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North Lake Ave Bridge Design

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Major Qualifying Project

Submitted to the Faculty

of

Worcester Polytechnic Institute

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Abstract

North Lake Ave is a road located in Worcester, MA that has been suffering from erosion for many years. Conceptual plans have been made to create a linear park and convert this road into a one-way. Options have been considered to address the erosion and collapse of the road. The design for a retaining wall and bridge will be compared based on safety, economic, constructability, environmental, societal and sustainability criteria. The design for the bridge includes the superstructure, substructure, and foundation, in compliance with *AASHTO LRFD Bridge Design Specifications*.

Capstone Design

To ensure successful completion of this project, the ABET requirements for a Capstone Design experience must be entirely fulfilled. Since the project focuses mainly on structural, geotechnical, and construction management aspects of civil engineering, many of the capstone design requirements are accomplished. For the structural and geotechnical designs, it was necessary to use real world data such as traffic records, boring logs and soil information for this specific site. This data combined with the social impact of the design and construction emphasized the importance of a universally accepted design pertaining to feasibility, performance, safety, and aesthetics. Following the AASHTO LRFD Bridge Design Specifications ensured the safety of the proposed design. The construction management component incorporated economical, constructability, and sustainability aspects into the design, which was essential in choosing the most feasible solution. The combination of these elements completed the capstone design requirements.

Constructability

The importance of constructability spans the entire project as it relates to the project's design and overall feasibility of construction. Different materials and orientations of bridge and retaining wall elements were selected based on the needs of the problem and were split into distinct activities. The materials and instructions for each activity were clearly laid out, and areas meant for storage were marked to allow distinct construction element retrieval. Regarding existing conditions, awareness of the unique steep hillside landscape warrants caution and understanding on the part of laborers and engineers in the field.

Social

The surrounding residential area, which closely borders the project site, and the traffic that is prominent on the road itself, warrant distinct social considerations for approaching the project. UMass Memorial Medical Center is also located in close proximity to the project site, which resulted in limiting vibrations during the foundation design. Reviewing articles on what the community and especially the residents, whose properties border the worksite, would like to see done is a critically important step in designing the project. Risking stonewalling from the residents on the project's progress could be a serious detriment to the restoration's cost and time of completion.

In every project involving residents around or possibly within the project, the social impacts must be taken into consideration. Reviewing the history and current state of the community around the project site put perspective and constraints on the entire design. The residents or community have specific goals that they want to see get done whenever something is being constructed. For North Lake Avenue, a residential neighborhood is located on the west side facing Lake Quinsigamond and a local hospital located a few miles away. This affected which deep foundation was chosen for the design. For instance, vibrations from the driving process of a pile foundation could disrupt medical equipment and cause discomfort for both patients within the hospital and residents within their own homes. The limited space between the residents and Lake Quinsigamond limited construction access for heavy vehicles and disrupts traffic flow. Traffic heading north on North Lake Avenue looking to travel on I-290 East, and traffic heading south leaving I-290 East will be less efficient and more time consuming if there are detours for construction purposes.

Economic

Economic constraints were evaluated for project development in order to reduce construction costs. The economics of the project must come into play throughout the report and re-evaluated to preserve efficiency. The cost, and roadway re-construction of a retaining wall was compared to that of a simple span bridge in order to determine the most effective solution. *International Project Estimating Limited,* FHWA Cost Data, and *RS Means 2016* were used to determine prices for the various elements of the project. The scope of the economic constraints includes materials, structural elements (concrete, steel, etc.), and construction management variables. These variables include cost of operation (engineers, construction workers, etc.), the construction plans, and project construction schedules.

Health and Safety

In the design of any construction involving human labor or occupancy, their health and safety are crucial factors to take into consideration in the project. In order to ensure public safety and integrity of both the retaining wall and bridge, every structural element was proportioned in accordance with the governing codes and standards. Traffic load and member size restrictions were calculated based on the *AASHTO LRFD Manual*. The factors of safety for retaining walls and bridge foundations were determined using *Foundation Design: Principles and Practices. LRFD Specifications* were used in order to determine the load and resistance factors for a simple span bridge.

Environmental

Construction for both retaining walls and ridges has environmental impacts that were considered during the project management process. The excavation of the site will produce

possible hazards on Quinsigamond Lake disrupting traffic, wildlife, and the lake itself as a body of water. The effect that the construction of these structures will have must be taken into account throughout the project. Keeping these issues in mind will help reduce the impacts on the surrounding environment around the site. Maximizing the use of on-site soil has beneficial environmental impacts since trucks will not be used.

Environmentally, with the construction and repair of the new road with a bridge or retaining wall, previously occupied shoreline, which was available for recreation and docking, will no longer be available in the same capacity. This may affect wildlife occupation along the shoreline of the affected area.

Erosion of the shoreline will be another thing to consider as an object of impact. Depending on the backfilling techniques of the site work as well as the new surcharge load of built material on the soil up to the shoreline may affect the shape and soil profile that exists along the shore.

Sustainability

The concept of sustainable civil infrastructure informs the goal of this project. The problem for North Lake Ave was that the road was not built as sustainable as it could have been. This solution provides a system with a greater service life than what the road previously had. The optimal materials were considered in order to design a sustainable structure.

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1.0 Introduction

The following Major Qualifying Project (MQP) report gives a comprehensive comparison of a retaining wall and bridge to improve the condition of North Lake Ave. The background includes information necessary to understand the components, aspects and governing factors impacting the design. A methodology is provided outlining and describing the steps taken during each of the designs.

This project focuses on the efforts made to design an economical and sustainable solution to rebuilding a 200-foot section of road along Lake Quinsigamond that collapsed due to heavy traffic coupled with erosion. Two basic solutions were explored. One being the replacement of the road with a bridge, and the other being a using a retaining wall to restore the embankment the road is on. Both solutions prevent the further damage to the road from erosion and will accommodate the higher traffic volumes.

Both designs follow specifications according to the *American Association of State Highway and Transportation Officials* (AASHTO) *LRFD Bridge Design Specifications 2012 Manual* (28), MassDOT recommendations (20), and the *Federal Highway Administration Steel Bridge Design Handbook* (17). The design of the bridge gave an opportunity to explore a steel and slab option, a composite steel deck option, and a composite prestressed concrete beam and deck option. The use of *RISA*, and Excel aided in hand calculations and helped to explore these options. The retaining wall solution also involved consideration of options such as a Mechanically Stabilized Earth (MSE) wall, or a cantilever wall. All of the solution options listed were evaluated based on cost, schedule, time to complete, and societal impacts for the surrounding neighborhoods and traffic. A design was chosen to best satisfy these criteria.

2.0 Background

North Lake Ave is a road in critical condition overlooking Quinsigamond Lake in Worcester, MA. Since 2009 this road has been eroding at an increased rate due to heavy rainfall, poor soil conditions, and an increase in traffic. This severe erosion has caused a five-hundred-foot section of the road to be diminished to one-lane; a loss of nearly 12 feet (Shulkin, J. 2009). A temporary solution was proposed in December 2009 to simply place jersey barriers, which paired with the smaller road width are the reason traffic is single lane only. The jersey barriers were placed along the East side of the road, closest to the Lake, and install traffic lights on both the northbound and southbound sides shown in Figure 3. Seven years later North Lake Ave is still restricted to one-lane traffic causing major traffic delays, environmental concerns from idling car exhaust, and noise pollution during peak hours.
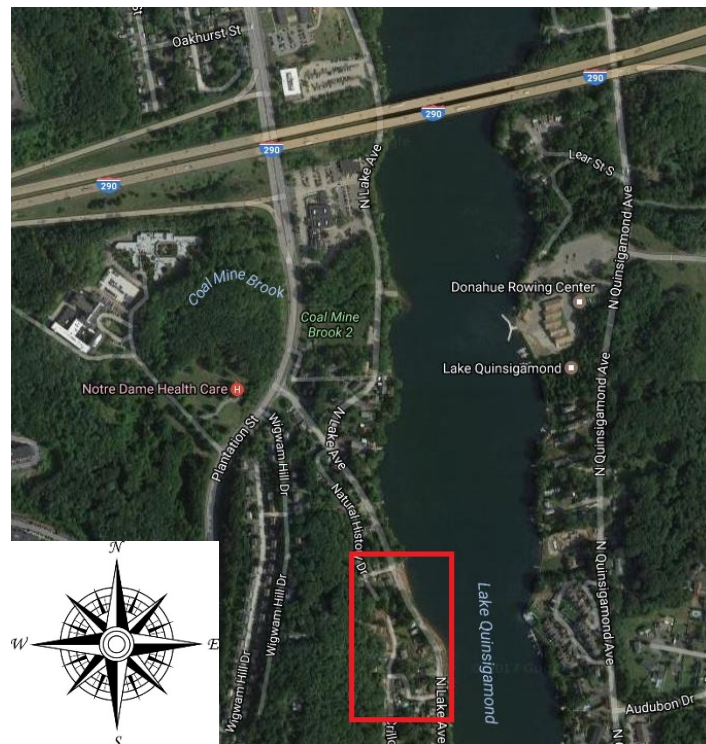
Figure 1 - Aerial View of Project Site

Figure 2 - Aerial View Zoomed

Figure 3: Image of North Lake Avenue

The city of Worcester has moved forward with a $75,000 traffic study analyzing the daily

traffic on North Lake Ave (Shulkin, J. 2009). This traffic study is the first step towards replacing

the temporary traffic lights and providing a permanent solution to repair this road. The proposed

plan is to create a $3.3 million linear park alongside Lake Quinsigamond and turn to North Lake

Ave into a one-way street (southerly). Secondary plans for a promenade along the two-mile

stretch of the road allowing access for bikers, joggers, and other pedestrians have been

conceptualized along with the park (Kotsopoulos, N. 2009). The residents of North Lake Ave

have expressed significant opposition towards this project suggesting a one-way street carries

many negative impacts on the neighborhood such as increased vehicle speeds, traffic and noise. Creating a one-way southerly travel also inhibits access to I-290 East from North Lake Ave and redirects ambulance routes from UMASS Memorial Hospital.

The goal of this MQP is to conceptualize, design, and recommend an alternative bridge and retaining wall design. This will be completed by designing a retaining wall structure necessary to support the volume of soil and resist the traffic loads considered to be imposed on North Lake Ave. The soil parameters were referenced from the boring logs of the geotechnical report completed by LGCI in 2010 (31). The retaining wall design will then be compared to an alternative bridge which will include the foundation, substructure, and superstructure design. The bridge design was accomplished in accordance the *AASHTO LRFD Bridge Design Specifications* (28). Various types of bridge designs were prepared and compared, and the selected design was compared with the retaining wall. Each design was compared focusing on safety, economic, constructability, environmental, societal and sustainability criteria.

The results obtained are shown in various sketches, renderings and comparative tables. Images taken from Google Maps and our own on-site photos compare the condition of North Lake Ave in the years 2007, 2011, and 2016 (Figure 4). The photos provided below show the constant degradation this road continues to suffer. The condition of the road will continue to worsen unless new construction is introduced.

Figure 4: North Lake Ave during 2007, 2011, and 2016 respectively

2.1 Cantilever Retaining Wall

Earth retaining structures are commonly categorized into two types, externally stabilized systems and internally stabilized systems. "Externally stabilized systems are those that resist the applied earth loads by their weight and stiffness" (Coduto, D. P. 2001). This includes gravity walls such as reinforced concrete cantilever walls and sheet piles. Figure 5 shows the terminology of a typical cantilever retaining wall cross section. For heights of 10 to 20 ft, cantilever walls are more economical since they consist of thin stems resulting in a smaller cross section and less material and construction costs.
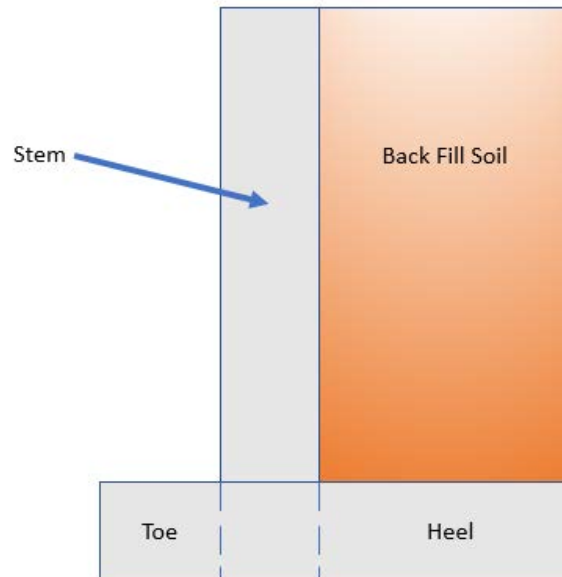
Figure 5: Cantilever Retaining Wall

2.1.1 Retaining Wall design Criteria

Retaining walls must abide by a criteria checklist before the process of designing the retaining wall. Four primary concerns must be met in order to meet the design criteria. Acceptable factors of safety for overturning and sliding must be met. The allowable soil bearing pressures should not be exceeded, and the structural integrity requirements should be within code allowable limits to be able to resist vertical and lateral loadings (Nielsen, H. 2013).

Before starting the retaining wall design, certain factors must be taken into consideration for this design criteria checklist. A soil investigation report with soil properties and parameters must be established. Is there a property line condition or water table that must be considered, and what building codes apply? Is there lateral restraint on the top of the wall? Is there a slab in front of the wall to restrain sliding or prevent erosion of the soil? Should the stem be reinforced concrete, masonry, or a combination of both? What is the slope of the backfill and how will the

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backfill be drained? Will there be any axial loading or seismic design required (Nielsen, H. 2013)?

Along with this design criteria checklist, the following values shown in the table below must be established/calculated to begin design process:

Table 1 - Design Parameters for a Cantilever Retaining Wall

Retaining Wall	Soil	Forces	Loads	Concrete Properties
Retained Heights	Coefficient of Friction	Allowable Soil Pressure	Axial Loading on Stem	Compressive strength of Concrete f'c
Embedment Depth of Footing	Backfill Slope	Passive Earth Pressure	Surcharge Load	Yield Strength of Steel Reinforcement f'y
--------------	Soil Unit Weight	Active Earth Pressure	Wind Loads	Unit Side Friction Resistance fs
--------------	--------------	--------------	Seismic Design (if applicable)	Mobilized Side Friction Resistance f'm
--------------	--------------	--------------	--------------	Unit Weight

2.1.2 Lateral Earth Pressure

Earth retaining structures are subjected to many vertical and horizontal loads. Lateral Earth Pressure is the pressure that soil, due to its own weight, exerts on a retaining earth structure. The Coefficient of lateral earth pressure (K) has an influence on the lateral forces

acting within the backfill soil being retained. This includes lateral earth pressure, surcharge loads, and pore water pressure. As defined, "the coefficient of lateral earth pressure is the ratio of horizontal to vertical effective stresses at any point within the soil" (Coduto, D.P. 2001). Loads applied to a confined section of soil create pressure pushing against the affected area. The loading that largely governs the design of retaining structures is the lateral force exerted from the backfilling soil (Coduto, D.P. 2001).

The three types of lateral earth pressure a retaining wall can be subjected to are at-rest, active, and passive. For the at-rest condition, it is assumed that the retaining wall resists flexural movements (rigid) and there is no lateral translation or rotation (unyielding). Also, making the assumption that there are no lateral strains within the ground will make the lateral stresses as if they were in a natural state (Coduto, D. P. 2001).

The active condition allows for very small movements away from the backfilling soil, which influences the magnitude of the lateral earth pressure. This movement may be translational or rotational which reduces part of the horizontal stress. If the wall is permitted to move a great enough distance, the backfill soil will fail in shear. The wall fails in shear when it hits the failure plane, which is governed by the effective frictional angle and cohesion of the soil.

The passive condition is the opposite of the active condition. Instead of the wall moving away from the backfill, the passive condition refers to the retaining wall moving towards the backfill. The passive condition involves more movement than the active condition. In the passive condition "the vertical stress remains constant whereas the horizontal stress changes in response to the induced horizontal strains. Engineers often use the passive pressure that develops along the toe of a retaining wall footing to help resist sliding" (Coduto, D. P. 2001).

2.1.3 External Stability

Engineers design structures in order provide safety for any occupants, and the surrounding environment. The term factor of safety defines whether the structure meets a certain criterion in order to be deemed "safe" against certain failure modes. This criterion can be defined as the ratio of resisting to driving forces or moments, and this ratio must be greater than a recommended value based on the analysis method, codes, or experience used in the design process using allowable strength design (ASD). Three significant failure modes for the external stability of a retaining wall are: sliding, overturning, and bearing capacity, which are illustrated below (Figure 6).
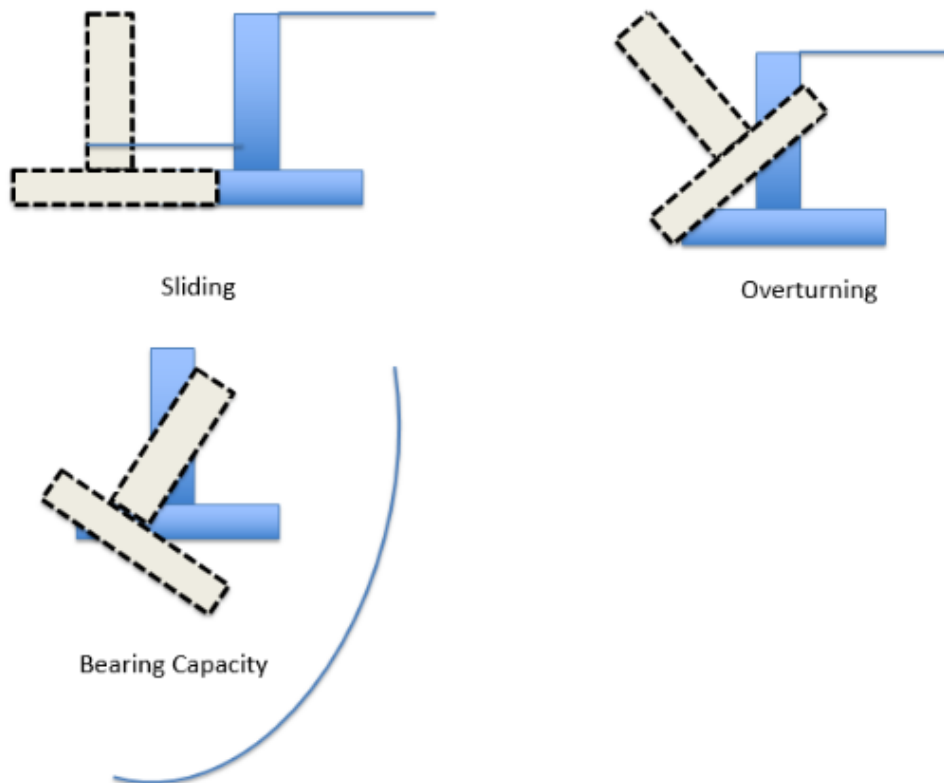
Figure 6: The three failure modes for external stability

In terms of external stability, a cantilever retaining wall must not slide. A limit equation is used when evaluating the sliding stability. The factor of safety is taken into consideration for

this limit equation, being the sum of the resistant forces divided by the driving forces. A "true" factor of safety of 2 to 3 is the most suitable result for a cantilever retaining wall (Coduto, D. P. 2001).

Along with sliding stability, overturning stability requirements must be taken into consideration. The factor of safety equation is similar in structure to the one for the horizontal sliding, but instead of the resisting and driving forces, it involves resisting and overturning moments. The resisting moments must be summed together in one direction divided by the overturning moments in the opposite direction (clockwise and counterclockwise). Since moment values depend on the selected axis of rotation, the factor of safety is calculated depending on the location of the chosen point about which the moments are taken (typically the toe of the footing). Typical overturning moments are caused by the horizontal component to the lateral earth pressure, hydrostatic forces acting behind the "wall-soil unit," surcharge loads, and seismic forces from the backfill (Coduto, D. P. 2001). Typical resisting moments are provided by the vertical component to lateral earth pressure, the weight of the "wall-soil unit," surcharge loads, and hydrostatic pressure acting on the front of the footing. Overturning analysis neglects the normal force between the footing and the ground since this force has no moment arm (acts through the center of the axis of overturning). This analysis also neglects the friction force for the same reason. The minimum factor of safety required for the overturning is 1.5 to 2.0 (Coduto, D. P. 2001).

The third failure mode is bearing capacity, which is a geotechnical strength requirement. The vertical load of the structure induce compressive and shear stresses in the soil creating one of three failure modes; general shear failure, local shear failure, or punching shear failure. General shear failure is the most common of these three types and occurs in incompressible,

normally consolidated soils. The ultimate bearing capacity of the given soil is solved using

Terzaghi's or Vesic's method

2.1.4 Internal Stability

Once the external stability requirements are met, the retaining wall's internal stability

(structural integrity) must be analyzed. The structural design must resist any applied loads with

sufficient factors of safety. The analysis of the internal stability begins with the stem, and then

goes into the footing of the retaining wall. The footing is almost always made from reinforced

concrete (Coduto, D. P. 2001). Tall retaining wall stems are made of reinforced concrete, while

shorter ones can use reinforced masonry. Reinforced concrete stems have much greater

flexibility, flexural strength, and shear strength, making it the most cost effective for tall

retaining structures (Coduto, D. P. 2001).

2.2 Mechanically Stabilized Earth Wall

Mechanically stabilized earth (MSE) walls are retaining structures that incorporate the

use of reinforced soil slopes (RSS). MSE walls are an economical alternative to common

reinforced concrete and gravity walls. The advantages of MSE walls are highlighted during the

construction of the retaining wall, which include a higher efficiency rate in the speed of the

construction due to the repetitious steps for each layer. Reinforced soil slopes (RSS) are

especially beneficial for road widening projects since lateral stability is increased. MSE walls

also provide beneficial savings, since in most cases the soil at the site can be incorporated into

the design, which provides savings from importing soils off site. Figure 7 below shows a typical

cross section of an MSE wall. In most cases the retained backfill and reinforcement fill are the

same soil. There are many types of soil reinforcement, however geosynthetic polymer geogrid reinforcement is preferred. The properties of the geosynthetic polymers are controlled more because they are manufactured with specific strength and resistance. They also do not have the risk of corrosion like metal reinforcement (U.S. Department of Transportation Federal Highway Administration 2007). The design criteria of an MSE wall involves establishing the project requirements and evaluating external and internal stability. External stability for an MSE wall includes the same three failure modes as a cantilever retaining wall, sliding, overturning, and bearing capacity failure. Internal stability differs for an MSE wall because soil reinforcement is used. This involves selecting the type of soil reinforcement and evaluating the critical failure surface, the vertical layout of the soil reinforcement, and checking the pullout resistance of each layer.
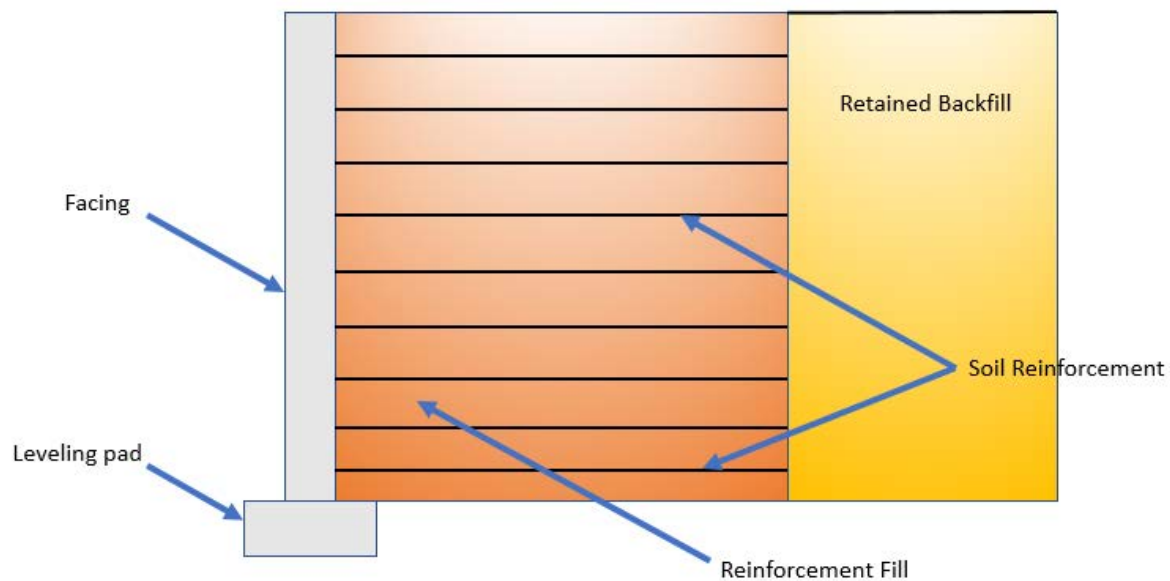
Figure 7: MSE retaining wall

2.2.1 Types of Soil Reinforcement

Geogrids are the most common soil reinforcement systems including High Density Polyethylene, PVC coated polyester, and Geotextiles. This project considered the use of geotextiles since they work in sync with RSS construction. The benefits of using geogrids over metallic reinforcement pertain to the cost of the chosen reinforcement. The reinforcement is given from the tensile properties of the geotextile fabric, which is specific to each manufacturer. It is essential that each layer constructed is pulled taut to properly reinforce the soil.

