing Capacity of Original Deep Pile Substructure Design**

| Variable | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| pile spacing - interior spacing | $s_i$ | 4 | ft | assumption |
| length of girders | $L_g$ | 98 | ft | assumption |
| reinforcing bars | - | 2 #11 bars | - | assumption |
| area of rebar | $A_s$ | 3.12 | $in^2$ | assumption |
| diameter of bar | $d_{bar}$ | 1 | in | assumption |
| pile cap height (depth) | $d_s$ | 36 | in | assumption |
| bottom cover | $d_s$-d' | 3 | in | assumption |
| width of pile cap for design | $b_d$ | 12 | in | assumption |
| stirrups | - | 1 # 5 bar | - | assumption |
| area of stirrups - | $A_v$ | 1 | $in^2$ | assumption |
| spacing of stirrups | $s_s$ | 10 | in | assumption |
| strength resistance factor | $\phi$ | 0.9 | - | ACI |
| total dead load of superstructure | $DL_{sup}$ | 1683 | kip | final superstructure calculations |
| maximum factored load of superstructure from an interior girder | $P_u$ | 141.73 | kip | final superstructure calculations |
| total DL of substructure | $DL_{sub}$ | 727.2 | kips | calculation |
| factored load due to the self weight of the pile cap and diaphragm | $\omega_u$ | 1.09 | kips | calculation |
| positive moment for flexural design | $M_u$ | 115.13 | kips-ft | calculation |
| effective depth | $d_e$ | 32.5 | in | calculation |
| neutral axis | a | 3.9 | in | calculation |
| nominal moment | $M_n$ | 5719.00 | kip-ft | calculation |
| design capacity | $M_r$ | 5147.06 | kip-ft | calculation |
| check: $M_r > M_u$ | - | 5031.93 | kip-ft | calculation - OK (+ difference) |
| maximum factored shear | $V_u$ | 143.91 | kips | calculation |
| effective shear depth | $d_v$ | 30.55 | in | calculation |
| nominal shear | $V_c$ | 49.33 | kips | ACI |
| design capacity | $V_r$ | 37.00 | kips | calculation |
| check: $V_r > V_u$ | - | -106.91 | kips | calculation - BAD (- difference) |

| | | | | |
|---|---|---|---|---|
| check if: $V_u \leq 0.5V_r$ for no needed stirrups | - | 125.413 | kips | calculation - BAD (+ difference) |
| check if: $V_u \leq 6\sqrt{f'_c}bd_v$ | - | -394.88 | kips | calculation- OK ( - difference) |

Conclusions: completed as routine

$M_u < M_r$ ∴ flexural reinforcement is adequate

$V_u < V_r$ ∴ shear reinforcement is adequate

c. Backwall reinforcement assumptions and calculations

Table 55 - Backwall Reinforcement Dimensions, Load and Bearing Capacity of Original Deep Pile Substructure Design

| Variable | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| pile spacing - interior spacing | $s_i$ | 4 | ft | assumption |
| length of girders | $L_g$ | 98 | ft | assumption |
| reinforcing bars | - | 1 #9 bars | - | assumption |
| area of rebar | $A_s$ | 1 | $in^2$ | assumption |
| diameter of bar | $d_{bar}$ | 1 | in | assumption |
| bottom cover | $d_s$-d' | 3 | in | assumption |
| backwall width | $w_b$ | 1.5 | ft | assumption |
| area of stirrups | $A_v$ | 0.62 | $in^2$ | assumption |
| spacing of stirrups | $s_s$ | 10 | in | assumption |
| abutment length | L | 48 | ft | assumption |
| number of lanes | $N_{lanes}$ | 2 | - | assumption |
| roadway width | $w_{road}$ | 32 | feet | assumption |
| depth of roadway asphalt | $d_{asph}$ | 0.375 | feet | assumption |
| abutment wall height | H | 18 | feet | assumption |
| backwall height | $h_b$ | 4 | feet | assumption |
| strength resistance factor | $\phi$ | 0.9 | - | ACI |
| strength of steel | $f_y$ | 60000 | psi | ACI |
| strength of concrete | $f'_c$ | 4000 | psi | ACI |
| shear multiplication factor | $\lambda$ | 2 | - | ACI |
| unit weight of asphalt | $\gamma_{asphalt}$ | 140 | pcf | standard |
| dead load of superstructure | $DL_{sup}$ | 1683 | kip | final superstructure calculations |
| back wall weight | $\omega_{backwall}$ | 600 | lb/ft | calculation |
| approach slab weight | $\omega_{slab}$ | 1680 | lb/ft | calculation |
| future wearing surface weight | *FWS* | 840 | lb/ft | calculation |
| approach slab lane load | $\omega_{lane}$ | 640 | lb/ft | AASHTO |
| factored load due to the self weight of the pile cap and diaphragm | $\omega_u$ | 4156.667 | lbs | calculation |
| maximum factored load of superstructure from an interior girder | $P_u$ | 141730.6 | kip | final superstructure calculations |