2.2.2 Reinforced Backfill Soil

MSE walls require material with specific requirements to be used as reinforced backfill soil. A well-graded granular soil is ideal considering the durability, constructability, drainage properties, and frictional angle of the material. If this type of soil is not available on-site, a local source must be utilized to obtain the necessary amount of soil per project requirements.

2.3 Bridge Design: The Superstructure

Bridges include in their design both a superstructure, or top, and a substructure, essentially, the bottom. The major components in the superstructure include the deck, slab, and girders. Data such as projected traffic loadings coincide with the design of these elements. The major components of the substructure include the piers and abutments, which are essential for transferring the loads to the foundations. Boring logs and the soil profile of the site are needed in the design of the sub structure elements as well as the foundation, which could either be deep piles or shallow footings.

The superstructure of a bridge includes the elements located above the substructure, and typically consists of the bridge deck, deck forms, structural members, cross frames, diaphragms, lateral bracing, bearings and other features such as the handrails, parapets, drainage, and wearing surface (Shaner, J. 2016). The deck forms, cross frames, steel girders, and the bearings are typical to a highway bridge. The bridge deck and steel girders, as seen in Figure 8, are crucial components to the design of a bridge since they play a major role in transferring the traffic loads to the substructure and foundation. The criteria that was important to consider in the design is the loading capacities based on MassDOT and LRFD design specifications. The strength of these members must be designed with consideration of safety, sustainability and long-term use. In addition, the serviceability of these members must be taken into consideration, to ensure the bridge is durable, crack resistant, and complies with MassDOT deflection limits.
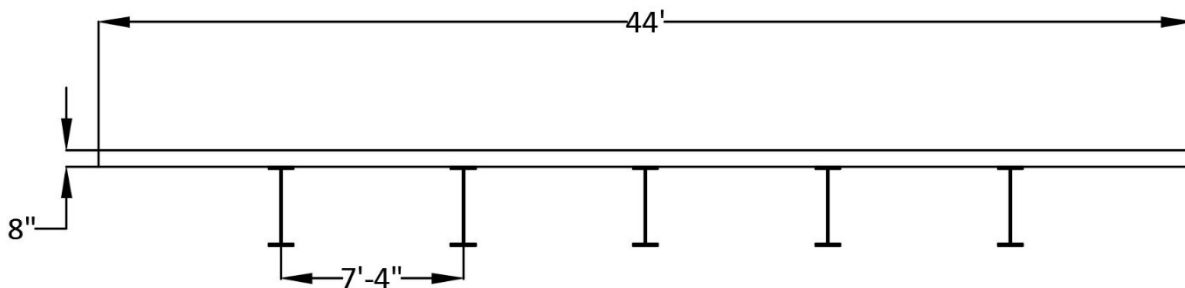
Figure 8: Bridge Superstructure

2.3.1 Wearing Surface

The wearing surface is the top layer of the deck that includes the bituminous pavement for the road. This is intended to provide a smooth riding surface for the drivers as well as protect the deck from the weather. The thickness of the layer is dependent on the volume of traffic at the location, as well as the weather conditions.

2.3.2 Bridge Decks

The deck of a bridge not only supports the wearing surface, but is responsible for transferring the vertical vehicular loads throughout the superstructure as well as providing lateral stiffness to the superstructure of the bridge (Modjeski & Masters Inc. 2003). The two common deck form types are stay-in-place deck or removable. The benefit of stay-in-place deck forms is added strength to the deck after construction is over. The bridge deck is located directly above the stringers of the bridge and has the option to be designed compositely or non-compositely. A composite design is when a concrete slab is firmly connected to the steel beams providing longitudinal shear transfer between the two members. This is accomplished by using steel anchors to connect the reinforced concrete slab to the stringers (McCormack, 2012). Composite designs provide increased strength and allow the steel beams and concrete slabs to act as a unit in resisting loads. The 1944 AASHTO Specifications approved the method of composite design, and it has been incorporated in the majority of bridge deck designs since the early 1950's (McCormack, 2012).

2.3.3 Cast-in-Place Concrete Slab

One of the most common types of concrete decks is cast-in-place concrete due to its low cost and constructability (CA.DOT. 2015). A layer of concrete is placed on site usually between 7 and 12 inches thick on top of the reinforcing steel (CA.DOT. 2015). Since concrete best provides its strength through compression, the reinforcing steel is beneficial in providing the necessary tensile requirements. As mentioned in section 2.3.2, the composite design between the deck and stringers benefits the strength of the deck allowing 33% to 50% more load to be

supported (McCormack, 2012). A cast-in-place slab has two options for construction; unshored and shored construction. Unshored construction is when the slab is cast in place after the girders are installed, this means the girders need to be capable of withstanding the wet concrete load. Shored construction is when temporary supports are added to aid the girder in withstanding the wet concrete load and construction loads.

Some disadvantages associated with a cast-in-place concrete deck are cracking and rebar corrosion. This could potentially increase the money spent on bridge maintenance and damage the wearing surface (CA.DOT. 2015). There are methods to reduce or prevent corrosion of the rebar within the concrete. These methods can include coating the rebar, using salt-free aggregates, adequate curing and complete hydration of the concrete, not using other metals in the concrete that would allow for galvanic coupling, and cathodic protection. The simplest of methods is coating the rebar with hot dip galvanizing or an epoxy and not using more than one metal type in the concrete (i.e aluminum and copper). Cathodic protection is more complicated because it requires an anode bag filled with zinc to be connected directly to the rebar (Cantrell, 2002).

2.3.4 Prestressed-Precast Concrete

The second most common deck is precast concrete, which is prefabricated concrete slabs that are either reinforced with steel rebar or are prestressed (CA.DOT. 2015). These pre-made concrete panels are delivered to the construction site ready to be connected. This advantage expedites the construction schedule and has less of a social impact than other methods. Similar to a cast-in-place concrete deck, this could be constructed to be a composite member. In order to make a prestressed structure composite, the prestressed beams would be manufactured off-site

and then a cast-in-place deck would be used. The prestressed girders will be connected to the cast-in-place slab with shear studs, the shear studs are installed manually using a welder (Shear Stud Products).

Normal concrete has a very low tensile strength, and thus cracks can develop in the early stages of loading. Prestressing fibers increase tensile and shear stress capacity at the midspan of the beam. A prestressed beam reacts more elastically, and has the ability to recover cracking and deflection, but once the tensile strength of the concrete has been exceeded it acts exactly as a reinforced member.

The use of prestressed concrete can be utilized in the bridge deck, or superstructure of the bridge, wherever concrete beams are used. Prestressed concrete comes in many varieties. Beams can be pretensioned, before the concrete is cast or post-tensioned after it has been cast. Pretensioning has an advantage in the manufacturability, as it is easier to mass-produce, and the compressive force is spread more evenly throughout the beam or slab. In post-tensioned beams there is less curing time and objects can be cast in place, and will resist elastic shortening better.

The main things to consider when designing a prestressed beam are the shape, the size, and the loading. These specifics allow the beam to be designed accordingly. The beam can be designed according to the specific project's needs.

The challenge of prestressed concrete design is that there are many variables to consider: the quality of concrete and steel components during manufacturing, compressive forces and losses after the concrete has cured, and accounting for shrinkage and creep in the beams long term. In addition, the type of anchor needs to be evaluated, as well as the size and type of tendon being used.

2.3.5 I-Girders: Rolled Beams

The most common steel beams used are W-shapes that have parallel inner and outer flange surfaces, which give the beam the distinct "I" shape. Various sizes and shapes are widely manufactured so it is essential to incorporate a size that is readily available. These types of girders are useful for short span bridges under 200 feet, otherwise a girder with a deeper web may be needed to span longer distances (AISC 2016).

2.3.6 Cross Frames, Diaphragms, and Lateral Bracing

Cross frames and diaphragms, as seen in Figures 9 and 10, provide lateral-torsional buckling resistance for steel girder bridges during construction and remain permanently fixed in the superstructure (Shaner, 2016). Lateral bracing, as seen in Figure 11, is different from cross frames or diaphragms; they provide lateral stiffness, which decreases the lateral deflections from the horizontal forces on the bridge. The horizontal forces may be due to traffic, wind, or seismic loads (Shaner, 2016)
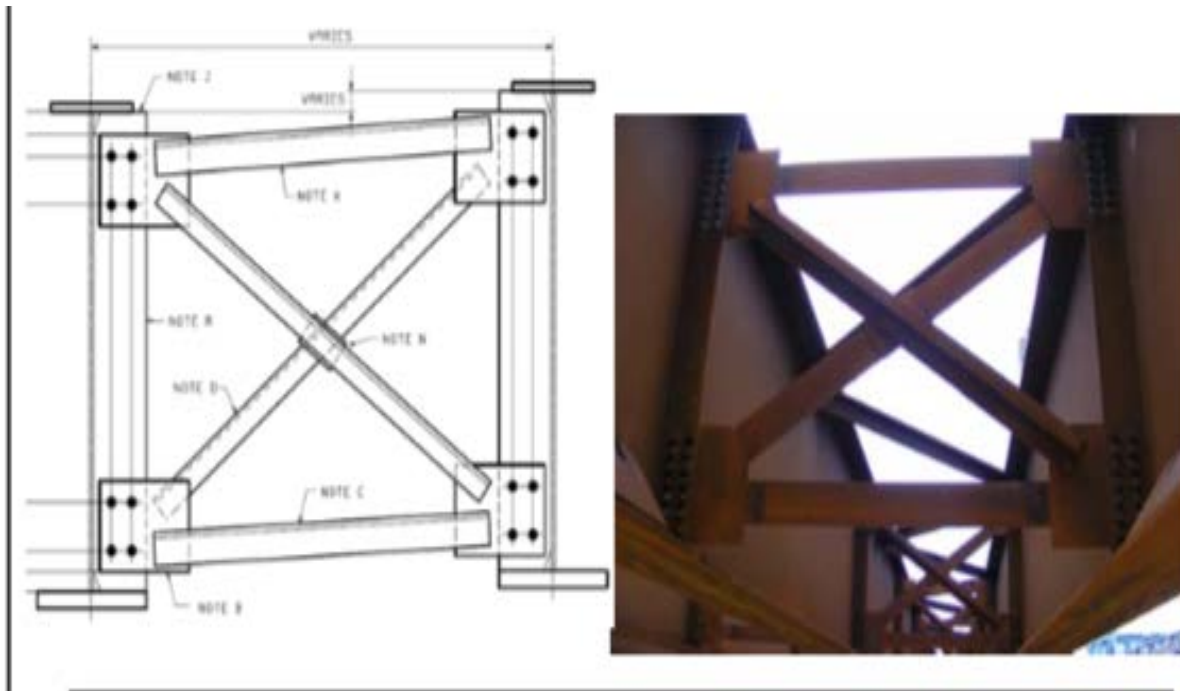
Figure 9: Cross Frames (Shaner, J. 2016)

Figure 10: Diaphragms (Shaner, J. 2016)

Figure 11: Lateral Bracing (Shaner, J. 2016)

2.3.7 Bearings

Bearings can be considered as a part of the substructure, or a component in and of itself. This is the component of the bridge that transfers the superstructure stresses through the substructure to the foundation. When designing the bearings for a bridge it is important to meet certain requirements (Fu, G. 2013):

1. Ability to transfer vertical forces from the superstructure

2. Ability to accommodate horizontal translation along the bridge's longitudinal axis due to thermal and load effects

3. Ability to accommodate rotation on the transverse axis of the bridge

4. Ability to function as a tie down system to secure the superstructure to the substructure to prevent uplift

In order to accommodate both steel and concrete girders, rollers (Figure 12) and elastomeric bearings (Figure 13) were both used for the design. These bearings allow translational and rotational movement to minimize the stresses given from the superstructure (Fu, G. 2013). The design of the bearings must focus on the maximum load carrying capacity and be able to withstand the translational and rotational stresses.
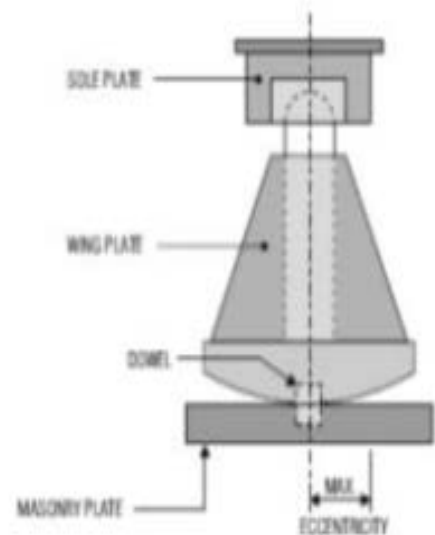
Figure 12:Rocker Bearing (Shaner, J. 2016)
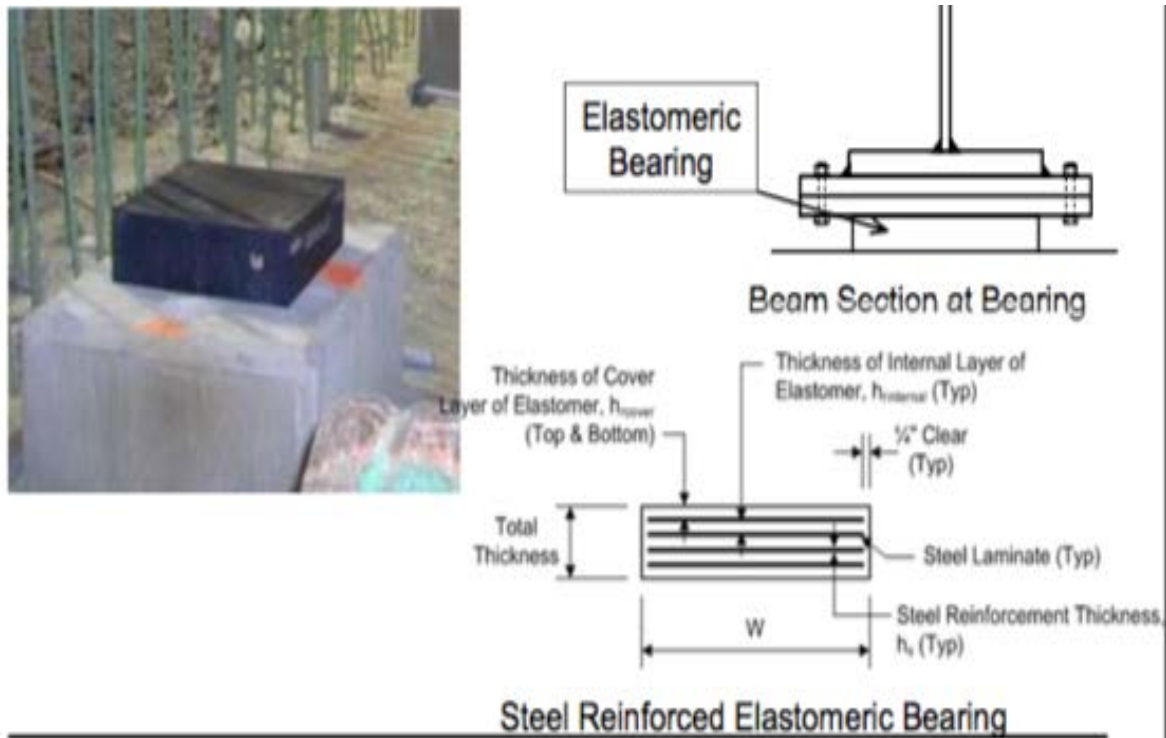
Figure 13: Elastomeric Bearing (Shaner, J. 2016)

2.4 Bridge Design: The Substructure

The basic definition of the substructure of a bridge is anything below the superstructure, which includes: any abutments (end bents), piers (bents), pier caps (bent caps), or columns (FIG 20/22 AISC). Each of these elements is critical in the design of the bridge and must be designed, like the superstructure, with consideration for sustainability, safety, and long-term use.

2.4.1 Abutments (End Bents)

The abutment is where the roadway ends and the bridge begins. Its purpose is to support the loads of the superstructure and the lateral soil pressures from the roadway embankments. Different characteristics need to be considered when choosing an abutment type, including

bridge geometry (e.g length, clearances), anticipated loads, future maintenance, and constructability. Figure 14 below is an example of an abutment. The abutment, similar to a retaining wall, has two main components. The footing as identified by the number two, which has a toe and a heel. The toe is exposed and the heel is buried. The heel in this case is on the right and the toe is on the left. The other component is the stem, which is labeled as number one.
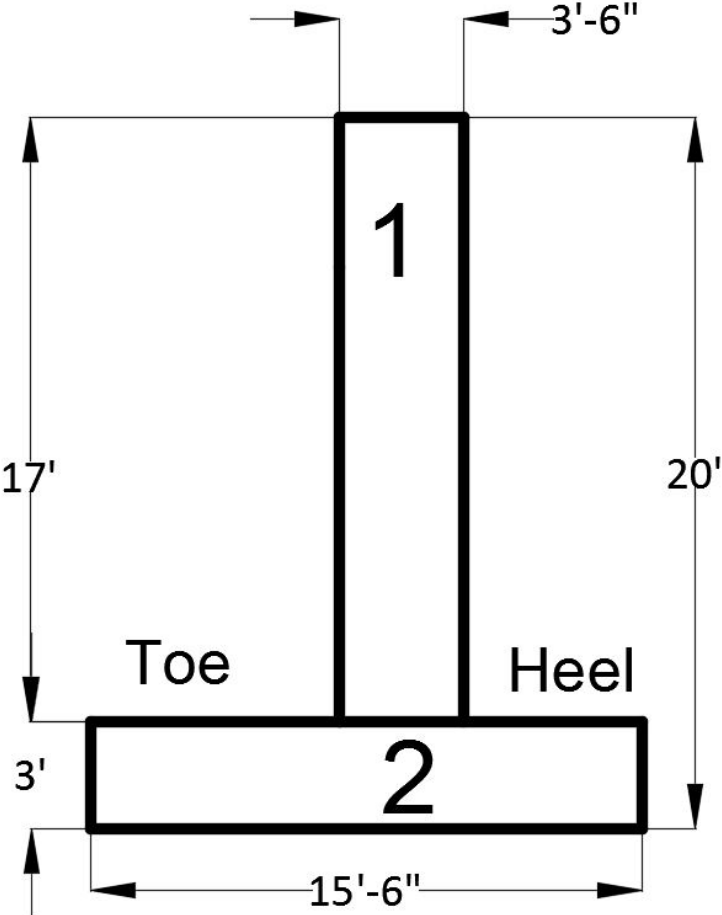
Figure 14: Typical Abutment

2.4.1.1 Conventional Abutments

This abutment type is characterized by a joint separating the bridge deck from the approach and back wall, expansion joints, wing walls, and includes a bearing that separates it

from the superstructure. A conventional abutment can be tall or "stub". Tall abutments can function as a retaining wall and do not require the use of a header slope. A header slope is used to reduce the lateral pressure on a wall. Stub abutments are usually capped at a nominal height and require a header slope of anywhere between 4:1 and 1:1. A stub abutment needs to be combined with a retaining wall in front of it.
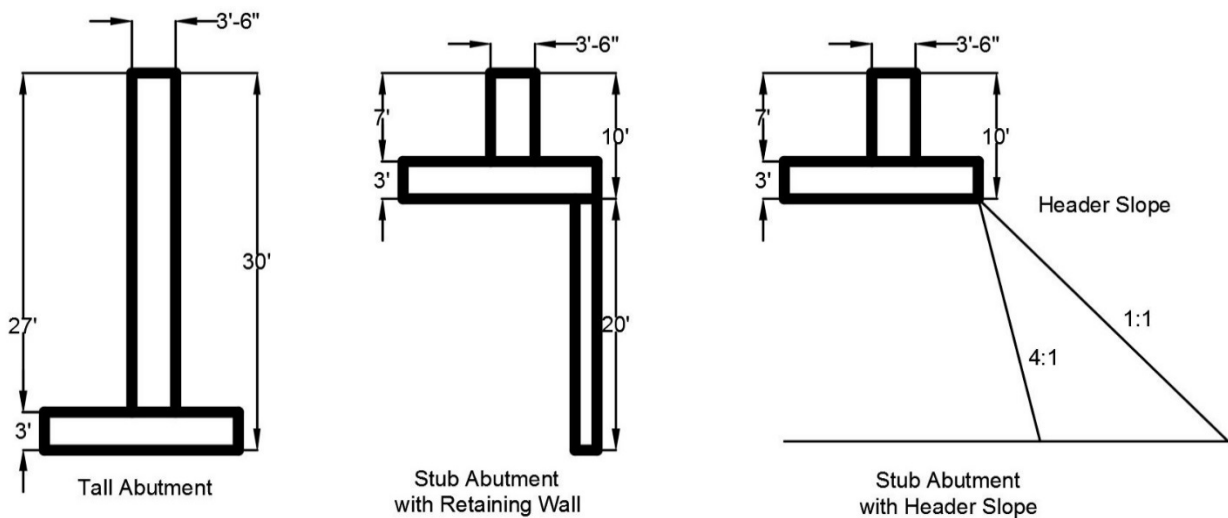
Figure 15: Different Conventional Abutment Types