| | | | | |
|---|---|---|---|---|
| positive moment for flexural design | $M_u$ | 120035.1 | kips-ft | calculation |
| effective depth | $d_s$ | 212.5 | in | calculation |
| neutral axis | a | 24.19355 | in | calculation |
| nominal moment | $M_n$ | 12024194 | kip-ft | calculation |
| design capacity | $M_r$ | 10821774 | kip-ft | calculation |
| check: $M_r > M_u$ | - | 10701739 | kip-ft | calculation - OK (+ difference) |
| check: $-M_r > -4/3M_u$ | - | 10661727 | kip-ft | calculation - OK (+ difference) |
| maximum factored shear | $V_u$ | 150043.9 | kips | calculation |
| effective shear depth | $d_v$ | 200.4032 | in | calculation |
| nominal shear | $V_c$ | 16665.2 | kips | ACI |
| design capacity | $V_r$ | 12498.9 | kips | calculation |
| check: $V_r > V_u$ | - | -137545 | kips | calculation - BAD (- difference) |
| check if: $V_u \leq 0.5V_r$ for no needed stirrups | - | 143794.4 | kips | calculation - BAD (+ difference) |

Conclusions: completed as routine

$M_u < M_r$ ∴ flexural reinforcement is adequate

$V_u > V_r$ ∴ shear reinforcement is inadequate

d. Bridge seat reinforcement assumptions and calculations

Table 56 - Bridge Seat Reinforcement Dimensions, Load and Bearing Capacity of Original Deep Pile Substructure Design

| Variable | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| reinforcing bars - stem wall | - | 12 #6 bars | - | assumption |
| area of rebar - stem wall | $A_s$ | 5.28 | $in^2$ | assumption |
| diameter of bar | $d_{bar}$ | 0.75 | in | assumption |
| bottom cover | $d_s$-d' | 3 | in | assumption |
| stem width | w | 4 | feet | assumption |
| area of stirrups | $A_v$ | 0.62 | $in^2$ | assumption |
| abutment wall height | H | 18 | feet | assumption |
| girder spacing | l | 5 | ft | assumption |
| vertical bar diameter | $d_{vert}$ | 0.625 | in | assumption |
| height of wall for design | b | 12 | in | assumption |
| passive earth pressure | $\omega_p$ | 58414.31 | lb/ft | calculation |
| distributed load factor | $\omega_u$ | 87621.47 | lb/ft | calculation |
| positive moment for flexural design | $M_u$ | 273817.1 | lb-ft | calculation |
| effective depth | $d_e$ | 44 | in | calculation |
| neutral axis | a | 7.764706 | in | calculation |
| nominal moment | $M_n$ | 12709271 | lb-ft | calculation |
| design capacity | $M_r$ | 11438344 | lb-ft | calculation |
| check: $M_r > M_u$ | - | 11164526 | lb-ft | calculation - OK (+ difference) |
| check: $M_r > 4/3M_u$ | - | 11073254 | lb-ft | calculation - OK (+ difference) |
| maximum factored load of superstructure from an interior girder | $P_u$ | 141730.6 | kip | final superstructure calculations |
| maximum factored shear | $V_u$ | 354326.4 | kips | calculation |
| effective shear depth | $d_v$ | 40.11765 | in | calculation |
| nominal shear | $V_n$ | 60894.31 | lbs | ACI |
| design capacity | $V_r$ | 54804.88 | kips | calculation |
| check: $V_r > V_u$ | - | -299522 | kips | calculation - BAD (- difference) |

Conclusions: completed as routine

$M_u < M_r$ ∴ flexural reinforcement is adequate; $V_u > V_r$ ∴ shear reinforcement is inadequate

3. Bearing Design

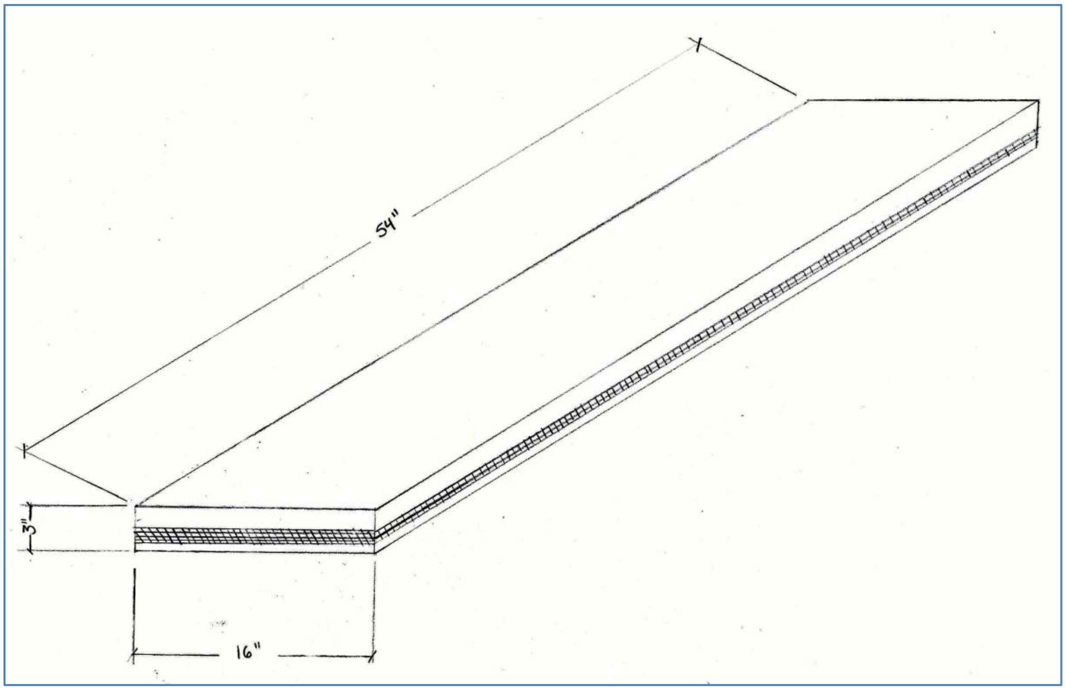



Figure 53 - Fabric Reinforced Elastomeric Bearing Pad (1/8" = 1")

Table 57 - Dimensions, Load and Bearing Capacity of Elastomeric Bearing Pad

| Dimensions | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| length of girders | $L_g$ | 98 | ft | assumption |
| girder bottom flange width | $w_{bf}$ | 54 | in | assumption |
| number of girders | $N_{girder}$ | 6 | - | assumption |
| temperature movement & concrete shortening coefficient | $\alpha_T$ | 0.000008 | - | concrete standard |
| moderate temperature zone rise and fall | T | 50 | o | assumption |
| maximum pressure the bearing pad can withstand | $P_{max\ load}$ | 0.8 | ksi | standard |
| modulus of rigidity of concrete | $G_c$ | 3000.00 | psi | standard |
| DL each girder | $DL_{girder}$ | 140 | lbs | calculation |
| LL each girder | $LL_{girder}$ | 0.986 | lbs | calculation |
| temperature movement | $\Delta_t$ | 0.7056 | in | calculation |
| minimum thickness of the pad | $t_{min}$ | 1.4112 | in | calculation |
| assumption of pad thickness | $t_{as.}$ | 3.4 | in | rounded |

| | | | | |
|---|---|---|---|---|
| length of trial pad | $L_p$ | 3.263565 | in | calculation |
| assumption of pad length | $L_{as.}$ | 16 | in | rounded |
| check max thickness of pad | $1/3L_{as.}>t_{as.}$ | 1.933333 | - | OK (+ difference) |
| check compressive strength is less than pad bearing capacity: $(LL+DL)/(w_{bf}L_{as.}) < 800$ psi | $P_{max\ laod}$ | -799.837 | psi | OK (- difference) |
| check compressive strength is less than pad bearing capacity of dead load only: $(DL)/(w_{bf}L_{as.}) < 200$ psi | - | -199.838 | psi | OK (- difference) |
| shape factor | ξ | 12.34286 | - | calculation |
| final thickness calculation | - | 2.875429 | | |
| final thickness assumption | $t_f$ | 3.0 | in | calculation |
| maximum shear | $V_{max}$ | 280.5 | kip | calculation |
| design shear force | $V_D$ | 614.6729 | kip | calculation |
| check: $V_D > V_{max}$ | - | 334.1729 | kip | OK (+ difference) |
| elastomeric pad size: | L | 16 | in | assumption |
| | w | 54 | in | assumption |
| | t | 3.0 | in | assumption |

Conclusions:

L = 16 in
w = 54 in
t = 3.0 inches

4. Final Substructure Design

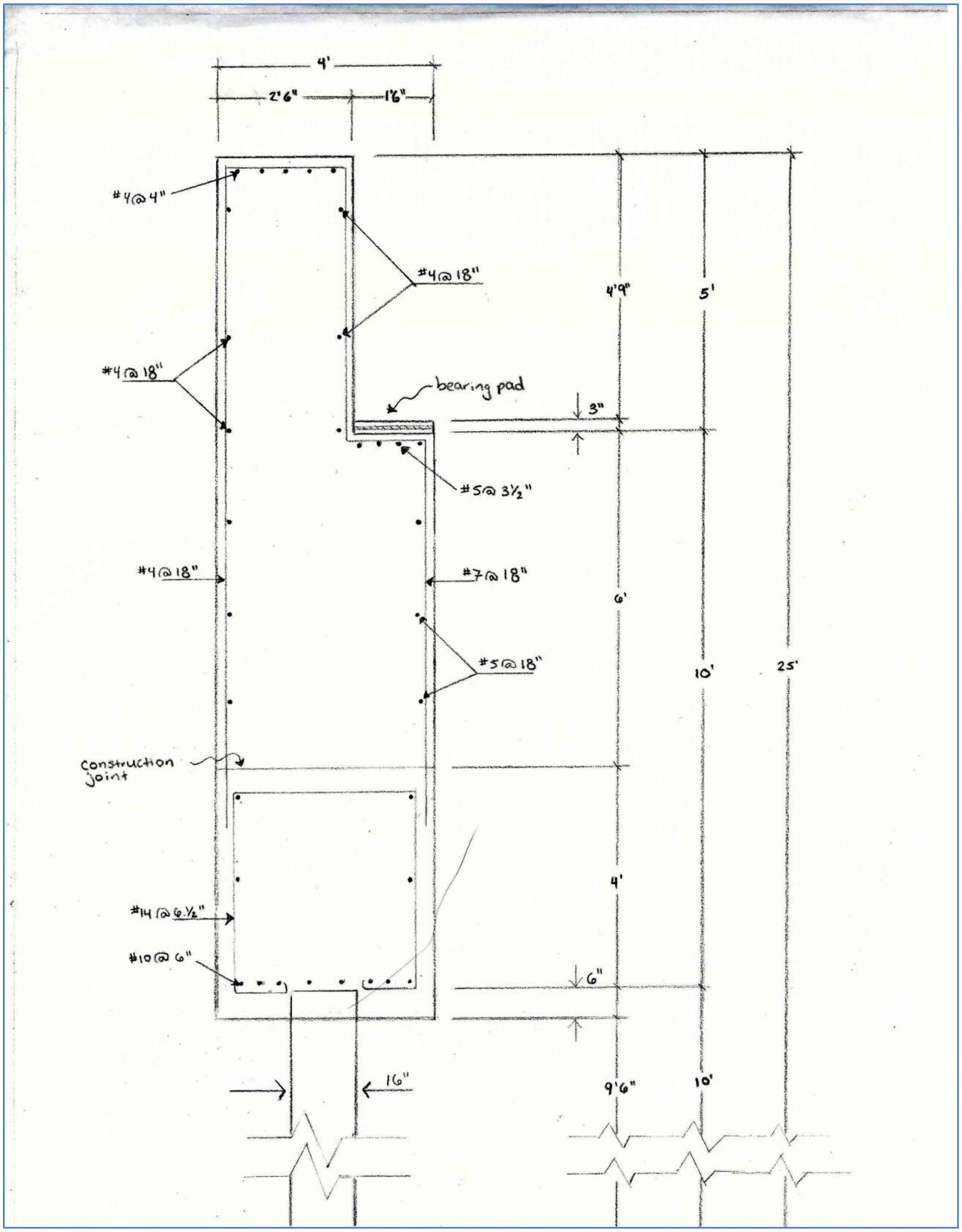



Figure 54 - N-S Elevation of Substructure Final Design (1/4" = 1')