2.4.1.2 Integral and Semi-Integral Abutments

An integral or semi-integral abutment is a system in which the different features of the bridge: superstructure, abutment, and foundation are all integrated together. The superstructure is set on top of the abutment cap and a closure pour ensures the superstructure is cast into the abutment. A concrete pour isn't always used. Other methods like reinforcing structures, or anchors are also employed. Integral abutments offer no designed moment relief, although sometimes have inherent moment resistance. The design implications of this are that no moment needs to be calculated because it is assumed the integral abutment is not a fixed end connection. However, it is still connected and can resist some moment. Since the foundation is integrated

into the entire abutment, H piles or drilled shafts and spread footings are used to support the abutments. The abutment can be similar to a conventional abutment, where the wall can be "stub" or tall. These types, like the conventional, also include wing walls, and expansion joints. What classifies the type as integral or semi integral is the extent to which the superstructure and foundation is connected to the abutment. The figure below shows an integral abutment; notice the notch at the top where the bridge girder and bearing sit. A semi-integral structure usually has some type of bearing to account for intentional moment relief.
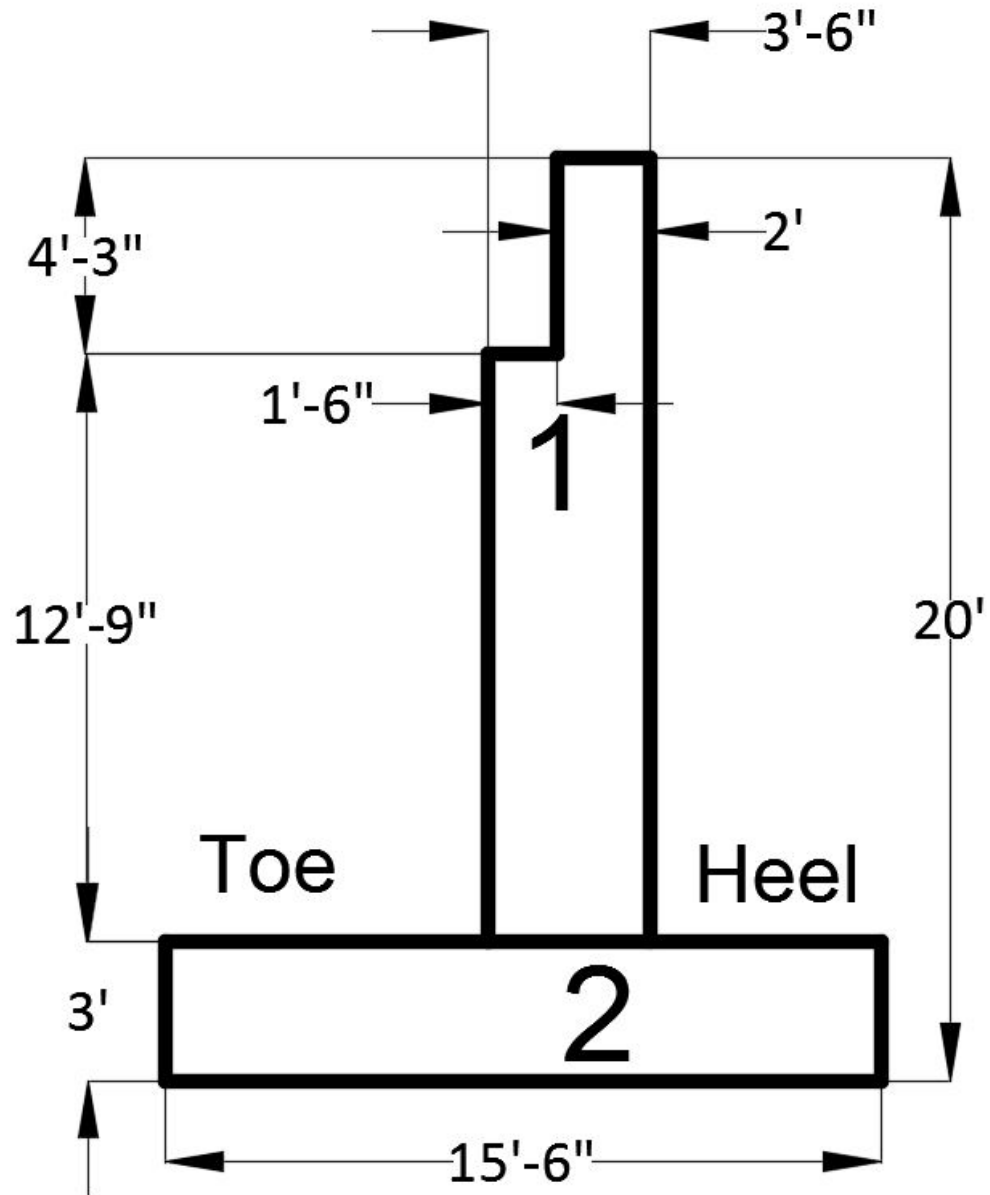
Figure 16: Typical Integral Abutment

2.4.2 Piers (Bents)

The basic pier elements include the pier cap, vertical support, and the footing/support; see Figures 17 and 18 below.
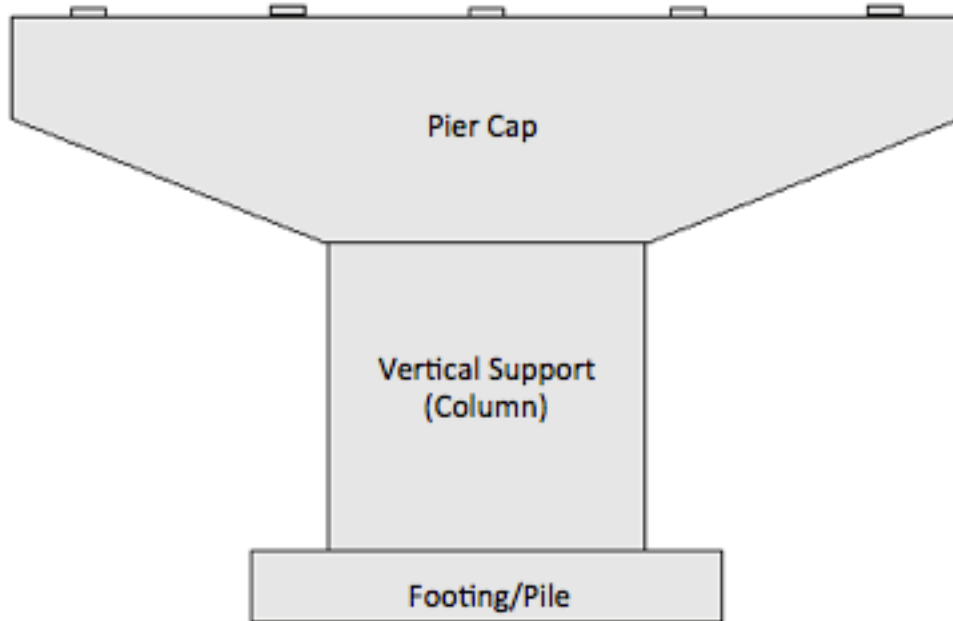
Figure 17: Pier Example (Single Column)
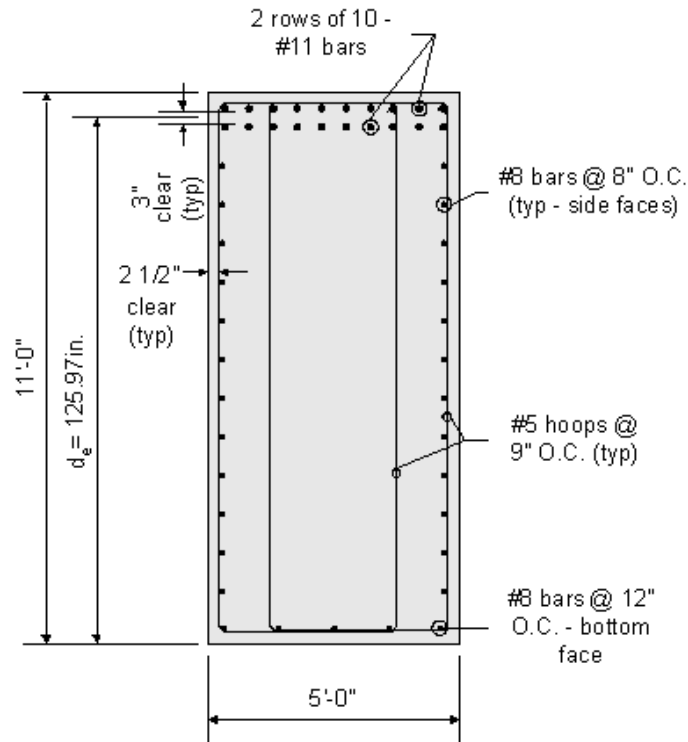
Figure 18: Pier Cap – Plan View (30)

All of these elements are most commonly fabricated from steel or concrete. There are a large variety of possibilities when designing because there are such different components to each pier. Each pier combination has its own benefits and risks; however, the goal is to choose the right pier for the bridge that is being designed. The local site conditions, and vehicle traffic are important to consider, as well as, the aesthetics and proportions. Piers should also be analyzed across both axes because the loading capacity and moment behavior will change depending on the direction of loading.

The pier caps are often integrated into the pier or superstructure. This can help improve efficiency in constructing or loading; it can also improve the aesthetics, and can improve clearances. As with the piers themselves, there are many options when choosing the best pier

cap. The pier cap choice comes down to the location, material, size and configuration or the piers. It's function is to capture the loads of the superstructure via contact with the girders and transfer them to the pier column and into the footing and foundation of the bridge's substructure. An expansion joint is included below the superstructure to alleviate any shrinkage or expansion of the materials due to temperature or curing, and to assist with any minor movements of the structure.

2.4.3 Driven Pile Foundations

Designing foundations involves a few sub disciplines of civil engineering in order to fully understand the key concepts and the process for determining dimensions, load resistance, and construction methods. Foundations are structural components that carry the loads from the structure to the soil beneath and around it (Coduto, D. P. 2001). Foundations significantly depend on the soil properties and parameters from the geotechnical report. Lastly, foundations must be economically built for the sake of construction costs. The materials, methods, and any sort of construction constraints must be planned and designed for ahead of time (Coduto, D. P. 2001). Driven piles have been the preferred deep foundation for bridge design, especially for marine or near shore applications. Driven piles are also environmentally friendly, leaving the construction site virtually clean and debris free, although there is a noise and vibration factor. This deep foundation is driven to a required design depth for sufficient resistance against compression, tension, and lateral loads. Pre-drilling may be necessary if the driving needs to penetrate dense soil to the required depth (Baker, H. 2016). Driven piles are created to ensure sufficient quality, reliability, and strength to conform to ASTM standards. Driven piles maintain

their shape and integrity during the driving or installation process and can be verified visually and dynamically.

Dynamic and static tests can determine adequate load carrying capacities and effects of hammer performance on the foundation. Usually driven piles are the most economical foundation option for many projects, and are the most structurally superior compared to other foundations. "The wide variety of materials and shapes available for driven piles can be easily fabricated or specified for high structural strength, allowing them to be driven by modern hammers to increased working loads thus requiring fewer piles per project, resulting in substantial savings in foundation costs," (Association, P. D. C. Benefits of Driven Piles. 2016). When driven into water, these foundations can immediately be ready for use, which reduces construction time of the project. "For bridges or piers, driven piles can be quickly incorporated into a bent structure allowing the bridge pier itself to be used as the work platform for succeeding piles in top-down construction," (Association, P. D. C. Benefits of Driven Piles. 2016). "After the pile is driven into the site, the foundation can actually have increased load carrying capacity because of the driving process. This phenomenon is called 'setup' which can produce a need for fewer or shorter piles, saving on construction costs such as time, labor, and materials" (Association, P. D. C. Benefits of Driven Piles. 2016). Driven piles are very adaptable according to structure type, site details, and budget constraints. These foundations can be steel (tapered, shell, or sheet pile), concrete (square, cylinder, or sheet pile), or timber (Association, P. D. C. Benefits of Driven Piles. 2016).

2.5 Project Management

In developing the design and constraints for solving the problems affecting North Lake Avenue, it is critical to assess how the design shall be brought to fruition, given the complexity of the problem. Much of the MQP, especially post-design phase, was the conceptual cost estimate and the creation of a scope of work in the form of activities. Beyond the design of the bridge and retaining wall structure was the implementation and building of the project, due to its relevancy in the solution of the engineering problem at hand. The social, traffic, complicated landscape, and environmental issues will be analyzed through cost, scheduling, and environmental impact.

The proximity of the road to the Lake Quinsigamond, the residencies bordering North Lake Avenue, and the bottleneck of traffic at the location justify an in-depth look at the methodology by which a framework for building the project can be designed. Coordinating equipment and scheduling to minimize noise and disruption of traffic flow is a paramount consideration in the approach to this project. Communication of the plan to local residents prior to commencement of work is also an important factor in building as their cooperation and our accountability to them is a large consideration at every point in the project.

2.6 Societal Impacts of Public Construction

Due to the proximity of the collapsed road to residential areas, construction cannot commence without disrupting the access that residents must their homes, as the inevitable closure of the road will prevent street access on the North Lake Avenue side. One of the main problems that the project must address is how to effectively perform construction such that the

residents are not significantly disturbed, and they are able to carry on with their day-to-day affairs in a way that is satisfactory to them.

In a study performed regarding road construction in China that impacted local residents, they determined that "inefficient communication is the most critical risk where public awareness plays a mediation role" (Wang, Han, et. al, 2016) Given that North Lake Ave, directly borders several properties, some of which depend on the road for partial to full access to parking and/or pathways to their home, disturbance of access to their home during paving or the installation of any bridge or retaining wall for some amount of time is virtually inevitable. Given this fact, it is necessary to approach this critical issue with the idea of efficient communication at the forefront of the project's mindset.

The nature of the repair of this road is ultimately a necessary thing, and formulating an understanding in the residents that are affected by this issue is thereby a necessity as well. In the approach to designing and constructing this project, designing a platform of open communication with the residents regarding the parameters of the project is just as necessary. Designing around this constraint, and including the strategies in our cost, will have to be a priority, given the fact that the creation and completion of road repairs on North Lake Ave depends on it.

3.0 Methodology

The purpose of the methodology is to define the critical steps taken for the design of each component. A variety of sources were utilized to establish the design criteria for the cantilever retaining wall, MSE wall, bridge superstructure, abutments and piers, and foundation. Each design was outlined using flow charts to depict each essential step. Spreadsheets and design software were used in conjunction with the hand calculations to efficiently input iterative data.

3.1 Evaluation Methods

The deliverable of this project is to propose an alternative bridge and retaining wall design to what is currently being implemented by the City of Worcester. The loading calculations for the soil and traffic will be following the *American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications 2012 Manual* (28) and the *U.S Department of Transportation Federal Highway Administration* (30), as well as the *Massachusetts Department of Transportation Highway Division Standard Specifications 1988* (25). Based on the intended use for the designs, it was determined that the specifications manuals were the most appropriate.

3.1.1 Criteria

The basic categories by which the scope and constraints will be evaluated are: cost, schedule, and environmental impact. These categories were based on the needs of the

surrounding area and the city and then based on capstone design requirements. The following subsections provide more detail on how our team evaluates each category.

3.1.1.1 Cost

The cost of the project was based on the scope of work that all the elements of the proposed designs included. For each design, including retaining walls, bridge superstructure, substructure, pile caps, abutments, girders, and other elements, the total quantities of material were gathered, a system of activities was developed, and the total man-hours for the implementation of the project was generated, which formed a detailed estimate for the designs.

3.1.1.2 Scheduling

The schedule of this project is an important logistical factor and necessary complement of the design in terms of real world application; knowing the breakdown and sequence of each specific activity is a critical element in comparing designs, and selecting a solution. In terms of a comprehensive and detailed plan to construct the designs, generating a schedule of activities and using other case studies to determine the correct order and duration of activities would fulfill this aspect of the project. Using case studies with project management applications, the bridge and retaining wall designs were compared to other similar projects and a sensible and coherent breakdown of activities for the construction of a bridge and retaining wall was constructed. Combined with a cost estimate, the schedule was an integral part of identifying a reasonable solution.

3.1.2 Soil Profile

A soil profile was created using the boring logs from the geotechnical report created in 2010. Borings 1 through 4 were used to create an average depth of each layer. To be conservative, the shallowest ground water table value recorded from the boring logs was used.

3.2 Cantilever Retaining Wall Design

The design of the cantilever retaining wall was based off examples referenced from *Foundation Design: Principles and Practices* by Coduto D. P. (2001) and *Basics of Retaining Wall Design 10th edition* by Brooks H. and Neilsen J. P. (2013). Before any calculations were performed, the primary flowchart below (Figure 19) was made to organize completing the tasks and outline the methodology of the design. From this primary flow chart, secondary ones were made to provide more step-by-step details for designing the elements of the cantilever retaining wall.
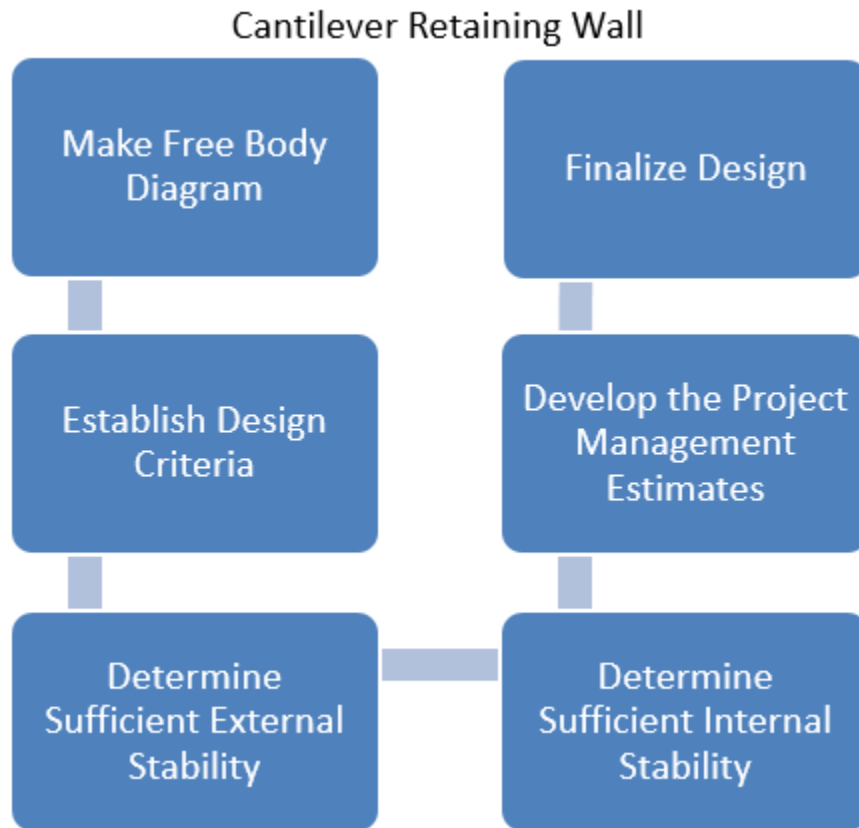
Figure 19: Cantilever Flow Chart

The design of the cantilever retaining structure began with the Free-Body-Diagram (F.B.D.), as seen in the flowchart above, and a basic rendering of the cross section of the structure. A free-body diagram sketch was made to visualize external forces and loads acting on the retaining structure. The free-body diagram was drawn with the forces and loads being considered in this design, shown below in Figure 20. Distributed loads were converted into resultant point loads to not only help visualize the forces acting on the structure but also to determine overturning moments. F1 represents the distributed load of the soil and F2 represents the surcharge load.
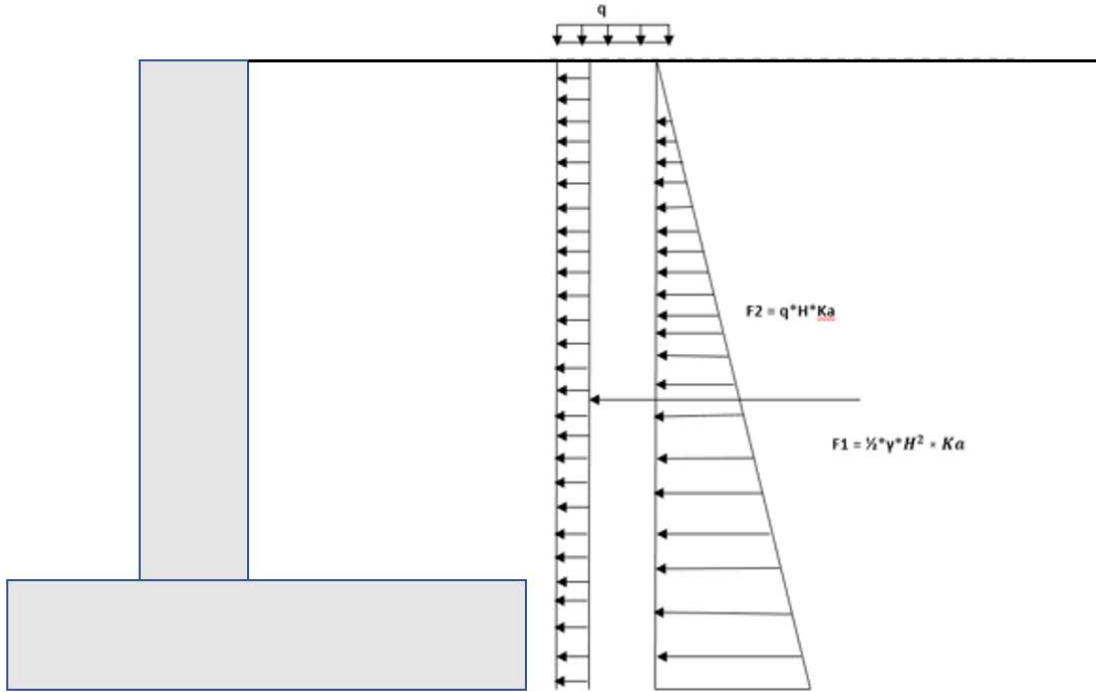
Figure 20: Free-Body-Diagram of Cantilever Retaining Wall

The dimensions such as footing length and stem height were chosen using the suggested first trial dimensions for cantilever walls backfilled with sandy soils, shown in Figure 21 below. Hand calculations for external and internal stability were based on these preliminary dimensions which also served as a reference to establish an excel spreadsheet for further repetitive calculations. Table 2 below has the dimensions used for the first trial of the stability analysis.
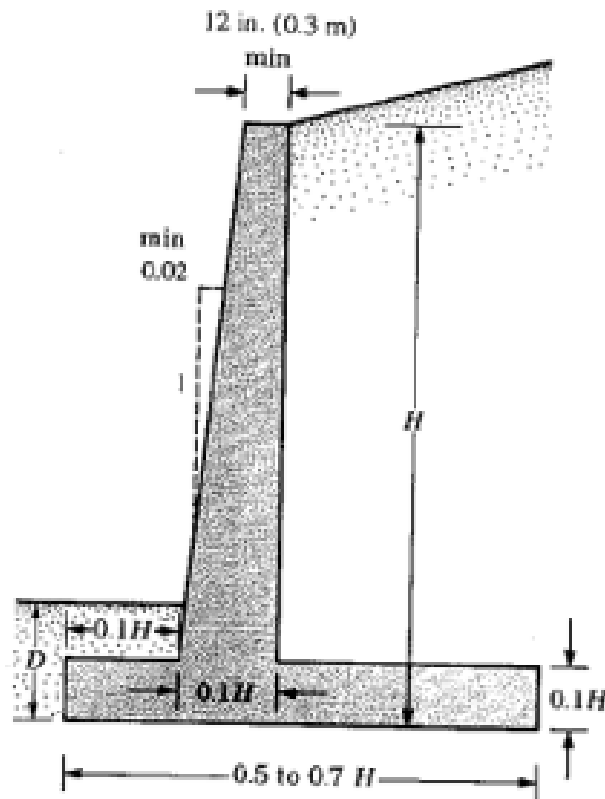
Figure 21: Suggested Retaining Wall Dimensions (Coduto, D.P. 2001)

Table 2: First Trial Dimensions

	H	0.1H	0.7H	D
Dimension (ft)	12	1.2	8.4	5.2

Once the dimensions and F.B.D. were completed, the design criteria were established based on the soil profile. The following flow chart below, shown Figure 22, assesses which criteria was necessary to perform external and internal stability calculations. Below in Table 3 is the list of soil and site parameters used for the stability analyses.