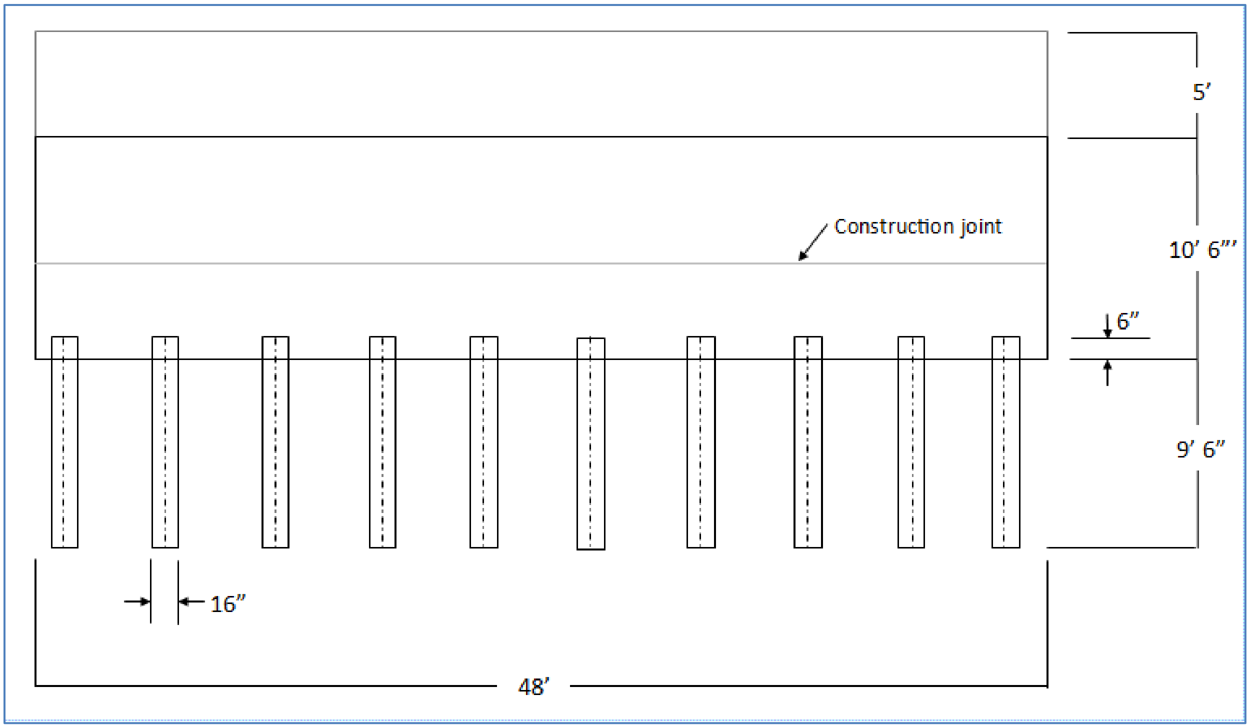



Figure 55 - E-W Elevation View of Substructure Final Design

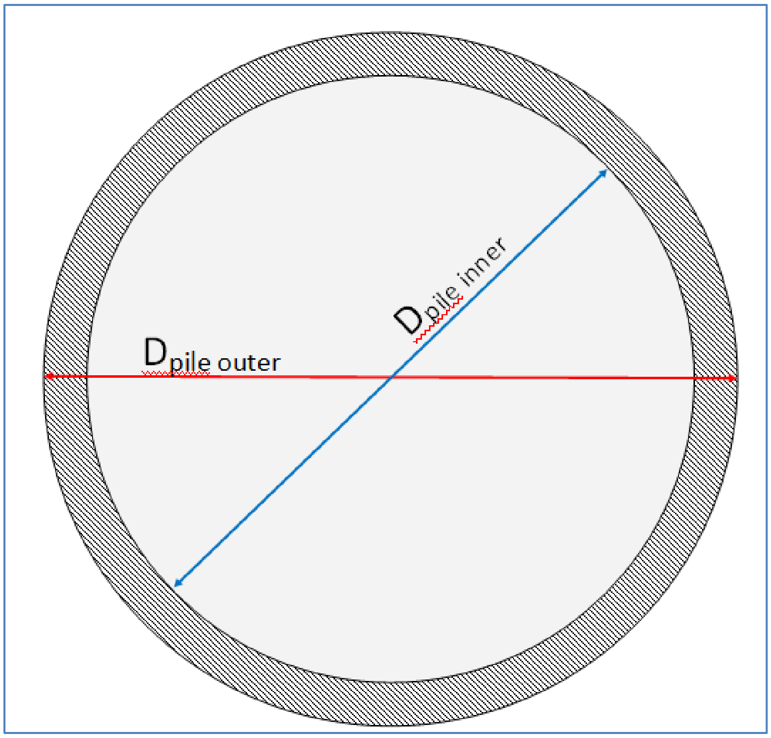



Figure 56 - Cross Section of Pile

a. Pile group assumptions and calculations

Table 58 - Pile Group Assumptions and Calculations

| Variable | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| pile cap width | w | 4 | ft | assumption |
| pile cap height | h | 4.5 | ft | assumption |
| pile cap length | l | 48 | ft | assumption |
| volume of pile cap | V | 864 | $ft^3$ | calculation |
| weight of pile cap | W | 129600 | lb | calculation |
| group efficiency factor | η | 0.8374 | - | calculation |
| number of piles in group | N | 10 | - | calculation |
| number of rows of piles | m | 1 | - | assumption |
| number of piles per row | n | 10 | - | assumption |
| tan-1(B/s) | θ | 16.26 | o | calculation |
| center to center pile spacing | si | 4.8 | ft | |
| pile length | D | 10 | ft | assumption |
| height from top of footing to ground | zw | 4 | ft | assumption |
| pile diameter | B | 1.33333 | ft | assumption |
| steel pile area (PP12.75x0.375) | AS | 0.32722 | $ft^2$ | assumption |
| diameter of pile | dpile | 16 | in | assumption |
| surface area of pile | SA | 41.9463 | ft | calculation |
| Specific Gravity soil | $G_S$ | 2.7 | - | assumption |
| unit weight of concrete | $\gamma_c$ | 150 | pcf | assumption |
| Unit weight water | $\gamma_w$ | 62.4 | pcf | standard |
| Unit weight dry soil | $\gamma_d$ | 120 | pcf | assumption |
| unit weight | γ | 133 | pcf | assumption |
| unit weight prime | γ' | 70.6 | pcf | calculation |
| void ratio | e | 1.404 | - | calculation |
| pore water pressure | u | 624 | psf | calculation |
| vertical effective stress at depth d | $\sigma'_{zD}$ | 706 | psf | calculation |
| effective friction method | ϕ' | 30 | o | assumption |
| soil foundation interface friction angle | $\phi_f$ | 21 | o | assumption |
| | $\phi_f/\phi'$ | 0.7 | - | calculation |
| Poisson's Ratio | v | 0.3 | - | assumption |

| | | | | |
|---|---|---|---|---|
| Soil modulus of eleasticity | $E_S$ | 7300 | psi | assumption |
| | tan(ϕ') | 0.57735 | - | calculation |
| | sin(ϕ') | 0.5 | - | calculation |
| | $\tan^2(45+\phi'/2)$ | 3 | - | calculation |
| Rigidity Index | $I_R$ | 6.888195 | psi | calculation |
| coefficient of lateral earth pressure at rest | $K_o$ | 0.5 | - | calculation |
| coefficient of lateral earth pressure | K | 0.5 | - | assumption |
| bearing capacity factor $N^*_y$ | $N^*_y$ | 5.247541 | - | Terzaghi Method |
| bearing capacity factor $N^*_q$ | $N^*_q$ | 16.14835 | - | Terzaghi Method |
| bearing capacity factor $N^*_\sigma$ | $N^*_\sigma$ | 24.22252 | - | Terzaghi Method |
| effective cohesion | $c'=c_T=S_u$ | 250 | psf | assumption |
| unit side friction resistance | $f_s$ | 247.1 | psf | calculation |
| pile shape factor | $K_{shape}$ | 1 | - | assumption |
| net unit toe bearing resistance | $q_t$ | 11894.70 | psf | calculation |
| toe load resistance | $P_t$ | 16630.32 | lbs | calculation |
| side friction | $P_s$ | 10364.92 | lbs | calculation |
| weight of pile cap | $w_c$ | 129600.00 | lbs | calculation |
| number of piles | η | 10.00 | - | calculation |
| weight of pile cap on each pile | $w_{pc/N}$ | 12960.00 | lbs | calculation |
| factor of safety | F | 3.50 | - | Coduto book |
| allowable load capacity | $P_a$ | 4.01 | kips | calculation |
| end bearing capacity of one pile | $q_t$ | 11894.70 | psf | calculation |
| bearing capacity due to friction | $f_s$ | 247.10 | psf | calculation |
| weight of pile cap | $DL_{PC}$ | 675 | psf | calculation |
| allowable bearing capacity per pile | $Q_{allowable}$ | 11.47 | ksf | calculation |
| allowable load capacity | $P_a$ | 4010.071 | lbs | calculation |
| allowable group load capacity | $P_{ag}$ | 33.58 | kips | calculation |
| allowable bearing capacity per pile | $Q_{allowable}$ | 11.47 | ksf | calculation |
| ultimate group bearing capacity | $Q_{ult}$ | 96.02 | psf | calculation |

| **Load** | **Description** | **Value** | **Unit** |
|---|---|---|---|
| DL of Substructure | Load per square foot of the substructure: abutment wall, backwall, and pile cap | 2.33 | ksf |

| | | | |
|---|---|---|---|
| DL of Superstructure | Load per square foot of the girders, deck, wearing surface, and parapets | 0.54 | ksf |
| LL of Superstructure | Load due to vehicular traffic | 0.00 | ksf |
| Remaining Group Bearing Capacity | Bearing capacity that the superstructure can exert on the substructure before failure | 30.72 | ksf |

Conclusions:

Remaining group bearing capacity of the substructure pile design = +30.72 ksf
∴ design is adequate