Design Criteria

Figure 22: Cantilever Retaining Wall Criteria Flowchart

Table 3: Site and Soil Parameters for the Site and Soil used in Stability Calculations

Site & Conventional Parameters	Soil Parameters
- Unit weight of concrete (150 pounds per cubic foot) - Surcharge (traffic) load (240 pounds per square foot) from AASHTO - For overturning calculations, the clockwise direction was considered the positive reference frame - For bearing capacity calculations Terzaghi's method and equations were used - The surcharge load was considered laterally as well as vertically	- Groundwater table location for hand calculations assumed to be negligible - Coefficient of friction between the soil and footing of the retaining wall (0.55) - Frictional angle of backfill soil (35 degrees)

40

3.2.1 External Stability

To acquire sufficient dimensions against the possible failure modes of overturning, sliding, and bearing, the factors of safety must be within the acceptable limits. To begin these design calculations, the necessary soil parameters were extracted from the geotechnical report and soil profile. The design criteria flow chart below shows the steps that were taken in order to design an externally sufficient and stable cantilever retaining wall.
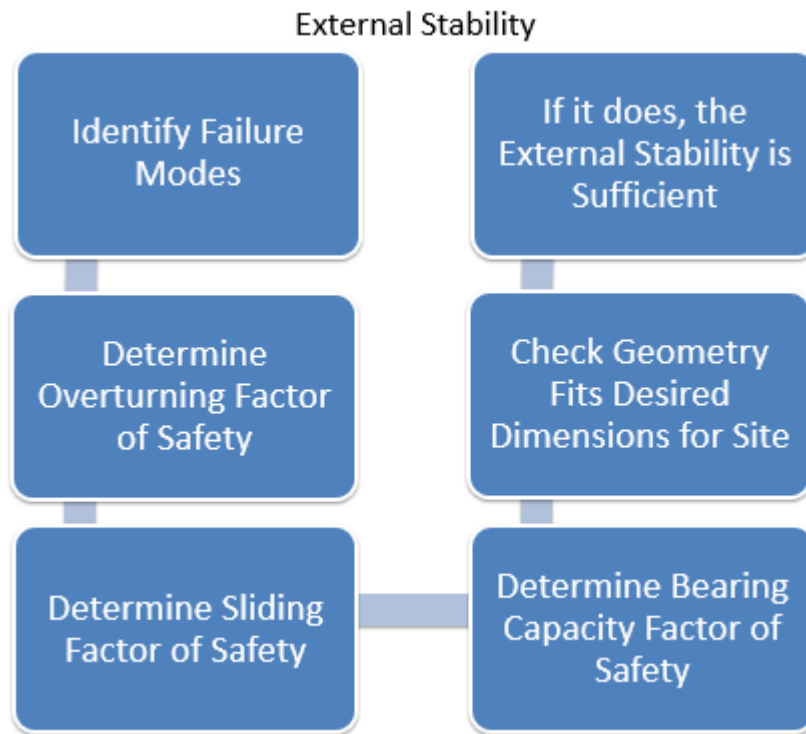
Figure 23: Cantilever Retaining Wall External Stability Flowchart

3.2.2 Overturning

The following Overturning flow chart shows the steps taken to determine the overturning factor of safety for the cantilever retaining wall. Figure 25 below the flowchart were the forces

that were considered in the overturning stability analysis. The overturning moments due to lateral forces and vertical forces were determined by multiplying each force by its corresponding moment arm (Hr and Hw respectively). The moment arm is the perpendicular distance from the line of action of a force to the point of reference. Point O was chosen as the toe of the structure, the point of reference, shown in Figure 25.

External Stability for Overturning Moment

Determine Moment Arm for Resultant Loading	If F.O.S is Above 1.5, There is Sufficient Overturning Resistance
Calculate Driving and Resisting Moment in to Point	If F.O.S is Below 1.5 Adjust Dimensions and Check Again
Calculate Factor of Safety for the Overturning Moment	Check Factor of Safety (F.O.S) is Above 1.5
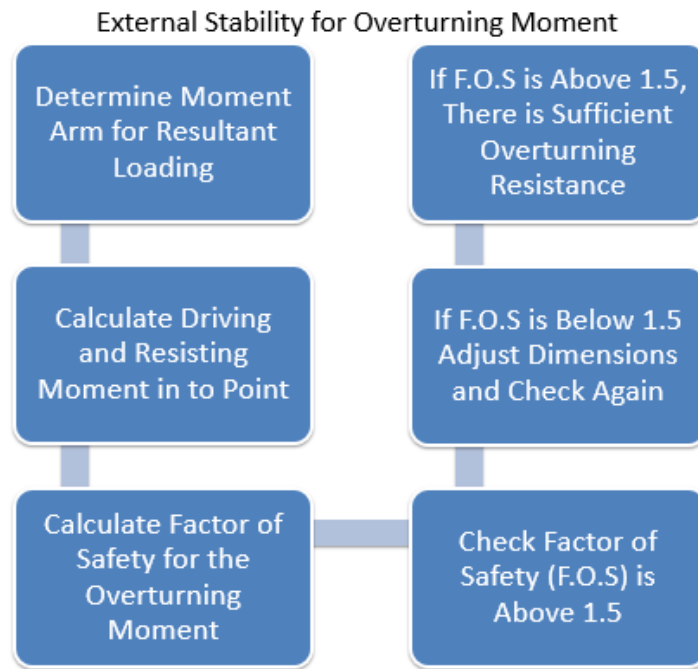
Figure 24: Overturning Flowchart
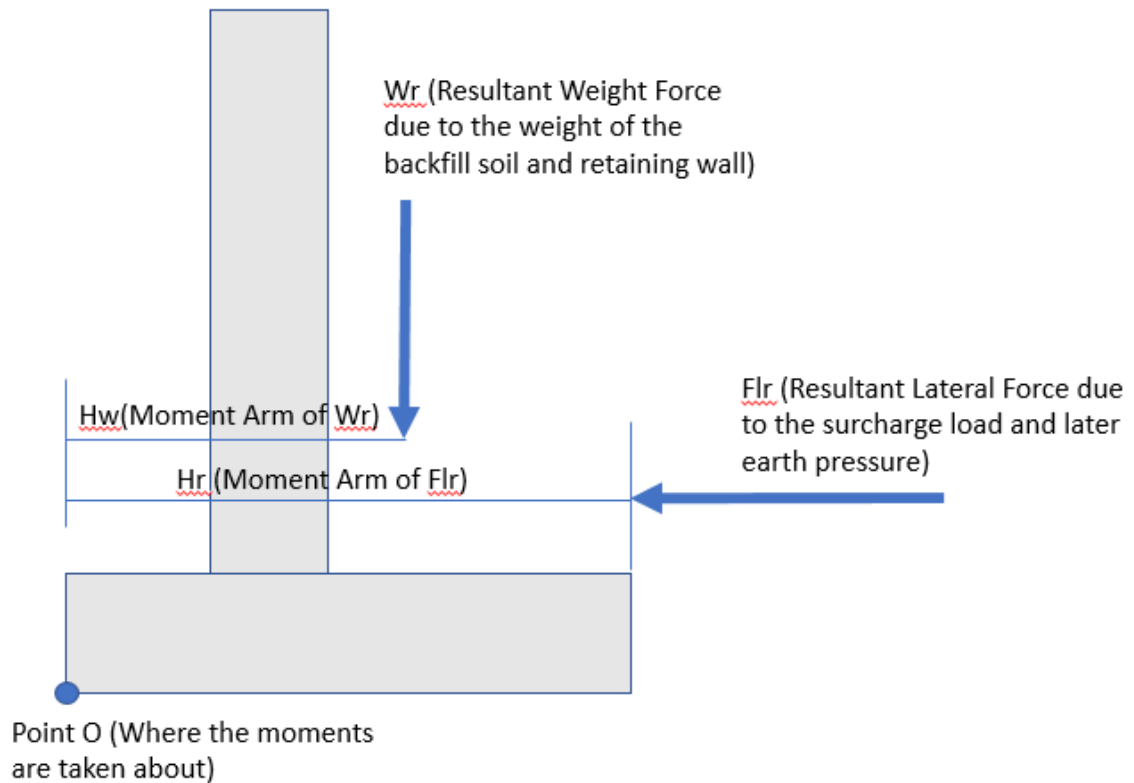
Figure 25: Soil Forces in Overturning Failure

The factor of safety for overturning failure was based on calculating the resultant lateral force, resultant vertical force (from the weights of the wall itself and backfill soil), and their corresponding moment arms. The moments of these resultant forces were then calculated with respect to point O from the Free-Body-Diagram shown in Figure 25. Once these were calculated, the driving and resisting moments based on the reference frame were then established. The driving moment was the moment producing rotation in the counterclockwise direction, which was the moment due to the resultant lateral force. The resisting moment was the moment producing rotation in the clockwise direction, which was the moment due to the resultant weight.

Once established, the factor of safety was calculated by dividing the resisting moment by the driving moment. The minimum factor of safety for overturning is 2 (Coduto, D. P. 2001).

3.2.3 Sliding

The Sliding flow chart below shows the steps taken in order to determine the sliding factor of safety for the cantilever retaining wall. The lateral forces on the passive side of the retaining structure were not accounted for to be more conservative when calculating the factor of safety for sliding as shown in Figure 27. The lateral forces on the opposite side of the backfill will only benefit the design, so it is conservative to not consider them to provide a sufficient factor of safety.

External Stability Sliding

Using Free Body Diagram Calculate Shear Force at Bottom of Footing

Calculate the F.O.S for Shear

If F.O.S is Above, The Sliding Resistance is Sufficient

Check Factor of Safety (F.O.S) is Above 1.5

If F.O.S is Below 1.5 Adjust Dimensions and Check Again
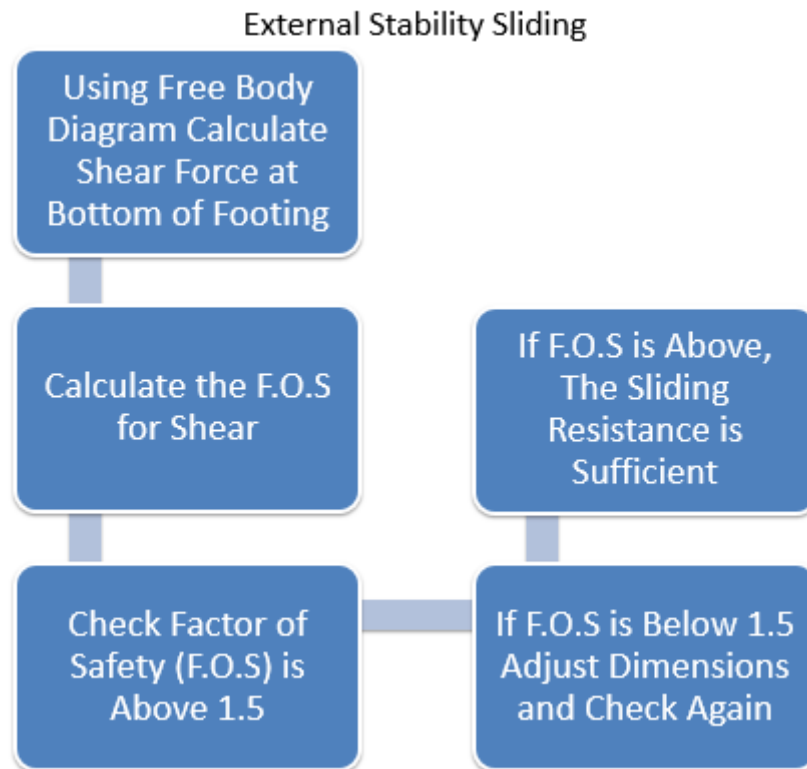
Figure 26: Sliding Flowchart

Wr (Resultant Weight Force due to the weight of the backfill soil and retaining wall)

Flr (Resultant Lateral Force due to the surcharge load and later earth pressure)

Vf (Frictional force due to Wr & the Coefficient of friction)
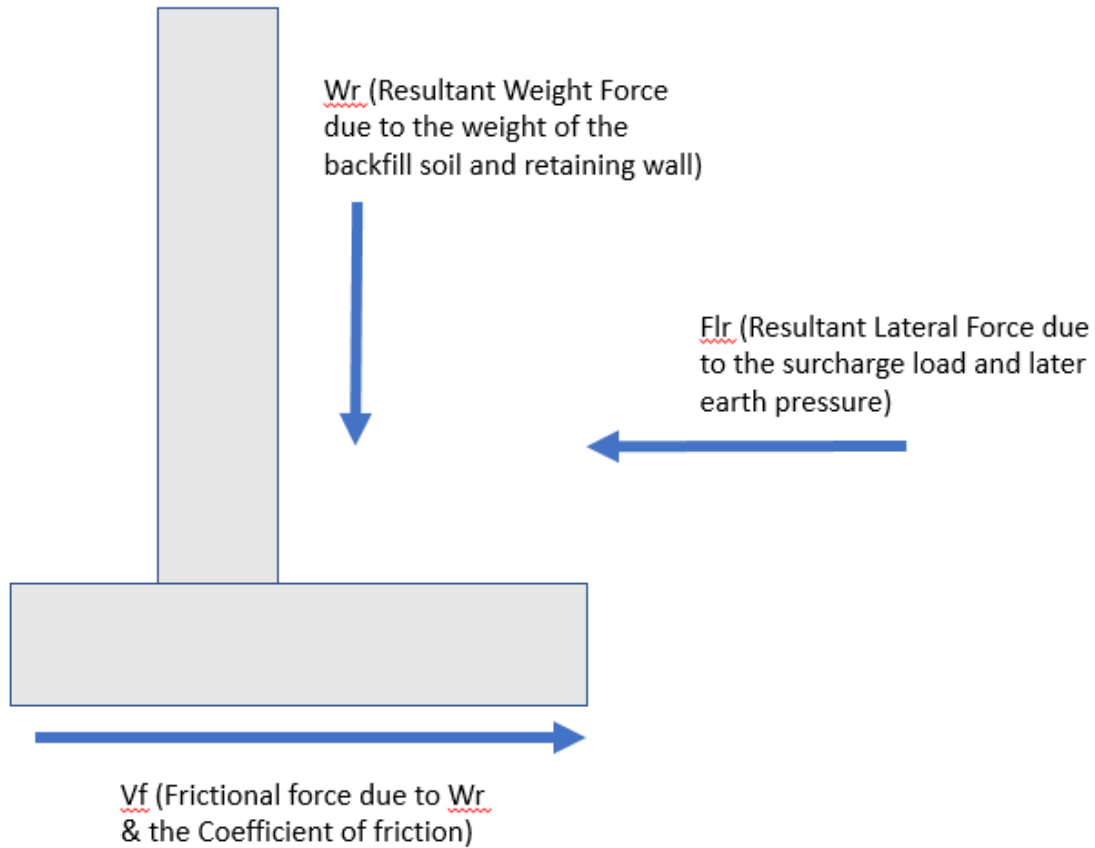
Figure 27: Forces in Sliding Failure

The factor of safety for sliding failure was based on calculating the shear force along the footing of the retaining structure and the resultant forces (lateral and vertical) previously calculated in the overturning factor of safety. To determine the shear force, the shear stress was multiplied by the retaining wall cross sectional area. Once this was calculated, the driving and resisting forces were then determined based on the reference frame. The driving force was determined as the resultant lateral force pushing the wall away from the backfill soil. The resultant force was determined as the shear force acting along the base of the footing towards the backfill soil. After these were established, the sliding factor of safety was calculated by dividing the resisting by the driving forces.

3.2.4 Bearing Capacity

To calculate the bearing capacity of the soil against the weight of the retaining wall and lateral forces, the following flow chart was made.

External Stability Bearing Capacity

Determine Moment Arm for Resultant Loading	Use Terzaghi's Method to Determine and Ultimate and Allowable Bearing	Is qmin Larger Than 0 and the Allowable Bearing Pressure Larger Than the Maximum
Calculate the Sum of the Moments	If It Is Calculate Minimum and Maximum Bearing Pressure based on e	If Both Are True bearing Capacity is Sufficient
Calculate Resultant Force from the Base Point	If it is Not Change Dimensions and Calculate Again	If It is Not True Change the Dimension
Calculate Eccentricity (e)	Check Eccentricity is Less B/6	

Figure 28: Bearing Capacity Flowchart

The first step in calculating the bearing pressure factor of safety was to determine how far away the resultant force (OE), due to the vertical loads (weight of structure and surcharge) and lateral loads (soil and surcharge), was from point O, shown in Figure 29. The distance was found by taking the ratio of the sum of the moments to the sum of the vertical forces, which was then subtracted from the length of the footing divided by two to determine the eccentricity. Once the

eccentricity was calculated, the minimum and maximum induced bearing pressure was

calculated.

Fr (Resultant Force due to Wr and Flr)

- Wr: Resultant Weight Force due to the weight of the backfill soil and retaining wall
- Flr: Resultant Lateral Force due to the surcharge load and later earth pressure

Point O

OE (Horizontal distance from Fr to point O)
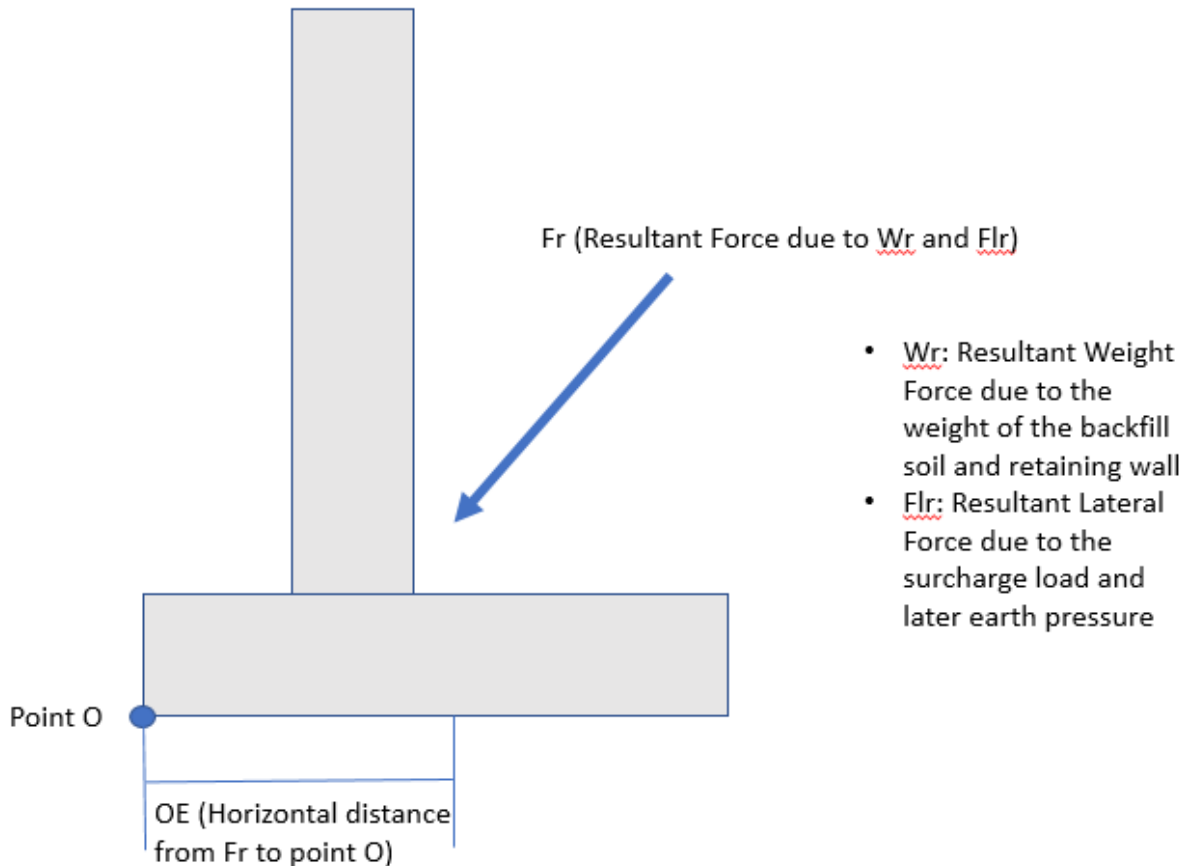
Figure 29: Forces in Bearing Capacity Failure

Terzaghi's bearing capacity theory was used to find the ultimate bearing pressure the soil

could withstand. This method is based on the three different zones: the wedge zone, radial shear

zone, and the linear shear zone underneath the footing. Using soil parameters from the soil

profile, the ultimate bearing capacity was calculated. The ultimate bearing pressure was then

divided by the maximum bearing pressure to get the factor of safety. Based upon the type of soil

beneath the footing of the retaining wall, the bearing capacity factor of safety must be equal to or

greater than 3 to provide sufficient resistance to bearing capacity failure.

3.3.5 Internal Stability

Once the external stability of the cantilever retaining wall was completed, developing a structural design with sufficient structural integrity to safely resist the loads completed the internal stability. The steel reinforcement was designed with a top-to-bottom approach beginning with the stem, then into the footing. The stem, footing heel, and footing toe designs were each conducted separately following the flow chart.
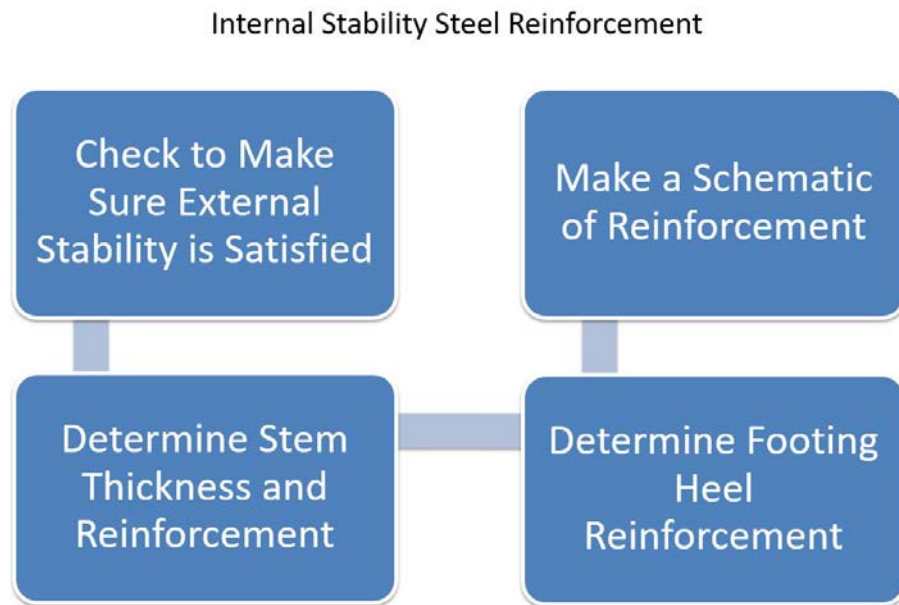
Figure 30: Cantilever Retaining Wall Internal Stability

3.2.6 Stem Thickness and Steel Reinforcement

The flowchart shown below shows the steps taken when determining the thickness and steel reinforcement of the stem. The design process began with calculating the nominal and factored shear force per unit length of the wall. These values governed the required minimum thickness of the stem as well as the effective depth of the steel reinforcement. Reinforcing steel

bars were chosen to satisfy the calculated required steel area per unit length of the stem for the

flexural and longitudinal design considered from the gross area of the system. Increasing the

vertical flexural steel bar size replaces the use of special bars to provide sufficient required steel

area per unit length of wall. Vertical flexural steel can be cut off, shortening the length of the

required amount of steel was then determined. The bending moment due to the lateral loads was

a cubic equation, so the flexural stresses in the stem decrease exponentially (Coduto, D. P. 2001).

The steel within the stem along the transverse direction was then determined, even though

theoretically there were no flexural stresses in this direction. Non-uniform soil or isolated

surcharge could induce these flexural stresses. Steel reinforcement along this direction also

protects against temperature and shrinkage stresses within the retaining wall (Coduto, D. P.

2001).