b. Pile cap reinforcement assumptions and calculations

**Table 59 - Pile Cap Reinforcement Dimensions, Load and Bearing Capacity of Final Design**

| Variable | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| pile spacing - interior spacing | $s_i$ | 4.8 | ft | assumption |
| length of girders | $L_g$ | 98 | ft | assumption |
| reinforcing bars | - | 8 #11 | - | assumption |
| area of rebar | $A_s$ | 10.16 | $in^2$ | assumption |
| diameter of bar | $d_{bar}$ | 1.27 | in | assumption |
| pile cap height (depth) | $d_s$ | 54 | in | assumption |
| bottom cover | $d_s$-d' | 3 | in | assumption |
| width of pile cap for design | $b_d$ | 12 | in | assumption |
| stirrups | - | 1 # 14 bar | - | assumption |
| area of stirrups - | $A_v$ | 4.5 | $in^2$ | assumption |
| spacing of stirrups | $s_s$ | 6.75 | in | assumption |
| strength resistance factor | $\phi$ | 0.9 | - | ACI |
| strength of steel | fy | 60 | ksi | ACI |
| strength of concrete | f'c | 4 | ksi | ACI |
| shear multiplication factor | $\lambda$ | 2 | - | ACI |
| dead load of superstructure | DLsup | 1683 | kip | final superstructure calculations |
| maximum factored load of superstructure from an interior girder | Pu | 169.58 | kip | final superstructure calculations |
| DL of substructure | DLsub | 392.4 | kip | calculation |
| positive moment for flexural design | Mu | 1248 | kip-ft | calculation |
| effective depth | de | 44.37 | in | calculation |
| neutral axis | a | 14.94 | in | calculation |
| nominal moment | Mn | 1874 | kip-ft | calculation |
| design capacity | Mr | 1687 | kip-ft | calculation |
| check: Mr > Mu | - | 439 | kip-ft | calculation - OK (+ difference) |
| check: -Mr > -4/3Mu | - | 23.21 | kip-ft | calculation - OK (+ difference) |
| maximum factored shear | Vu | 1300 | kips | calculation |
| effective shear depth | dv | 37 | in | calculation |

| | | | | |
|---|---|---|---|---|
| nominal shear | Vc | 148 | kips | calculation |
| design capacity | Vr | 133 | kips | calculation |
| check if: Vu ≤ 0.5Vr for no needed stirrups | - | 1233 | kips | calculation - BAD (+ difference) |
| check if: Vu ≤ 6√f'cbdv for good effective depth | - | -19277 | kips | calculation- Good ( - difference) |
| capacity of steel | Vs | 1476 | kip | calculation |
| design capacity of steel | Vn | 1328 | kip | calculation |
| check if: ϕVs ≥ Vu - ϕVc | - | 161.33 | kip | calculation - OK ( + difference) |

Conclusions:

$M_u < M_r$ ∴ flexural reinforcement is adequate

$V_u < V_r$ ∴ shear reinforcement is adequate

c. Backwall reinforcement assumptions and calculations

Table 60 - Backwall Reinforcement Dimensions, Load and Bearing Capacity of Final Design

| Dimension Assumption | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| girder spacing | lg | 5 | ft | assumption |
| length of girders | $L_g$ | 98 | ft | assumption |
| reinforcing bars | - | 5 # 4 bars | - | assumption |
| area of rebar | $A_s$ | 1 | $in^2$ | assumption |
| diameter of bar | $d_{bar}$ | 0.625 | in | assumption |
| bottom cover | $d_s$-d' | 4 | in | assumption |
| backwall width | $w_b$ | 2.5 | ft | assumption |
| area of stirrups | $A_v$ | 0.62 | $in^2$ | assumption |
| spacing of stirrups | $s_s$ | 10 | in | assumption |
| abutment length | L | 48 | ft | assumption |
| number of lanes | $N_{lanes}$ | 2 | - | assumption |
| roadway width | $w_{road}$ | 32 | feet | assumption |
| depth of roadway asphalt | $d_{asph}$ | 0.375 | feet | assumption |
| abutment wall height | H | 11 | feet | assumption |
| stirrups | - | 1 # 4 bar | - | assumption |
| area of stirrups | $A_v$ | 0.4 | $in^2$ | assumption |
| spacing of stirrups | $s_s$ | 18 | in | assumption |
| strength resistance factor | ϕ | 0.9 | - | ACI |
| strength of steel | fy | 60000 | psi | ACI |
| strength of concrete | f'c | 4000 | psi | ACI |
| shear multiplication factor | λ | 2 | - | ACI |
| unit weight of asphalt | γasphalt | 140 | pcf | standard |
| dead load of superstructure | DLsup | 1683 | kip | superstructure calculations |
| back wall weight | ωbackwall | 750 | lb/ft | calculation |
| approach slab weight | ωslab | 1680 | lb/ft | calculation |
| future wearing surface weight | FWS | 840 | lb/ft | calculation |
| approach slab lane load | ωlane | 640 | lb/ft | AASHTO |
| factored load due to the self weight of the pile cap and diaphragm | ωu | 4.22 | kip | calculation |
| maximum factored load of superstructure from an interior | PStr -II | 344.72 | kip | superstructure calculations |