Stem Reinforcement
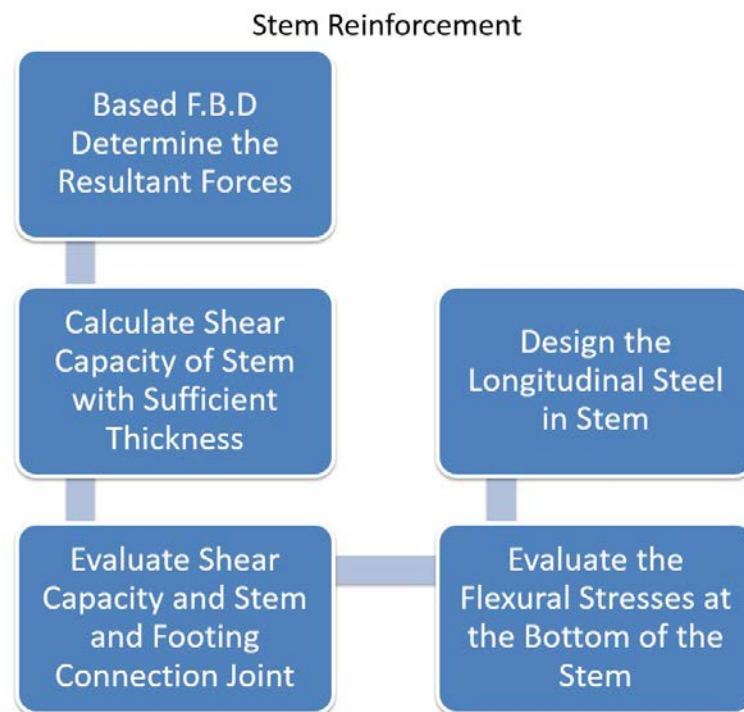
Figure 31: Stem Thickness and Steel Reinforcement Flowchart

3.2.7 Steel Reinforcement for Footing Heel

The flowchart shown below shows the steps for determining the thickness and steel reinforcement of the heel of the footing. The development length of the vertical steel from the stem governs the minimum required thickness of the footing, due to the 90-hook connecting the stem and footing. Since at least 2 inches of cover will be used beyond the hook, the development length was multiplied by a modification factor of 0.7.

Heel Reinforcement

Based F.B.D Determine the Resultant Forces	Design the Longitudinal Steel in the Heel
Calculate the Minimum Required Footing Thickness	Compute the Flexural Analysis for Heel
Design the Heel Extension	Check Shear Capacity from the Shear and Moment along the Vertical Plane of Footing
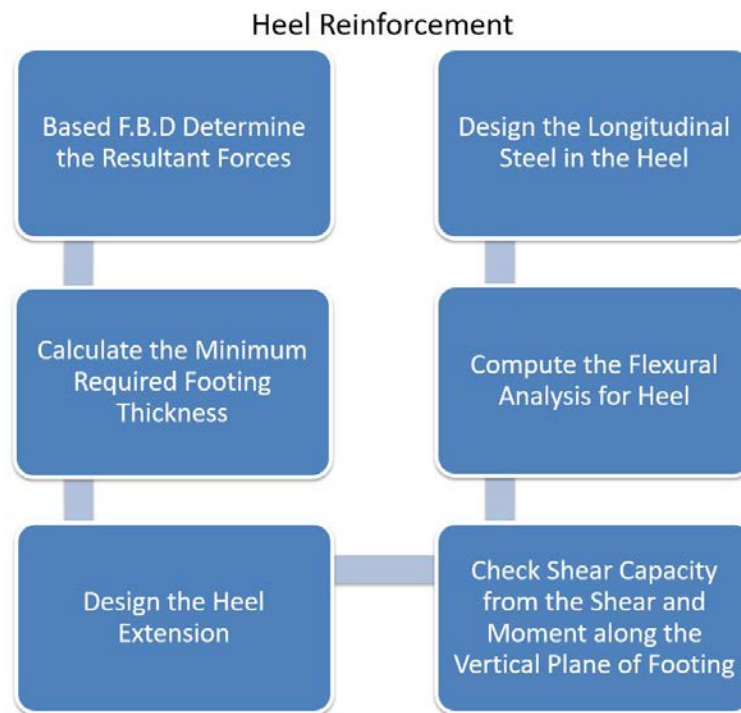
Figure 32: Heel Reinforcement Flowchart for Cantilever Retaining Wall

The loads acting on the portion of the heel are directly due to the weight of the backfill soil and the weight of the concrete, which govern the shear and flexural stresses. To be conservative, the bearing pressure acting along the bottom of the heel is ignored. The factored shear and moment that were calculated were used to check shear capacity and select the flexural reinforcement. Since the weight of the backfill soil and footing are dead loads, a load factor of 1.4 was used.

The reinforcing steel in the footing heel should be placed 3 inches from the top of the footing and

extend 3 inches above the bottom of the footing (Figure 33).
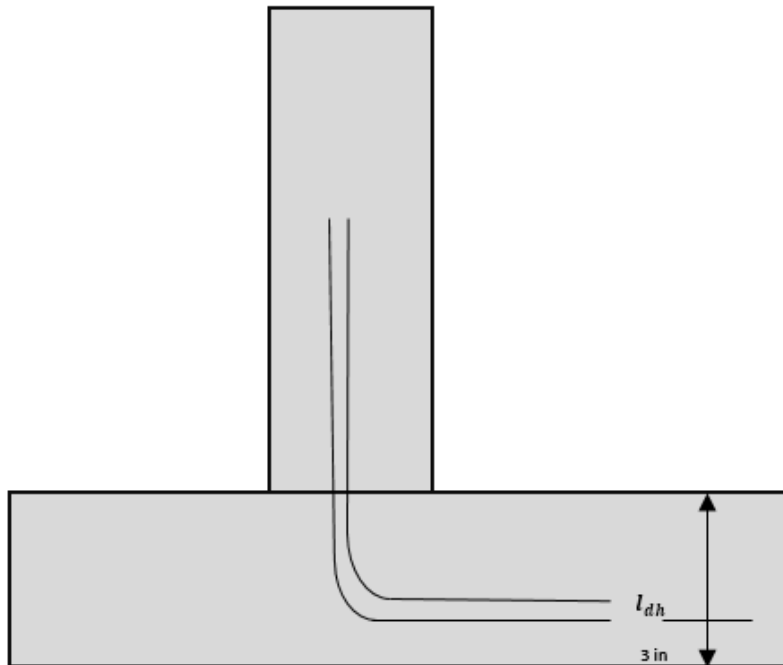
Figure 33: Heel Reinforcement in Cantilever Retaining Wall

3.2.8 Settlement Analysis

The settlement calculations began with Split-Spoon Penetration Tests (SPT) acquired

from the boring logs in the geotechnical report and based off the flowchart below.

Schemertmann's method was used to calculate the settlement. The SPT tests used a rod sampler

that was driven into the soil using a safety hammer. The number of blows taken to drive the

hammer 6-24 inches' was recorded at specified increments of soil layers. The summation of the

number of blows for the last twelve inches were calculated to acquire the N, N_{60}, and $(N_1)_{60}$.

These N-values were used to determine the equivalent modulus of elasticity of the soil (Es). The

settlement also depends on a strain influence factor (I_ϵ), which was based on bearing pressure (q'_t), vertical effective stress (σ'_{zD}), and initial vertical effective stress (σ'_{zp}). Schmertmann's method needed correction factors for embedment depth, secondary creep, and the shape of the footing ($C_1, C_2, and\ C_3$ respectively). C_1 was calculated based on vertical effective stress of the embedment depth, C_2 was based on the time span of fifty years, and C_3 was based on the length of the retaining wall footing. The settlement was then calculated using the following equation: $\delta = C_1 C_2 C_3 (q - \sigma'_{zD}) \Sigma \frac{I_\epsilon H}{E_s}$.

3.3 MSE Wall Design

The design of the MSE wall began with establishing the necessary requirements and parameters such as design methods, guidelines, soil information, and load combinations. Along with these, the geometry of the wall and reinforcement were arbitrarily chosen to calculate external and internal stability. The loads were then calculated based on the previous information and guidelines. The flowchart below describes the steps that were taken to design the MSE wall. The design of the MSE wall was referenced from the *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Volume I* published by the U.S Department of Transportation Federal Highway Administration (2009). This document is in accordance with the provisions of the 2007 AASHTO LRFD Specifications 4th Edition.
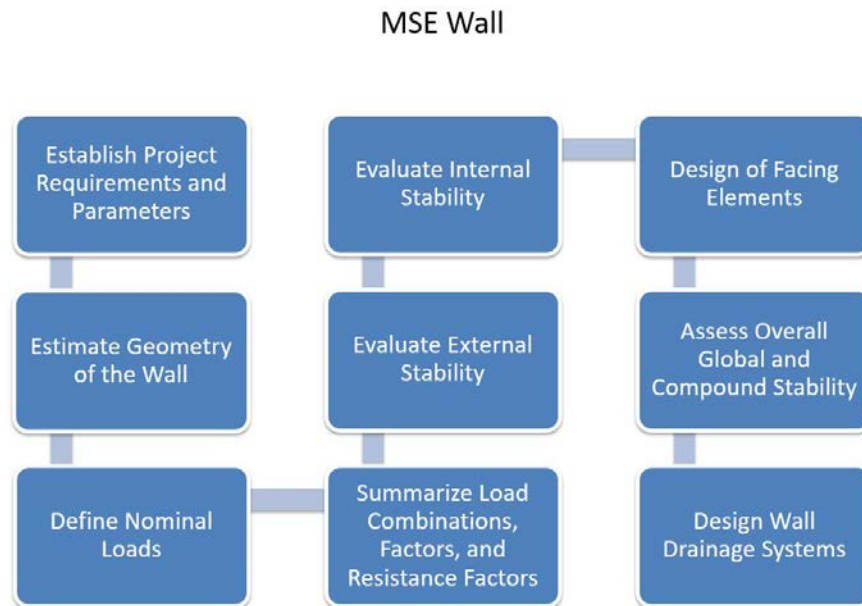
Figure 34: MSE Wall Flowchart

3.3.1 External Stability for MSE Wall

The MSE sliding flowchart below provides the steps that were taken to ensure sufficient external stability. The sliding failure mode was designed to provide sufficient resistance against all lateral loads, shown in Figure 36. The resistance for the MSE wall was calculated by multiplying the weight of the soil by the minimum coefficient of soil friction based on the soil's frictional angle since it is a shear strength parameter. The reinforced soil aids in resisting the lateral forces applied to the wall. The ratio of resistance to the resultant lateral force, or the sliding factor of safety, needed to be greater than 1.5. Depending on the origin of the load, a resistance factor was applied per Table 11.5.6-1 (AASHTO, 2007).

External Stability Flow Chart
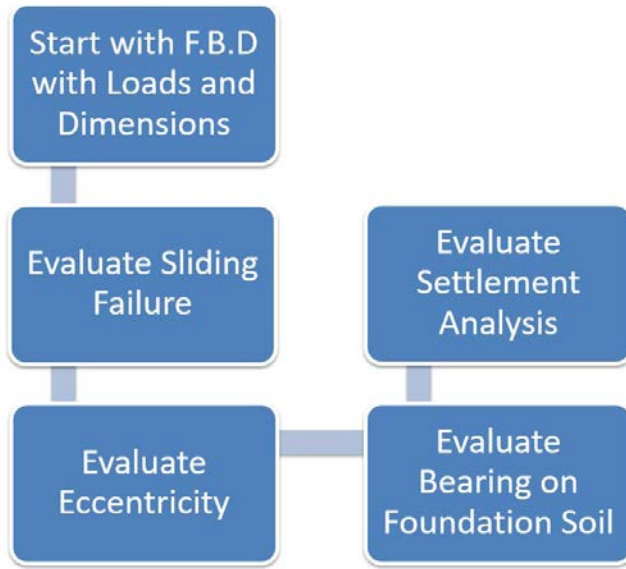
Figure 35: MSE Wall External Stability Flowchart
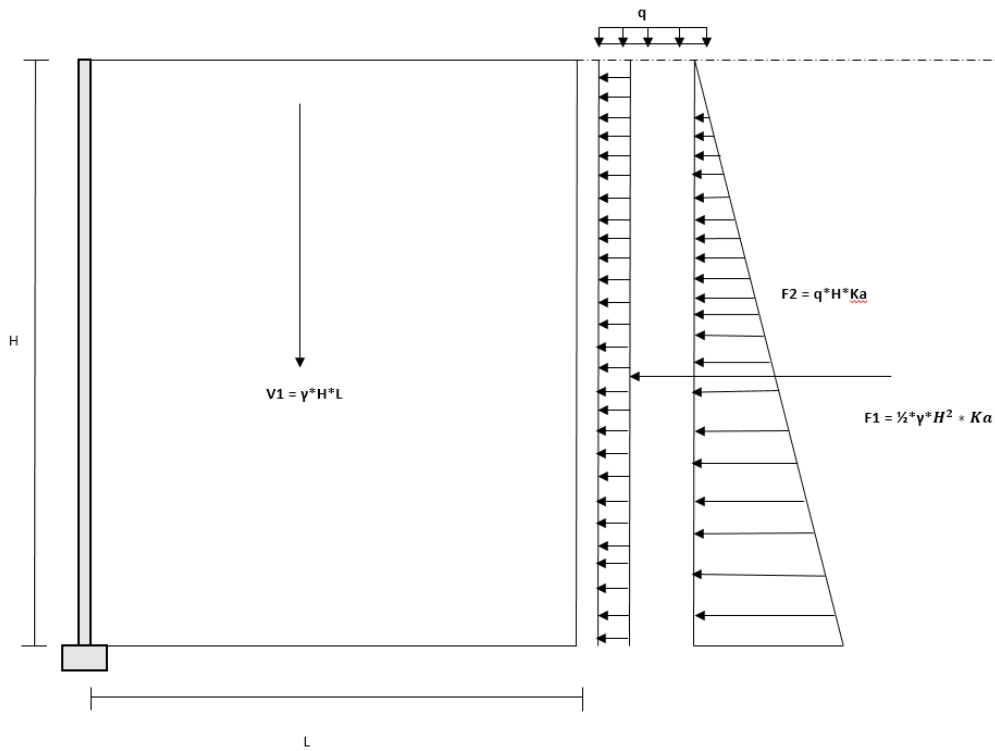
Figure 36: Forces Acting on MSE Wall (FHWA, 2007)

The eccentricity, or overturning failure, was then designed for based on the sum of the moments and vertical weights. The ratio of the sum of the driving and resisting moments to the weight of the soil needed to be within the middle one-half of the width of the base.

The uniform Meyerhof-type (U.S. Department of Transportation Federal Highway Administration, 2007) distribution was used to resist bearing resistance failure. General shear was calculated, which should not exceed the foundation bearing capacity. Eccentricity was calculated including the bearing capacity of the foundation soil, which was used in solving for the induced bearing resistance. Terzaghi's method was used to compute the nominal, or ultimate, bearing resistance of the soil. The embedment term Nq, was neglected when calculating the ultimate bearing capacity because it is not typically used in MSE wall design. The ultimate bearing pressure was then multiplied by a resistance factor to compute the factored bearing capacity. The factored bearing resistance must be greater than or equal to the induced bearing pressure.

3.3.2 Internal Stability for MSE Wall

There are two ways the internal stability of an MSE wall can fail, both caused by large tensile forces. The first internal failure is known as reinforcement elongation or breakage and the other is called pullout failure. The elongation failure is when the inclusions undergo large tensile forces which cause excessive elongation or breakage. The pullout failure is caused by tensile forces greater than pullout resistance, causing excessive wall movement (U.S. Department of Transportation Federal Highway Administration, 2007). The internal stability was designed by determining the maximum tension stresses for each layer of reinforcement, then checking the

resistance of the slip surface and pullout capacity. The flowchart below shows the sequence of steps taken to design the internal stability of the MSE wall.

An extensible (geosynthetic) soil reinforcement was considered for the MSE wall design. The vertical spacing of the reinforcement was arbitrarily chosen, then checked to satisfy the reinforcement resistance and pullout failure requirements.

Internal Stability Flowchart

Select Soil Reinforcement	Calculate Nominal and Factored Pullout Resistance	Check Connection Resistance Requirements at Facing
Define Critical Failure Surface	Select Grade of Soil Reinforcement and or Number of Required Soil Reinforcement per Level	Estimate Lateral Wall Movements
Define Unfactored Loads	Calculate Nominal and Factored Long-Term Tensile Resistance of Soil Reinforcements	Check Vertical Movements and Compression Pads
Establish Vertical Layout of Soil Reinforcement	Calculate Factored Horizontal Stress and Maximum Tension at Each Reinforcement Level	

Figure 37: MSE Wall Internal Stability Flowchart

3.3.3 Final Design Criteria for MSE Wall

The facing elements for the MSE wall must be designed in order to resist lateral forces. These facings were to be flexible and consist of concrete, steel, or timber. Overall/global and compound stability was assessed for potential failure modes behind the reinforcement cross section. The factors of safety were checked for satisfactory values to ensure a stable and

sufficient MSE wall. The final design criteria were developing wall drainage systems for subsurface drainage, surface water runoff, and scour.

3.4 Bridge Design

The second option identified was to construct a bridge along the length of the collapsed road. This bridge combats the erosion by removing some soil and reinforcing the areas around it with wing walls and abutments. The options for the bridge included various superstructures and a set substructure. The design options explored were a steel girder and slab deck, a composite deck superstructure, and a prestressed girder and cast in place deck. The bridge was located along a similar area as the retaining wall option. The locations are shown below.

3.4.2 Bridge Loading Environment

The *2012 AASHTO LRFD Design Specifications* loading traffic scheme HL-93 from section 3 of the AASHTO manual was used to predict and produce the vehicular stresses imposed on this road. This loading is identified in Figure 40 below. It is outlined as a truck with three axle loads: one of 8 kips and then two at 32 kips spaced at 14 feet and then anywhere from 14 feet to 30 feet respectively. The HL-93 traverses the bridge laterally as a live load; the bridge span is shown below in Figure 38. A plan view of the bridge is also pictured below, in Figure 39. The plan view outlines the barriers and lanes are shown in solid lines and the wing walls are shown in dotted lines.
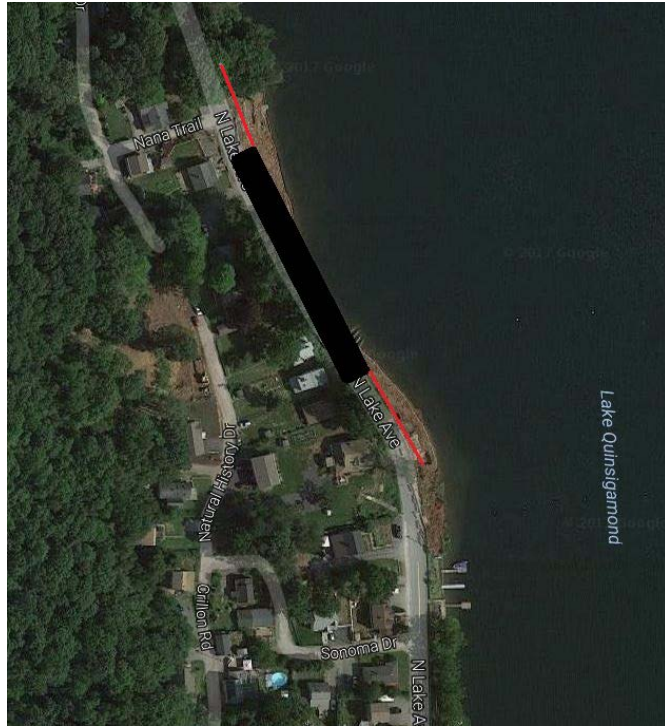
Figure 38: Plan View of Bridge on-site
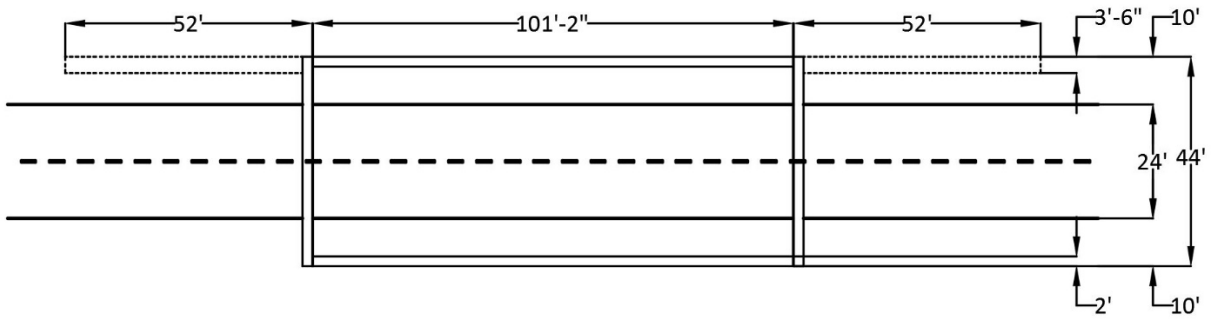
Figure 39: Plan View Schematic of Bridge
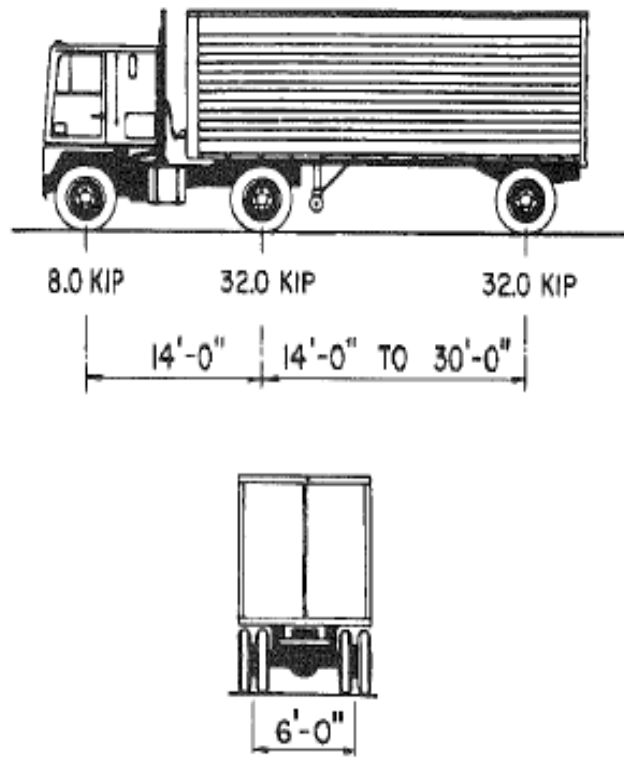
Figure 40: HL-93 Design Truck (AASHTO LRFD)

The major loading cases relevant to this bridge were the dead load and the live load. The dead load came directly from the beams, the slab, an allowance for utilities, and the parapet. The live load, depending on the case, included the HL-93 lane loading and the construction and wet concrete in the case of a composite slab with unshored construction. Each section of the bridge carried different loading. For example, both distributed and concentrated forces were considered for the beam and deck; the abutment and pier were subjected to concentrated reaction forces from the bridge superstructure. All the designs were evaluated with one and two lane loading. One-lane loading referred to only one truck on the bridge. Two-lane loading referred to two trucks placed next to each other on the bridge, creating a more significant negative moment around the single pier. RISA was used to test each of the scenarios. The difference in moments,

positive or negative, and the magnitude affected the calculations for the bridge design. Once the loads and moments were calculated the distribution factors for the superstructure were found. These are simply a way to quantify how much of the load each girder will be assuming. These are calculated via the FHWA manual and the spacing of the beams. After these factors were calculated they were used to adjust the loading values across each beam in order to more accurately reflect the loading per beams.

3.5 Superstructure

3.5.1 Simple Versus Continuous Span

In considering span types for the superstructure design, it was important to check whether or not it would make greater sense to use two simple spans of 50 feet to equal the proposed 100 feet span, or to use a single, continuous span to satisfy the problem. In order to determine which would be more effective in resisting the AASHTO loading scenarios for bridges, two examples were set in RISA 2D with virtually identical loading structures to determine which, in basic terms, provided for a lesser moment under the same load.
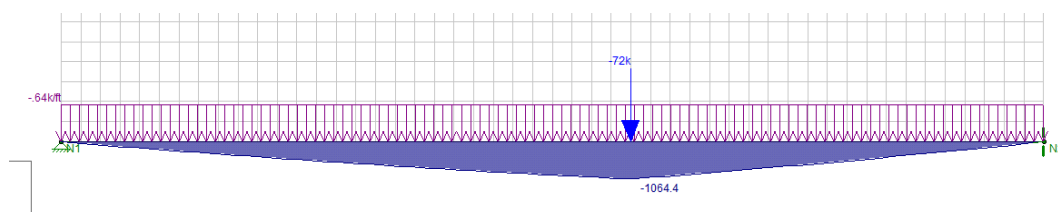
Figure 41: Simple Span Loading

The first span was set as shown in Figure 41, with the design truck and standard distributed live load on display. The figure signifies the maximum moment encountered by the

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simple span beam. Equivalent placement was established on a continuous span beam as well, as shown in Figure 43.
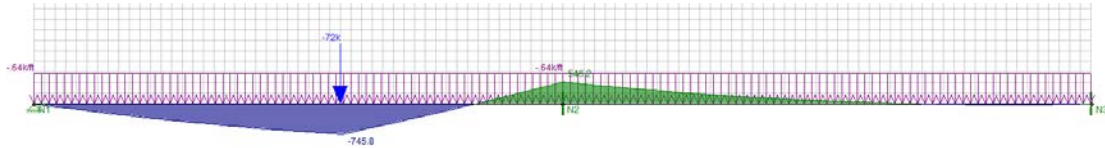
Figure 42: Continuous Span Loading

Although the loading structure is somewhat basic and incomplete compared to a complete model of the bridge, it does serve as a reliable approximation for the purposes of determining a simple span versus continuous span bridge. The continuous span configuration results in a lower maximum positive moment by a significant amount, which was adequate justification for moving forward with a Continuous Span Bridge Design.
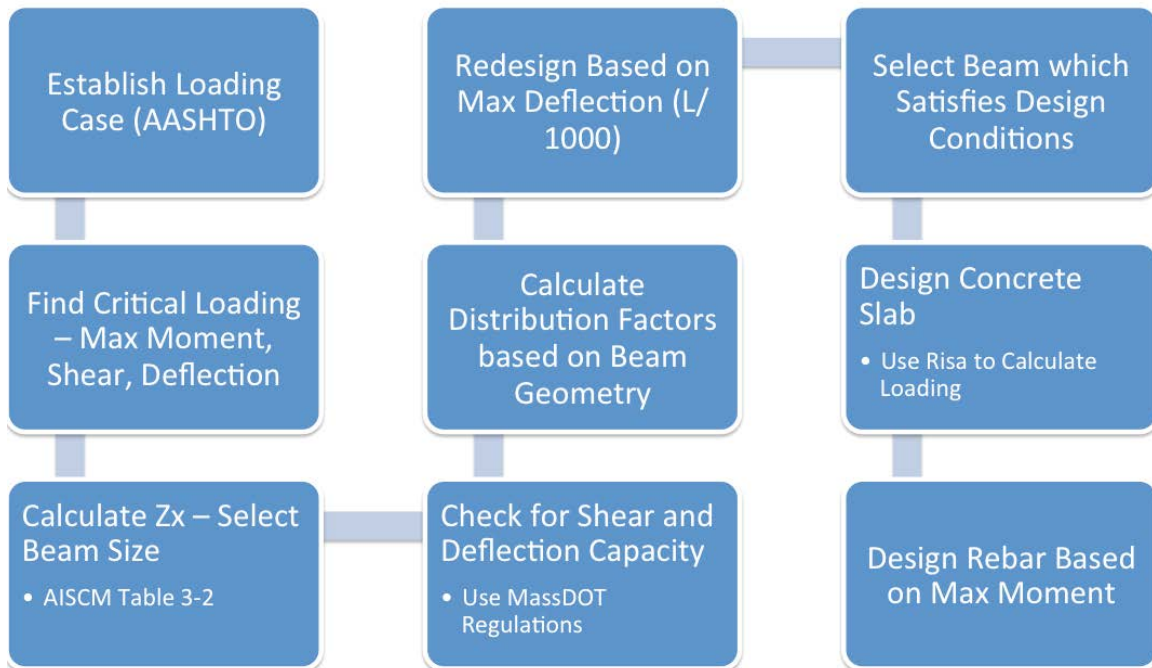
Figure 43: Steel Superstructure Flowchart

3.5.2 Critical Moment Value and Final Non-Composite Girder Selection

In order to select a beam size for the preliminary non-composite girder and superstructure design, the problem was approached with a strategy to find at which point along the span that the HL-93 design truck, represented by a 72 kip concentrated load, would generate the highest moment value, and design from there.

Essentially, the method consisted of repeated analyses of the continuous span beam with the design truck placed at different points along the span. This was done to adequately simulate a moving load along the span in order to pinpoint the critical moment through a trial-and-error approach.
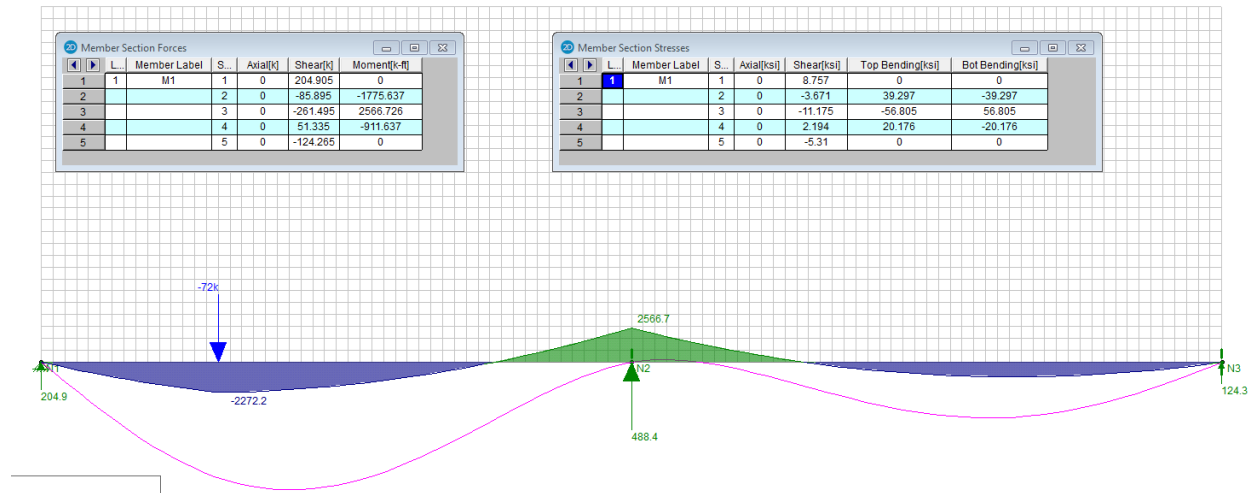
Figure 44: Sample Loading 1
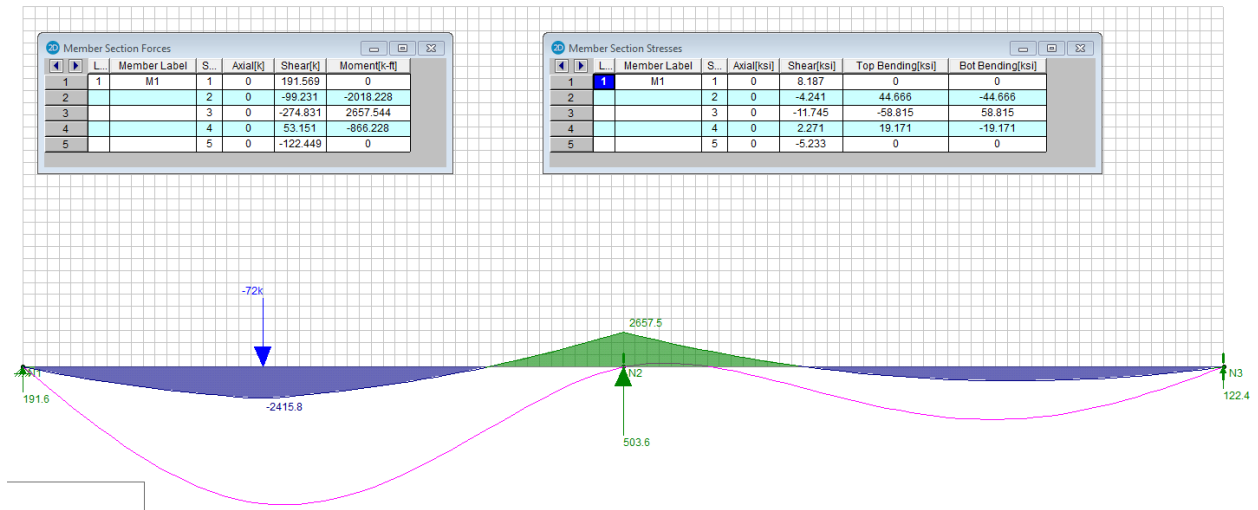
Figure 45: Sample Loading 2
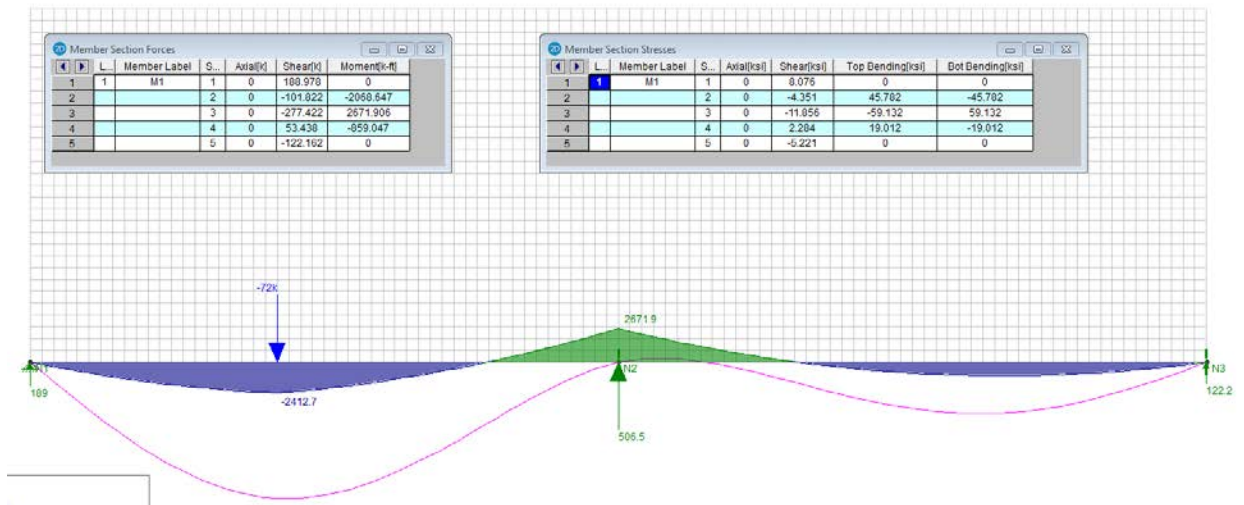
Figure 46: Sample Loading 3
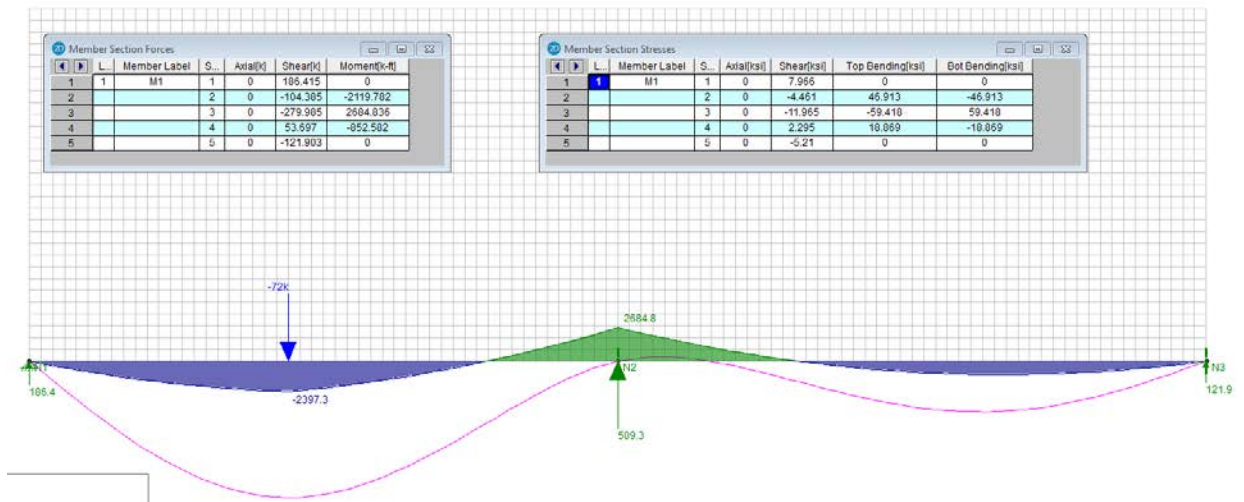
Figure 47: Sample Loading 4
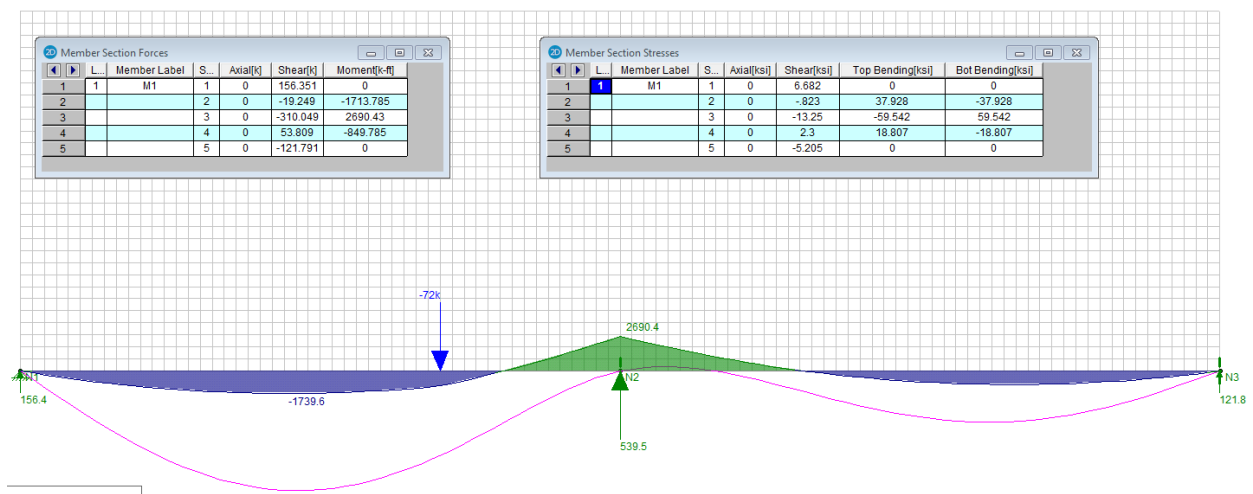
Figure 48: Sample Loading 5

As shown in Figures 45-50, or samples of analyses performed, the maximum moment varies greatly depending upon the placement of the concentrated load, or the representation of the design truck on the span.

Along with the actual RISA solution to the critical moment loading scenario in question, Figure 49 shows that at a distance of 29 feet from the pin joint along the span, the bridge enters a maximum moment state (static determination).
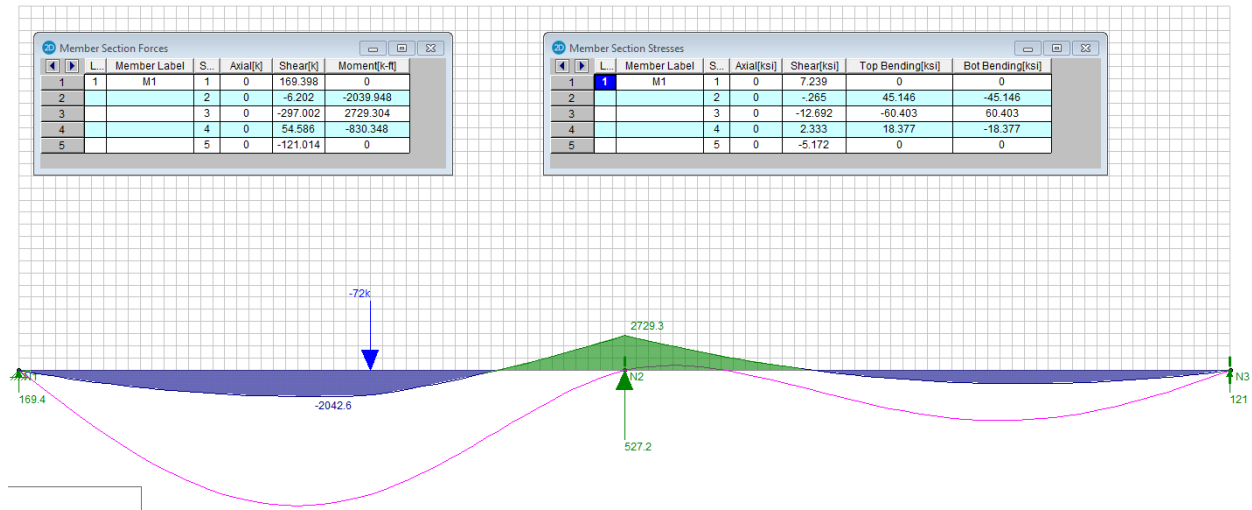
Figure 49: RISA Image of Critical Moment Loading

With the critical moment value determined, selecting a beam for the design was now possible. The required Z_x value was found based on the moment, and, assuming a compact section, a preliminary beam was chosen based on strength. After choosing and checking for deflection limits through subsequent trials, a W40x215 beam was chosen as it satisfied the maximum deflection limit of L/1000 as prescribed by AASHTO.

3.5.3 Slab Design

In order to model appropriately the loads that a concrete slab will undergo as it gathers and transfers loads to the bridge superstructure, a section taken along the longitudinal axis of the bridge was input into RISA software as a beam to approximate the section of the slab. The purpose of this test was to determine the horizontal top and bottom rebar that the slab would be needed. Below is the initial test:
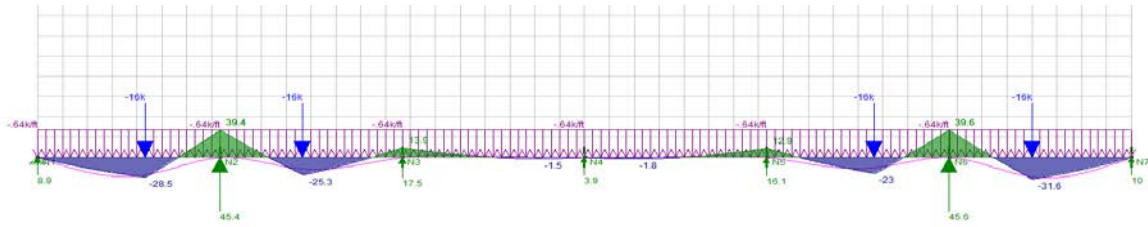
Figure 50: Basic Slab Loading Approximation

The logic in setting up the loading situation was directed at two AASHTO HL-93 Design Trucks driving in opposite directions in the greatest divergence from the girder locations running along the span of the beam underneath the slab. This means that the six foot width of the trucks' axles have their midpoints directly above a girder location in order to produce a critical moment, in this case, negative.
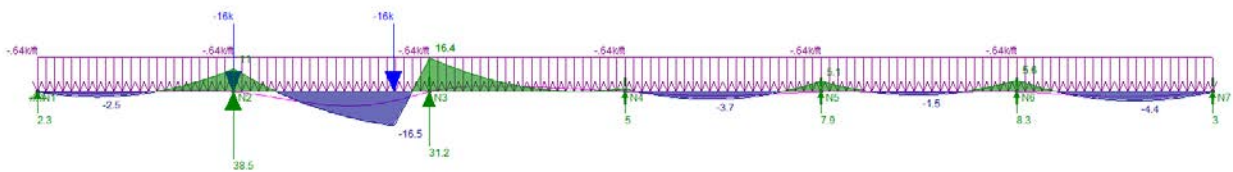
Figure 51: Joint Load - Affirming Non-Critical Positive Moment

As seen in Figure 52, investigation into a governing positive moment in the spans between the girders shows that it is actually the maximum negative moment that governs the design, and that the moments within the girders spans are non-critical. Having the critical moment determined allowed for the design of the necessary reinforcement bars.

To certify that the logic that produced the critical moment was sound, other loading situations were considered, particularly one with only one lane loaded on the slab, to see whether

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having both lanes loaded would counteract a possible magnification of moment enabled by an

open lane.
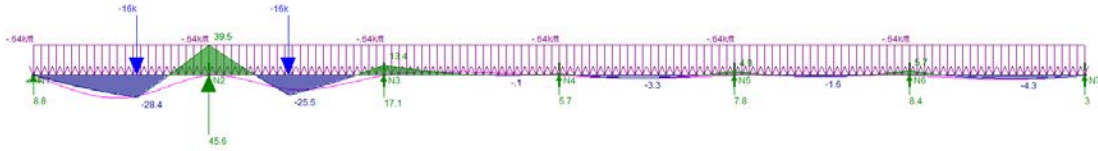
Figure 52: Left Lane Loading

As shown in Figure 53, the moment tapers off dramatically as the point of interest moves

farther away from the truck's location. This analysis demonstrated that no significant moment

presence is induced across the slab in an unmirrored loading scenario. One final test performed

was a shift in the truck's location within the lane itself, shown below.
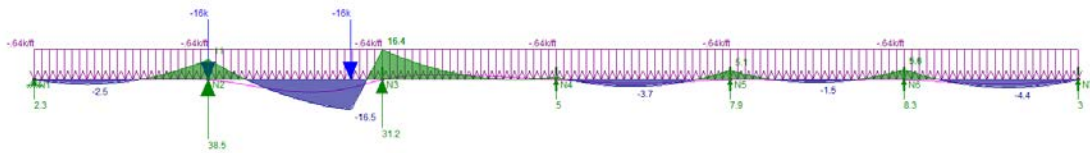
Figure 53: Slab Joint Loading – See above matching Figure

This test was conducted to confirm that shifting the load outside of what was previously

determined to be a critical location resulted in a drop in moment effect on the slab.

Finally, once the maximum moment was determined to be correct, the following

equations were used to calculate the necessary rebar:

$$A_s = M_u/\Phi f_y(d-a/2)$$

$$a = A_sf_y/.85f'_cb$$

where M_u is moment, Φ is constant (0.85), f_y is the yield strength of steel, d is the length of the

far end of concrete to the center of the innermost rebar placement, a is the size of the Whitney

block, f'c is the compressive strength of concrete, and b is the width of the base of concrete.

Top rebar consisted of 2#8 bars and bottom rebar consisted of 2#6 bars, spaced evenly

throughout the entire width of the slab. Longitudinal rebar for temperature and shrinkage

reinforcement, according to ACI 318-02, requires a ratio of reinforcement area to gross area of

concrete of 0.0018, results in a required A_s of 0.1728 in^2. Therefore, 2#3 bars spaced every foot

longitudinally will suffice for this requirement.