| | | | | |
|---|---|---|---|---|
| girder | | | | |
| positive moment for flexural design | Mu | 355 | kips-ft | calculation |
| effective depth | ds | 127.69 | in | calculation |
| neutral axis | a | 1.47 | in | calculation |
| nominal moment | Mn | 635 | kip-ft | calculation |
| design capacity | Mr | 571 | kip-ft | calculation |
| check: Mr > Mu | - | 216 | kip-ft | calculation - OK (+ difference) |
| check: -Mr > -4/3Mu | - | 98 | kip-ft | calculation - OK (+ difference) |
| maximum factored shear | Vu | 355 | kips | calculation |
| effective shear depth | dv | 126.95 | in | calculation |
| nominal shear | Vc | 317 | kips | ACI |
| design capacity | Vr | 237 | kips | calculation |
| check if: Vu ≤ 0.5Vr for no needed stirrups | - | 236.53 | kips | calculation - BAD (+difference) |
| check if: Vu ≤ 6√f'cbdv for good effective depth | - | -17925.8 | kips | calculation- Good ( - difference) |
| capacity of steel | Vs | 169.27 | kip | calculation |
| design capacity of steel | ϕVs | 152.34 | kip | calculation |
| check if: ϕVs ≥ Vu - ϕVc | - | 25.39 | kip | calculation - OK ( + difference) |

Conclusions:

$M_u < M_r$ $\therefore$ flexural reinforcement is adequate

$V_u < V_r$ $\therefore$ shear reinforcement is adequate

d. Bridge seat reinforcement assumptions and calculations

Table 61 - Bridge Seat Reinforcement Dimensions, Load and Bearing Capacity of Final Design

| Dimension Assumption | Symbol | Value | Unit | Source |
|---|---|---|---|---|
| pile spacing - interior spacing | $s_i$ | 4.8 | ft | assumption |
| reinforcing bars | - | 4 # 5 bars | - | assumption |
| area of rebar | $A_s$ | 1.24 | $in^2$ | assumption |
| diameter of bar | $d_{bar}$ | 1.27 | in | assumption |
| bottom cover | $d_s$-d' | 3 | in | assumption |
| stirrups | - | 1 # 7 bars | - | assumption |
| area of stirrups - | $A_v$ | 1.2 | $in^2$ | assumption |
| spacing of stirrups | $s_s$ | 18 | in | assumption |
| strength resistance factor | ϕ | 0.9 | - | ACI |
| strength of steel | fy | 60000 | psi | ACI |
| strength of concrete | f'c | 4000 | psi | ACI |
| shear multiplication factor | λ | 2 | - | ACI |
| factored load due to the self weight of the pile cap and diaphragm | ωu | 1.58 | kips-ft | superstructure calculation |
| maximum factored load of superstructure from an interior girder | Pu | 220.5 | kip | superstructure calculation |
| positive moment for flexural design | Mu | 224.44 | kips-ft | calculation |
| effective depth | ds | 68.37 | in | calculation |
| neutral axis | a | 1.82 | in | calculation |
| nominal moment | Mn | 418.21 | kip-ft | calculation |
| design capacity | Mr | 376.39 | kip-ft | calculation |
| check: Mr > Mu | - | 151.95 | kip-ft | calculation - OK (+ difference) |
| check: -Mr > -4/3Mu | - | 77.13 | kip-ft | calculation - OK (+ difference) |
| maximum factored shear | Vu | 224.29 | kips | calculation |
| effective shear depth | dv | 67.45 | in | calculation |
| nominal shear | Vc | 10.38 | kips | calculation |
| design capacity | Vr | 7.78 | kips | calculation |
| check if: Vu ≤ 0.5Vr for no needed stirrups | - | 220.39 | kips | calculation - BAD ( + difference) |

| | | | | |
|---|---|---|---|---|
| check if: Vu ≤ 6√f'cbdv for good effective depth | - | -82.87 | kips | calculation- Good ( - difference) |
| capacity of steel | Vs | 269.81 | kip | calculation |
| design capacity of steel | ϕVs | 242.83 | kip | calculation |
| check if: ϕVs ≥ Vu - ϕVc | - | 18.55 | kip | calculation - OK ( + difference) |

Conclusions:

$M_u < M_r$ $\therefore$ flexural reinforcement is adequate

$V_u < V_r$ $\therefore$ shear reinforcement is adequate

e. Abutment wall reinforcement assumptions and calculations

**Table 62 - Abutment Wall Reinforcement Assumptions**

| Abutment Element | Flexural R.F. | | Shear R.F. | | Total |
|---|---|---|---|---|---|
| | Bars | Spacing | Bars | Spacing | |
| abutment wall - under bridge seat | 4 #5 bars | 18 in | #7 | 18 in | 5.2 in |
| abutment wall - backwall | 7 #4 bars | 18in | #4 | 18 in | 8 in |

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} = 0.0018bh$$

$$A_{s_{bridge\ seat}} = 5.2\ in\ > 0.0018bh = 0.072\ \therefore acceptable\ reinforcement\ design$$

$$A_{s_{backwall}} = 8.0\ in\ > 0.0018bh = \ 0.108\ \therefore acceptable\ reinforcement\ design$$

Conclusion: acceptable reinforcement design in the abutment wall to ACI specifications