3.5.5 Composite Superstructure

After the option of a steel girder with concrete slab was explored, another option

designed was steel girders with a composite slab. Composite girder-and-slab systems are a better

non-composite construction because their composite nature allows for a larger moment of inertia

and therefore a larger resistance to moment and deflection. The minimum required slab thickness

according to MassDOT and AASHTO is 8", so that was the slab thickness considered. The flowchart below, Figure 55, outlines the steps taken to design a composite bridge deck.
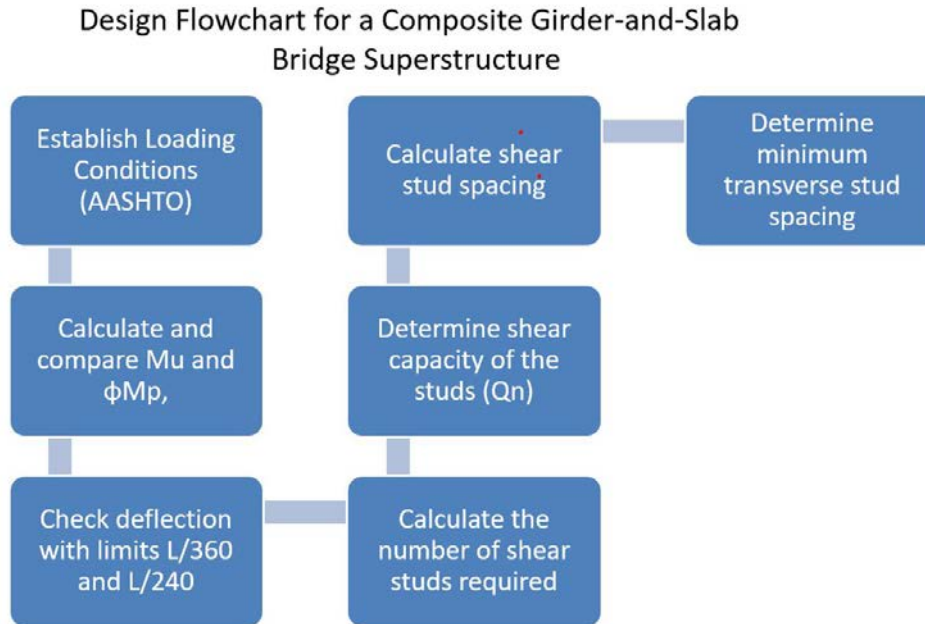
Figure 54: Flowchart of steps to design Composite Beam Section

The first step listed above was to establish the loading conditions. Unshored construction was considered, so there are loads for the slab, construction, and the dead weight of girder itself. During unshored construction there were two major points in time to design for. One being during construction, when the slab was being poured on the frame and the other being during service after the slab cured. The loading is outlined in the table below, which totals 1857.7 plf.

Table 4 – Loading for Composite Deck Design

Load	Load Type	Load Value	Load Factor	Factored Load
Girder	Dead	167	1.2	200.4 plf
Wet Concrete	Live	100 psf	1.6	1240
Metal Decking	Dead	3 psf	1.2	3.6 psf
Maintenance	Live	25 psf	1.6	40 psf
Total				1857.7

The next step was to calculate the moments and M_u and $\emptyset M_n$. The moment for the factored loading in the table above and expresses the loading during construction of the beam. Table 3-19 for AISC was used to calculate the $\emptyset M_n$ for the composite structure based on the a value and the Y_2. The moment with the reduction factor phi is the design capacity moment. If the value Mu is greater than the value, then a new beam needs to be chosen. After a beam was chosen that can support the unshored construction weight of the composite slab according to the moment, the deflection was checked. MassDOT section 3.5.6.1 states the maximum deflection is limited to L/1000 , and the calculations are done according to AASHTO specifications. The value of L/1000 was used for both construction and in service loading. This permits a maximum deflection of 0.6 inches. The equations are stated below. The moment of inertia for the composite slab, I_{LB}, was calculated using the Y_2 from above and AISC Table 3-20. The unshored deflection, using the equation below, came to be .586 inches. The load for deflection at service is important because it is unfactored and includes the live load and the distribution factor because it is unlikely the entire live load will be in effect on any one girder.

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

The next step in the design is to calculate the shear stud capacity. Q_n or the shear capacity needs to be calculated and is the smaller of the two values below, to be consistent with the FHWA example and the AASHTO Specification S6.10.7.4.4c.

Shear Stud Capacity

$$Q_n = .5A_{sc}\sqrt{f'_c E_c} \le A_{sc}F_u$$

Using this equation from MassDOT and AASHTO manuals, Q_n was found to be 21.55 kips, which ended up being the ultimate strength multiplied by the area of the steel and reduction factors. The capacity is used to calculate the number of studs needed. Which is the amount of shear transfer compared to the shear capacity, as seen in the equation below. The number of studs was calculated to be 115 per half the span. After this was found the spacing needed to be checked comparing the length over the total amount of stud spaces which is one more than the number of studs for the whole span. The shear stud spacing was 5.19", which falls between the minimum and maximum according to AISC, which are 4.5" and 36" respectively.

Number of Shear Studs in L/2

$$N = \frac{A_s F_y}{Q_n}$$

Shear Stud Spacing

$$S = \frac{L(12")}{2N+1}$$

AISC Shear Stud Spacing Limits

$$Miniumum = 6d$$

$$Maximum \le 8t_s \le 36" \, Maximum$$

3.5.4 Prestressed-Precast Concrete Girder Design

As an alternative to a continuous span bridge superstructure with steel girders, a prestressed concrete option was explored. A prestressed concrete superstructure, as well as a steel superstructure, was commonly used in bridges. The PCI and AASHTO handbooks both identify it as a viable option. As with the steel girder superstructure, the first step was to choose a shape for the beam, as seen in the flowchart below.
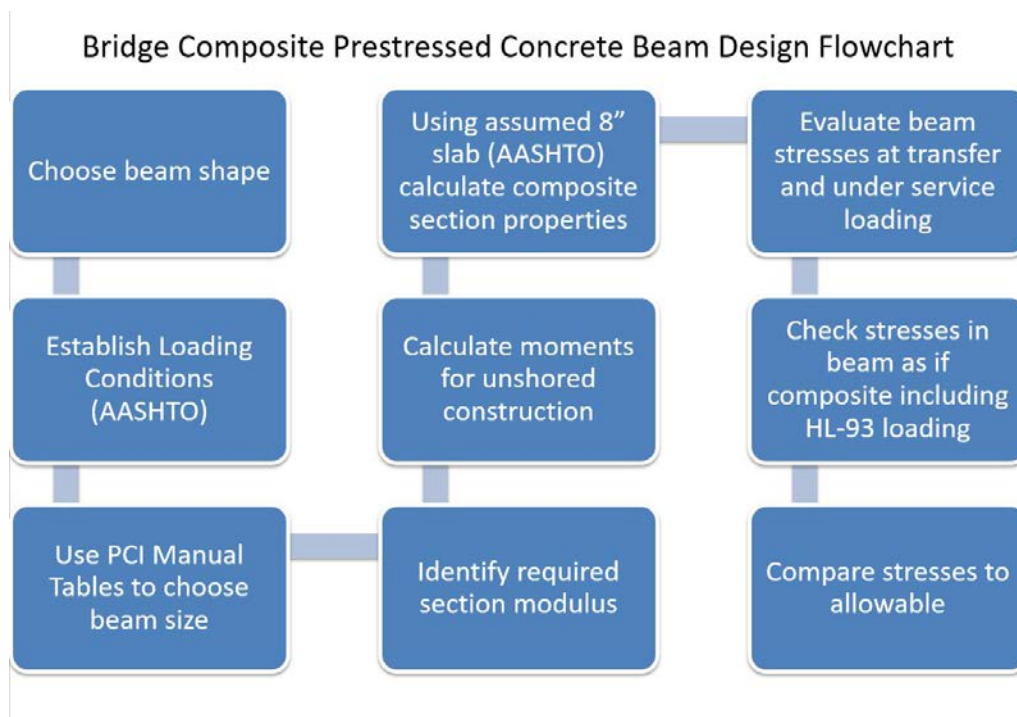
Figure 55: Flowchart to design Prestressed Beam

A double-tee beam was chosen because this shape is best for a shorter span; because it is wide with a reasonable depth and is often used in parking garages. The alternatives, a box girder or an I-shaped beam, were determined to not be the best choice because a box girder, although it could accommodate a large width, would need an equal depth and often covers spans upwards of

100 feet. An I-girder of the same depth would have a smaller moment of inertia and section moduli, and wouldn't be as effective resisting moment as a double tee.

The PCI handbook has tables that list standard double tee sizes with section properties and prestressing steel arrangements, as a function of the to the span and maximum loading. The span used for the prestressed beams was still 50 feet, to stay consistent in comparing the design with the steel option. Since the span and shape were already established the loading conditions were evaluated as the next step in the flowchart. The beam will be composite with the slab and therefore unshored construction loading was accounted for. This includes the wet concrete and live load for equipment and personnel needed to place the concrete slab. The loads are outlined in the table below.

Table 5 - Preliminary Beam Choice Live Loading for Unshored Construction

Load	Load Type	Load Value	Load Factor	Factored Load
Wet Concrete	Live	100 psf	1.6	160 psf
Construction	Live	25 psf	1.6	40 psf
Total	Live	125 psf	1.6	200 psf

The wet concrete load is simply a normal weight 150 pcf concrete multiplied by the 8" depth of the slab. The live load for construction is for personnel that could be on the beam during unshored construction. Once this load was calculated the tables in chapter three of the PCI handbook were used to identify an appropriate double-tee beam. When using the PCI tables the dead load of the beam is already included in the figure below presents a sample beam, and the paragraph below outlines how the properties are read from the table.

8'-0" in. × 32"
Normalweight Concrete

Section Properties

	No Topping	2 in. Topping
A =	567 in.²	–
I =	55,464 in.⁴	71,886 in.⁴
y_b =	21.21 in.	23.66 in.
y_t =	10.79 in.	10.34 in.
S_b =	2615 in.³	3038 in.³
S_t =	5140 in.³	6952 in.³
wt =	591 lb/ft	791 lb/ft
DL =	74 lb/ft²	99 lb/ft²
V/S =	1.79 in.	

f_c' = 5000 psi

f_{pu} = 270,000 psi

½-in.-diameter regular strand
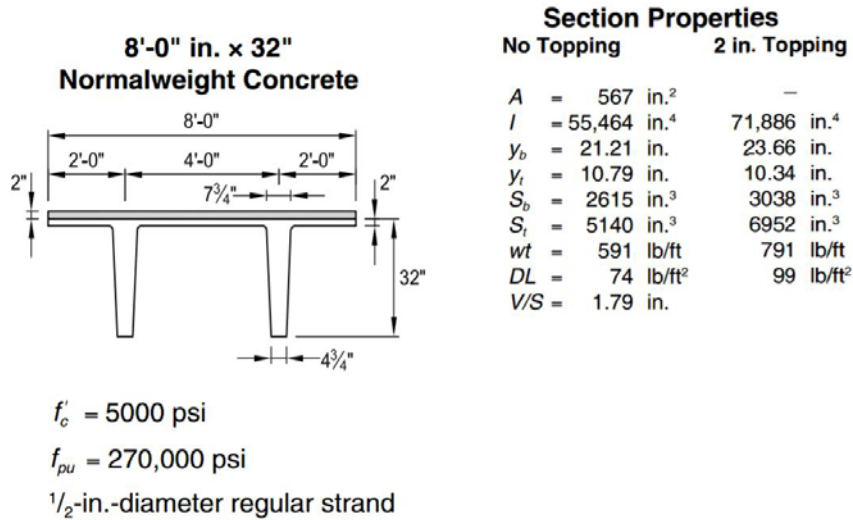
8DT32

Figure 56: Double Tee 8DT32 Section Properties (PCI Manual)

To interpret the designations 8DT32: the "8" stands for 8 feet, which is the width of the beam, "DT" stands for double tee, and "32" refers to the depth of 32 inches. The prestress arrangement of 148-S, mentioned in the PCI manual, means that there are 14 strands with a diameter of 8/16 of an inch and they are all straight, with one eccentricity, and stretched across the beam.

The next step in the flowchart is to determine the required section moduli based on the equations below (Nawy, 2010). The equations are derived from the total moments divided by the stresses in concrete and steel after losses and reduction factors. This will ensure the minimum required section could withstand the potential overload and understrength conditions of resistance from the materials (Nawy, 2010).

$$S^t \geq \frac{(1-\gamma)M_D + M_S + M_L}{\gamma f_{ti} - f_c}$$

$$S_b \geq \frac{(1-\gamma)M_D + M_S + M_L}{f_t - \gamma f_{ci}}$$

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The moments used in these equations refer to the construction condition per the flowchart. The values for f_t and f_{ti}, and f_c and f_{ci} are the limit stresses calculated using the compressive strength of concrete in the prestressed beam and are outlined in Table 6. Table 7, presents similar limiting stresses values for the prestressing steel. The starting values for f'_c and f_{pu} were established based on the PCI handbook requirements for concrete compressive strength and ultimate steel strength. The f'_c value was calculated based on concrete used for bridges of 5000psi and the f_{pu} for low relaxation steel of 270ksi per the MassDOT Supplemental Specifications Subsection regarding prestressed beams (MassDOT, Supplemental Specifications, M4.03.00, 2015).

Table 6 - Limiting Stress Values for Concrete

f'_c	f'_{ci} (psi)	f_{ci} (psi)	f_c (psi)	f_{ti} (psi)	f_t (psi)
	$0.8f'_c$	$0.55f'_{ci}$	$0.4f_c$	$3sqrt(f_c$	$3sqrt(f_c$
5000 psi	4000	- 2200	-200	212	424

Table 7 - Limiting Stress Values for Prestressing Steel

f_{pu} (psi)	f_{py} (psi)	f_{pi} (psi)	f_{pe} after losses (psi)
	$0.9f_{pu}$	$0.7f_{pu}$	$0.8f_{pi}$
270,000 psi	243,000	189,000	151,200

After the required section moduli were calculated they were compared to the section moduli for the 8DT32 beam. The comparison proved this beam to be sufficient based on these criteria. A prestressed beam and composite slab needs to be evaluated at three points in time: at transfer, during construction, and after it has become composite. Consistent with the flowchart, the next step is to calculate these values.

Transfer Loading Conditions

The loading conditions at transfer is when the concrete has cured at the plant and the tension in the strands is released. The two loads considered were the prestressing force which, is calculated using the eccentricity, neutral axis location, and prestress force in the tendon over the area of the beam. At transfer, the moment from the dead load is included too. The prestressing force acted counter to the dead load so the top fibers should be under tension and the bottom fibers should be under compression. The equations used to find these stresses are given below.

$$f^t = -\frac{P_i}{A_c}\left(1 - \frac{e_c c_t}{r^2}\right) - M_D$$

$$f_b = -\frac{P_i}{A_c}\left(1 + \frac{e_c c_t}{r^2}\right) + M_D$$

Construction Loading Conditions

During construction is once the beam has been installed, the slab is being placed which includes a dead weight of wet concrete and a live load for placing and construction. The moments from these dead and live loads found using RISA needed to be accounted for. The construction live load and the dead load are divided by the section moduli to spread the entire moment out over the beam cross section. The equations used to find these stresses are given below.

$$f^t = -\frac{P_i}{A_c}\left(1 - \frac{e_c c_t}{r^2}\right) - (M_D + M_{LL})/S^t$$

$$f_b = -\frac{P_i}{A_c}\left(1 + \frac{e_c c_t}{r^2}\right) + (M_D + M_{LL})/S_b$$

Service Loading Conditions

Once the slab has cured, the structure will be composite and was evaluated again to include the HL-93 lane loading. This was considered at the service loading condition. The

section characteristics changed and needed to be calculated prior to finding the final stress in the beam. During service the section moduli for the composite structure are used. The composite structure reduces the moment on the bridge significantly because the section moduli are so large (Nawy, 2010). The stresses in the top fiber and the bottom fiber were calculated using the equations below.

$$f^t = -\frac{P_i}{A_c}\left(1 - \frac{e_c c_t}{r^2}\right) - (M_D + M_{HL-93LL})/S^{tc}$$

$$f_b = -\frac{P_i}{A_c}\left(1 + \frac{e_c c_t}{r^2}\right) + (M_D + M_{HL-93LL})/S_{bc}$$

3.5.5 Deck Slab Design

The deck slab design will be completed with reference from an example design flowchart given by FHWA. A different slab will have to be designed for both the steel girder and concrete girder bridges. AASHTO and PCI specifications will be considered for the design of the deck.

3.6 Substructure

3.6.1 Abutments

The abutments were calculated according to the FHWA bridge design example. The first step of the abutment is to choose a type. An integral abutment was chosen because it was simple and commonly used. This type of abutment allows bearings to be used on the top for each girder. The next step was to assume the abutment would be similar to a retaining wall to the effect it would need to abide by the limits for bearing capacity, and factors of safety for overturning and sliding. This assumption was made because in addition to the superstructure loading and live loading for the bridge, the abutment also must withstand the soil loading and surcharge as well. This was to ensure the design chosen could withstand all loading necessary. The bridge

superstructure and traffic load along with the surcharge load would constitute vertical loads

applied to the top of the abutment. The loads are normally in pounds per foot and in this case

they are point loads because they were only contacting the abutment at the bearing. The soil

loading was the lateral loading and referred to as the lateral earth pressure. The load is calculated

as a function of the depth of the abutment in comparison to the soil's unit weight. This would

give the resultant force at ⅓ the height of the wall and from there a moment could be calculated.
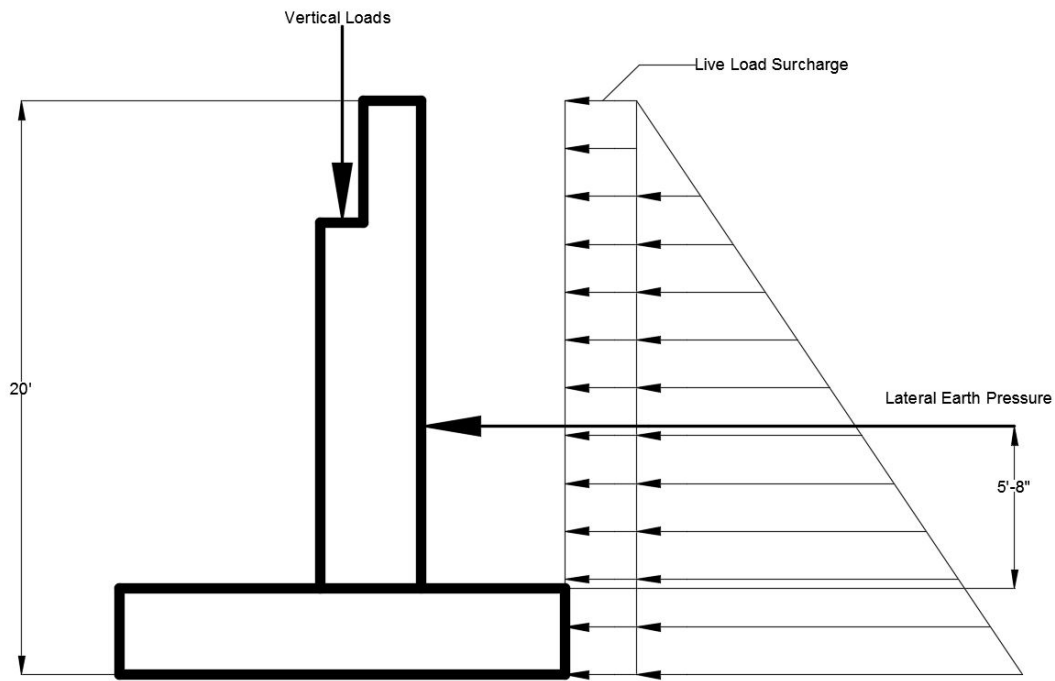
Figure 57 - Abutment with Resultant Forces

The abutment, since it was treated as a retaining wall, follows the same design for

reinforcement. The pier footing design had the similar process to the abutment footing. It

required calculating the load for induced by the pier, and then checking for one and two-way

shear forces. One-way shear required checking the values of φV_n against V_u. V_n is the

summation of shear at the critical surface and shear resistance from reinforcement. For the

preliminary calculations the shear reinforcement was neglected just to test the shear resistance

form the concrete. φV_n is calculated using the compressive strength of the concrete (f'_c), the width of the footing (b_w), and a reduction factor. V_u in contrast was found using the dimensions of the footing, the column width, the effective depth, and the soil bearing pressure (q_u), which was given from the soil report. The preliminary dimensions for the footing are 23 feet by 15.5 feet, this was given by a design example use in the FHWA manual, that has a similar design. These dimensions were tested, and the equations used for V_u, and V_c are shown below.

$$\phi V_n = V_c + V_s$$
$$\phi V_c = \phi 2 \sqrt{f'_c} b_w q_u$$
$$V_u = \left(\frac{l}{2} - \frac{c}{2} - d\right) bq_u$$

It is important to note that φV_n is the capacity for the concrete block, so it needs to be larger than the actual shear in the block, given by V_c. This was just for one-way shear. Two-way shear was calculated using the b_o the column width and the effective depth, and three equations and the minimum value governs the shear capacity in two-way shear. The actual shear calculated was using a reduction factor or 40 for normal weight concrete, and the compressive strength of the concrete.

$$V_c = 4\lambda \sqrt{f'_c}$$
$$V_c = (2 + \frac{4}{\beta})\lambda\sqrt{f'_c}$$
$$V_c = (\frac{\alpha d}{b_o} + 2)\lambda\sqrt{f'_c}$$

The final step was to calculate the shear reinforcement. This was found using the moment from the soil, and the shape of the footing. The equation used is shown below.

$$a = \frac{A_s f_y}{.85 f'_c b}$$

$$\phi M_n = \phi A_s f_y (d - \frac{a}{2})$$

The Whitney block, a, was used to calculate the required steel area. The required area of steel, and the minimum area of steel were calculated using the equations below. The value A_s, the required area of steel, was solved for in the Whitney block equation and substituted into the moment capacity equation. The moment was already known because the capacity was assumed to be equal to M_u, which was calculated using the equation

$$M_u = q_u (\frac{l-c}{2})^2 \left(\frac{b}{2}\right)$$

It was assumed #8 bars were being used. The number of bars was found using the required area of steel and the area per #8 bar. The spacing was simply calculated using the number of bars over the length of the footing.

3.6.2 Wing Wall

The figure in section 3.2 (Figure 20) illustrates the basic rendering of the wing wall with the loads that were taken into consideration for stability analyses. The wing wall was placed perpendicular to the abutment of the bridge. The purpose of the wing wall was to confine the backfill soil behind the abutment provide sufficient stability against the lateral soil pressure and surcharge load due to traffic. The wing wall was designed as a cantilever retaining wall to withstand lateral earth pressure and surcharge loads. The flow charts and steps taken to design the wing wall are the same as for the design of the cantilever retaining wall in section 3.2 of the methodology. The only difference was the assumed dimensions, which were chosen to reflect similar abutment geometry (such as stem height and footing thickness). With the similar dimensions, designing internal stability was essentially using the same steel reinforcement for the wing wall and the abutment.

3.6.3 Piers

The pier is required to provide necessary support in the middle of the bridge's span to allow for a continuous span to be utilized. It connects the middle of the span to the substructure and allows transfer of loading along the middle span to the foundation elements of the bridge. The main pier located in the middle of the bridge's 100ft span was designed according the FHWA AASHTO LRFD Design Example for the design of a Pier for a bridge with a steel girder superstructure. The design was created adhering to the recommended design process shown below:

Pier Design Flowchart
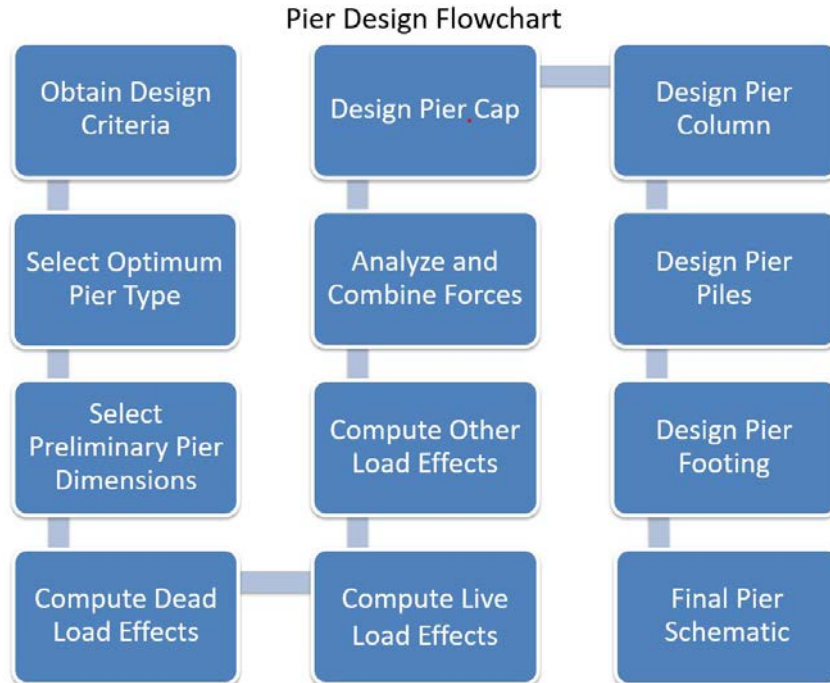
Figure 58: Pier Design Flowchart

Using normal weight concrete, design began with the determination of superstructure depth and concrete cover requirements. With that information, the pier type was chosen. According to the FHWA design example, common pier types used are hammerhead (single column), solid wall type, or bent (multiple column) type. Hammerhead was chosen due to its

typical use in most bridge applications. The preliminary dimensions were determined based on the example's recommendations, which were virtually identical to the proposed bridge's dimensions.

With the dimensions and material properties established, it was necessary and possible to compute all of the load effects. Split into the three different main categories, the load effects represented dead, live, and other loads.
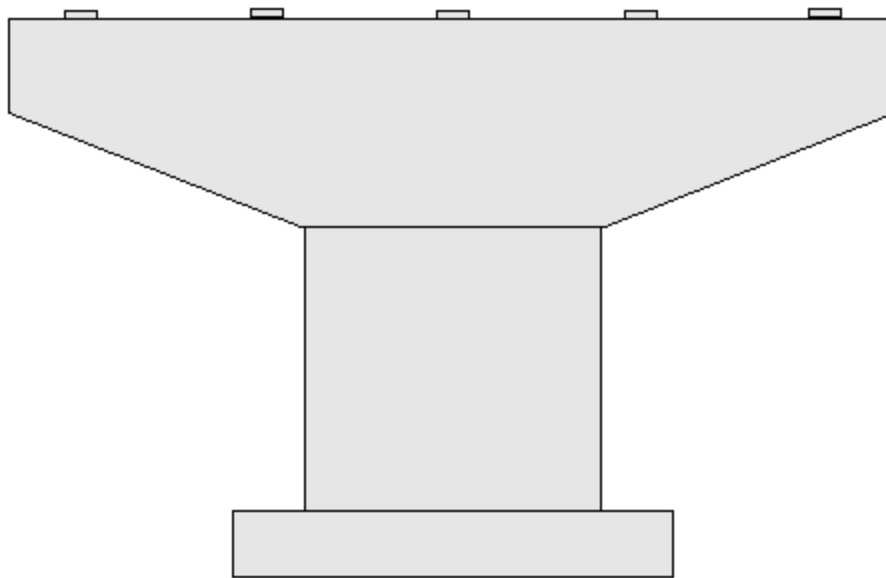
Figure 59: Hammerhead Pier Type

With all loads acting on the pier calculated on account of the dead and live loads of the superstructure and vehicular loading, as well as wind and temperature loading, it was then possible to analyze and combine the force effects in order to determine comprehensive moment, shear, and axial effects on the pier.

Due to the extensive and complicated nature of the example and general acquisition of these values, it was decided that an approximation based on the ratio of the moment strength of the design example compared with the required flexural strength for the proposed bridge and its

loading was the most time sensible method of obtaining the force effects. Note that the previously outlined flowchart is still relevant to pier design, but for the purposes of this project, an effective approximation was utilized to avoid an overbearing scope. Applying this moment ratio to the values determined in the example allowed for the determination of the force effects on the pier cap, column, and pile, shown in the corresponding results section. Knowledge of these values allowed for proceeding with the design of the rebar needed for the pier.

The continuing design for the pier from this point was adapted directly from the FHWA Pier Design Example for the express purpose of simplifying and lessening the extensive scope of the project and to ensure a successful design, if a bit overdesigned. The similarities of this project to the example outlined by the Federal Highway Administration allow for minimal difference in the optimal design for the example and for this project's proposed constraints and recommendations. Calculations were carried out similar to the overall process outlined in the example, with lesser loading approximations based on the relative size of this project's design and superstructure compared to the one utilized in the example. Calculations and checks for the Pier Cap and Pier Column were carried out a reasonable length based off the FHWA example to justify its use in this project. This method overall allowed for the completion of the Pier Cap and Column design.

3.6.4 Foundation Design

The design of the underreamed drilled shaft, seen in Figure 60, began with determining sufficient dimensions and reinforcement based off the axial compressive load provided from the pier analysis. From the soil profile, the greatest depth given governed how deep this foundation was going to be designed. The following equation was used to calculate the minimum shaft

diameter (B): $B = \sqrt{\dfrac{3.86P}{f'_c}}$, P being the axial compressive load. The underreamed diameter

(B_s) was determined by satisfying the ratio $B_s/B < 3$ (Coduto, D. P. 2001).
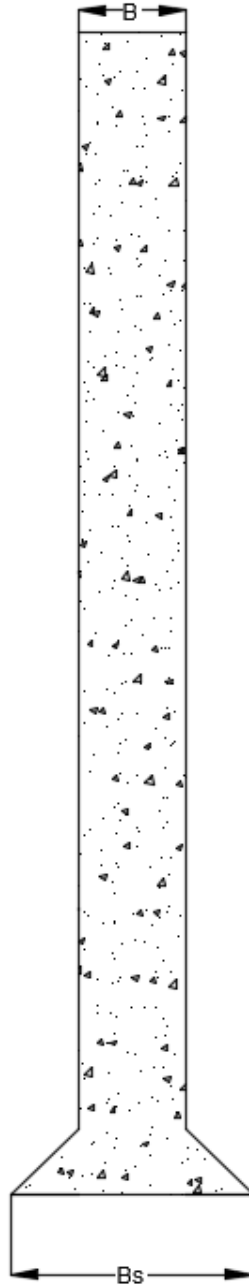
Figure 60: Underreamed Deep Foundation

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The following assumptions in figure 61 below were made in order to calculate the reinforcement. The variable γ was determined based off of figure 61 below (B'= γB), which was used to calculate the steel reinforcement ratio (ρ) through interpolation and figures 12.6 through 12.9 in Coduto, D. P. (2001). With ρ determined, the area of the steel required was calculated, and a number steel rebar was chosen accordingly. The spiral reinforcement was calculated in a similar manner, where ρ was using the following formula $\rho = \dfrac{18\pi A_s}{\pi 18^2 p/4}$, p being the pitch which governs the spacing of the spiral reinforcement on center (Coduto, D. P. 2001). The number bars and spacing on center were then chosen accordingly.
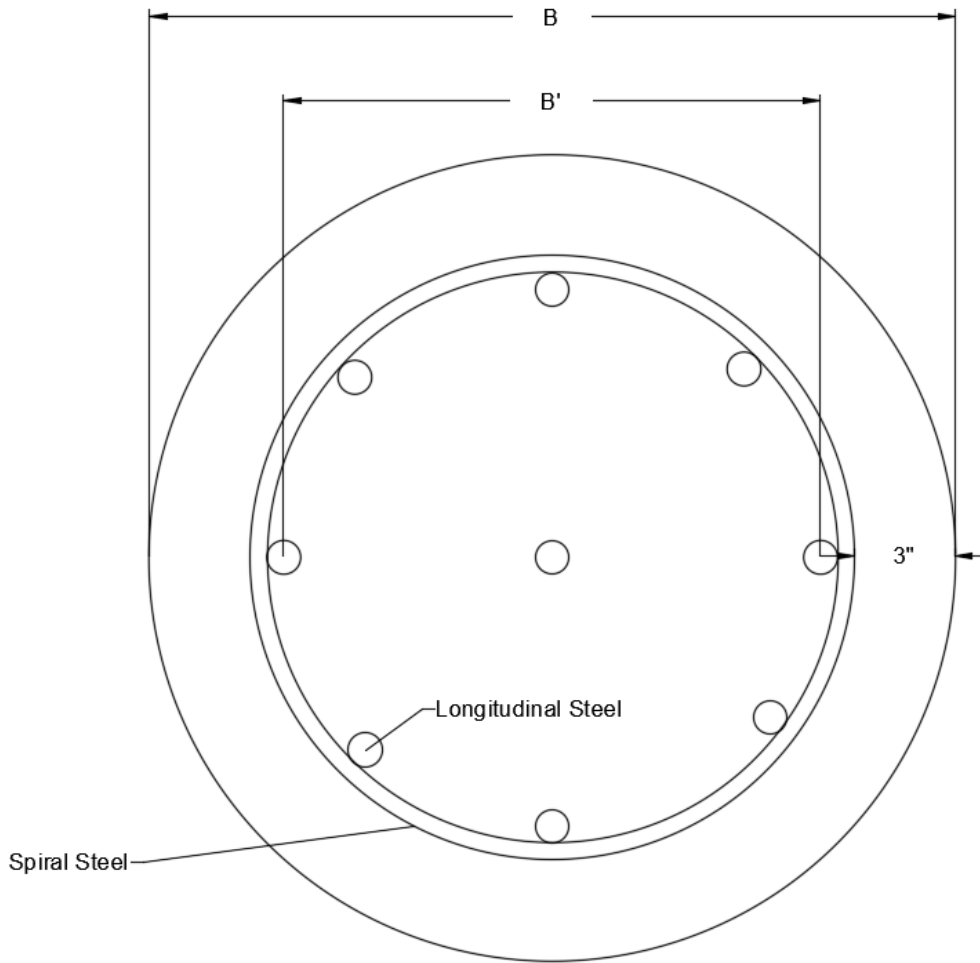
Figure 61: Deep Foundation Reinforcement

After the dimensions and reinforcement were chosen, the net toe bearing (q'_t) and side frictional (f_s) resistance were calculated. The net toe bearing resistance was calculated by multiplying 1200 by the SPT N_{60} value. The side frictional resistance was calculated using the beta method, which was the variable β multiplied by the vertical effective stress (σ'_z). The following equation was used to find β: $\beta = 1.5 - 0.135\sqrt{z}$, z being the depth to midpoint of soil layer. The upward, and downward allowable axial compressive loading was then calculated based on the net toe bearing and side frictional resistance using the following equations:

$$P_{aupward} = \frac{N_u \sigma'_{zD} A_b}{F.O.S.}$$

$$P_{adownward} = \frac{q'_t A_t + \Sigma f_s A_s}{F.O.S.}$$

4.0 Results & Discussion

The following section outlines the final design values found for the retaining wall and bridge options as outlined in the steps in the Methodology. The steps are outlined in the text and in flowcharts for clarity. The design is compared to the MassDOT and AASHTO design guidelines for allowable and regulated design requirements. The following section depicts the viability of these design options is accordance with these guidelines. The following section also discusses and explains the design criteria used to pick a final design option, and how this relates to the capstone design requirement.

4.1 Soil Profile

Excluding the 12 inches of asphalt, the soil profile shows 4 different layers of soil for the location of this site shown in Figure 62. Borings 1, 2, and 4 extended to depths between 32-34 feet. Boring 3 was drilled to a depth of 44 feet, exposing an 8-foot bedrock layer.
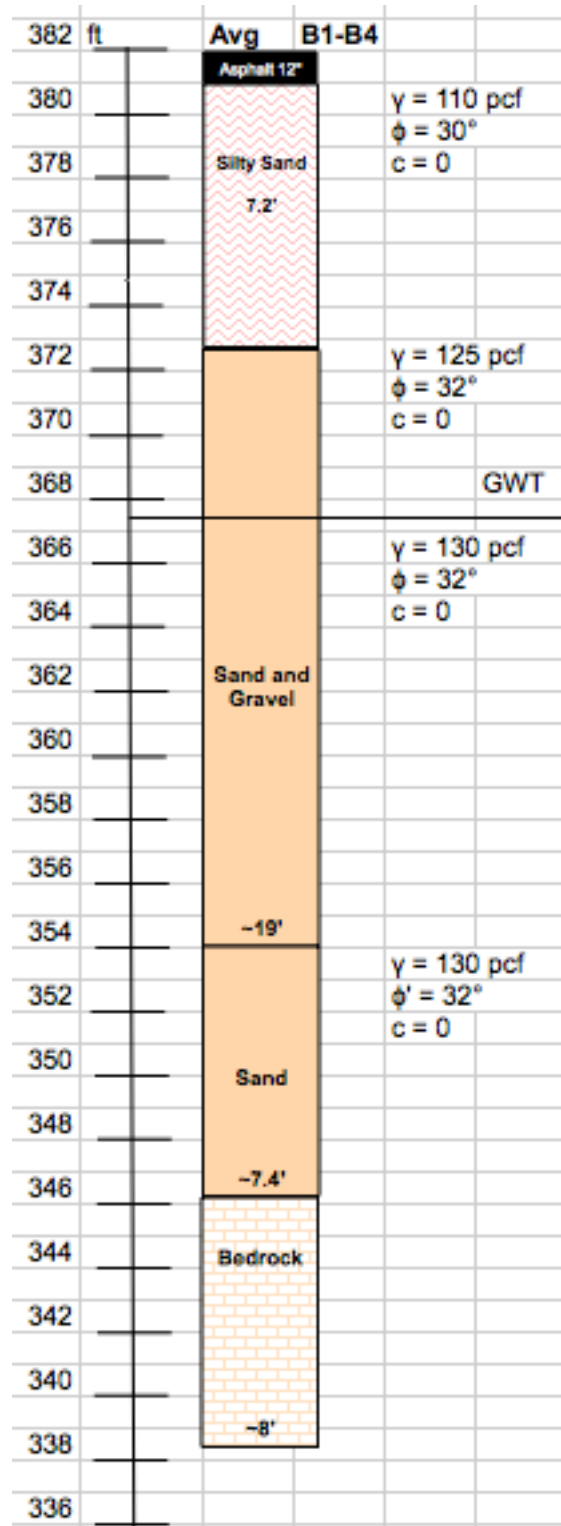
Figure 62: Soil Profile

4.2 Cantilever Retaining Wall

The lateral earth pressure, surcharge load due to traffic, and the weight of the retaining wall and backfill soil were used in this design. The calculations done by hand can be found in Appendix B, along with an excel spreadsheet that was used for iterative calculations. In accordance with sections 3.2.2 to 3.2.4 of the methodology, three factors of safety were calculated to provide sufficient resistance against the three corresponding failure modes, which are provided below in Table 8. The dimensions in Figure 63 below were used when calculating these factors of safety.
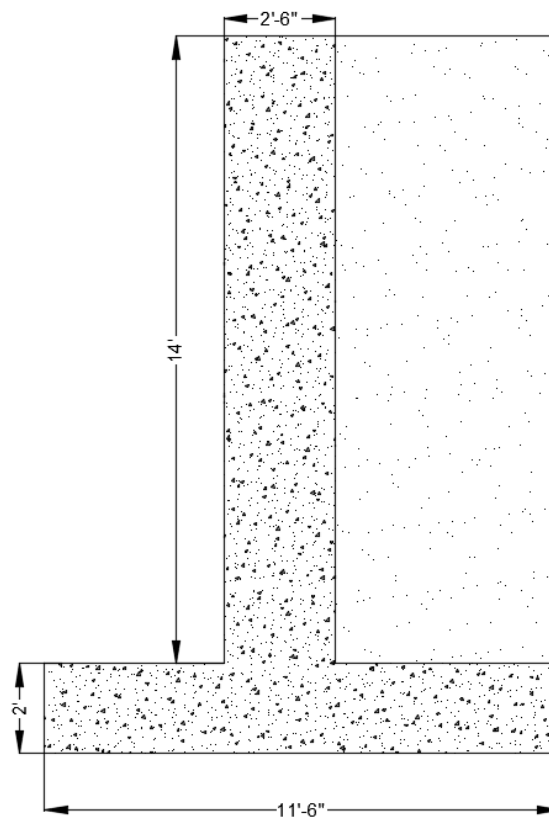
Figure 63: Final Dimensioned Cantilever Retaining Wall

Table 8 - Required and Calculated Factors of Safety for the Cantilever Retaining Wall

	Required	**Calculated**	**Sufficient?**
Sliding Factor of Safety	1.5	2.43	YES
Overturning Factor of Safety	2.0	2.51	YES
Bearing Capacity Factor of Safety	3	9.62	OK

The calculated sliding and overturning factors of safety were not much greater than the required ones, unlike the bearing capacity. The calculated bearing capacity factor of safety is more than three times greater than the required value. The reason this could be so high was because of Terzaghi's bearing capacity method. When calculating the ultimate bearing pressure, the effective frictional angle of 32 degrees resulted in large values for the variables $N_c, N_q, and\ N_\Upsilon$, which governed the calculated ultimate bearing pressure.

The settlement of the cantilever retaining wall was calculated to be about **3.85 inches**. Table 9 below shows the variable values used in calculating the settlement using Schmertmann's method and solving for the correction factors for embedment depth, secondary creep, and the shape of the footing ($C_1, C_2, and\ C_3$). The hand calculations and *Excel* spreadsheet can be found in Appendix B.

Table 9: Settlement Calculations and Variables

Variable	Equation	Value
C_1	$1 - 0.5\left(\dfrac{\sigma'_{zD}}{q - \sigma'_{zD}}\right)$	0.92
C_2	$1 + 0.2log\left(\dfrac{t}{0.1}\right)$	1.54
C_3	$1.03 - 0.03\dfrac{L}{B} \geq 0.73$	0.76
$\Sigma\dfrac{I_\epsilon H}{E_s}$		$9*10^{-5}$

The table and figure below in Table 10 and Figure 64, respectively shows the steel reinforcement required for the final dimensions of the cantilever retaining wall. The calculations were done by hand and can be found in Appendix B.

Table 10 - Cantilever Retaining Wall Reinforcement

	Stem Reinforcement	Footing Reinforcement
Longitudinal	#14 bars at 9-inch spacing	#9 bars at 7-inch spacing
Vertical	11 #9 bars	13 #4 bars
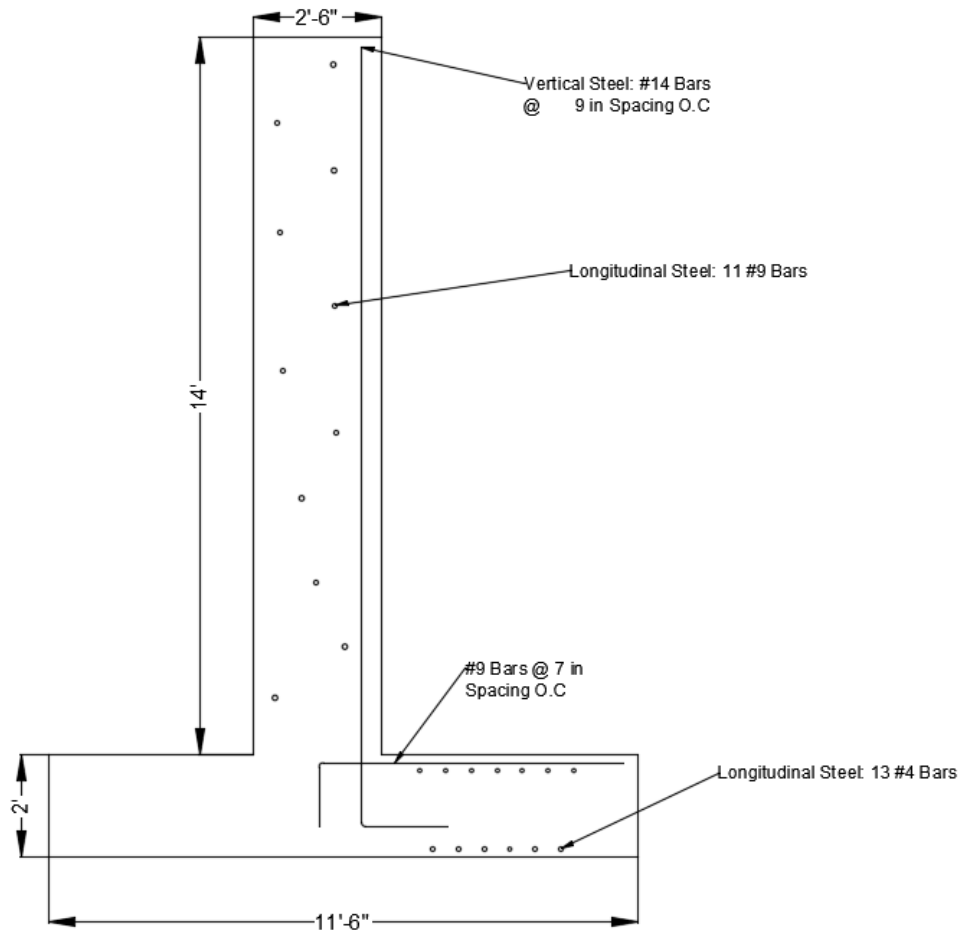
Figure 64: Steel Reinforcement for Cantilever Retaining Wall

A 3-inch concrete cover was used for the steel rebar for both the stem and footing of the retaining wall. The purpose of the steel reinforcement is to resist against shear force. Since the stem and footing concrete are poured separately, there must be a cold joint between them to allow the shear force to pass from the stem to the footing. Weep holes were implemented using 6-inch pipes spaced horizontally every 10 feet of the wall to provide a proper drainage system.

4.2.1 Cantilever Retaining Wall Cost

The estimated cost to construct the cantilever retaining wall was based on cost per cubic yard estimates (International Project Estimating Limited, 2017). The total cost for this design was calculated in two parts: one for the excavation and back fill cost, and another for the construction cost. Table 11 shows the unit costs for man-hours, labor, equipment, and job and permanent materials for the excavation, backfill, and construction.

Table 11 - Unit Costs for Cantilever Retaining Wall Design

	Excavation & Backfill	**Construction**
Labor	6.32 ($/$yd^3$)	160.51 ($/$yd^2$)
Equipment	5.62 ($/$yd^3$)	91.68 ($/$yd^2$)
Job Materials	0.11 ($/$yd^3$)	2.18 ($/$yd^2$)
Permanent Materials	6.17 ($/$yd^3$)	641.51 ($/$yd^2$)

The quantity of the design was determined to be 511.11 cubic yards of volumes for excavation purposes, and 225.83 square yards of surface area for construction purposes. The quantities for excavation, backfill, and construction were multiplied by the unit costs from Table 11 above to produce Table 12 below, and summed to determine the total cost of the project.

Item	Cost
Excavation and Backfill	$9,315
Construction	$202,461
Total (Rounded Up)	$220,000

4.2.2 Cantilever Retaining Wall Schedule

Once the site has been excavated, the construction time for a cantilever retaining wall varies according to the design geometry, and Department of Transportation (DOT) estimations and requirements. The construction of formwork and placement of concrete should take approximately 6-7 weeks, or even up to 8 weeks, including the installation of the steel rebar reinforcement. Backfill soil cannot be placed behind the cantilever retaining wall until the concrete has cured for 28 days to achieve maximum compressive strength. With these given time estimates, the construction and backfill of the cantilever retaining wall was estimated to be 3 months from start to finish.

4.3 MSE Wall

The final dimensions of the MSE include a height of 12 feet and reinforcement length of 10 feet, shown in Figure 65. Extensible (geosynthetic) reinforcement was used in 7 layers at 1 foot spacing for the first and seventh layer, and 2 foot spacing for layers two through six. Masonry block units (MBW) were selected as the facing elements and connected using the friction between the units and reinforcement. ¾ inch gravel was placed in the core of the MBW units to further increase the friction of the connection. A 6-inch wide layer of ¾ inch gravel was

incorporated between the face of the wall and reinforced soil to provide sufficient draining. The

supporting calculations for the results displayed in Table 13 are given in Appendix C.
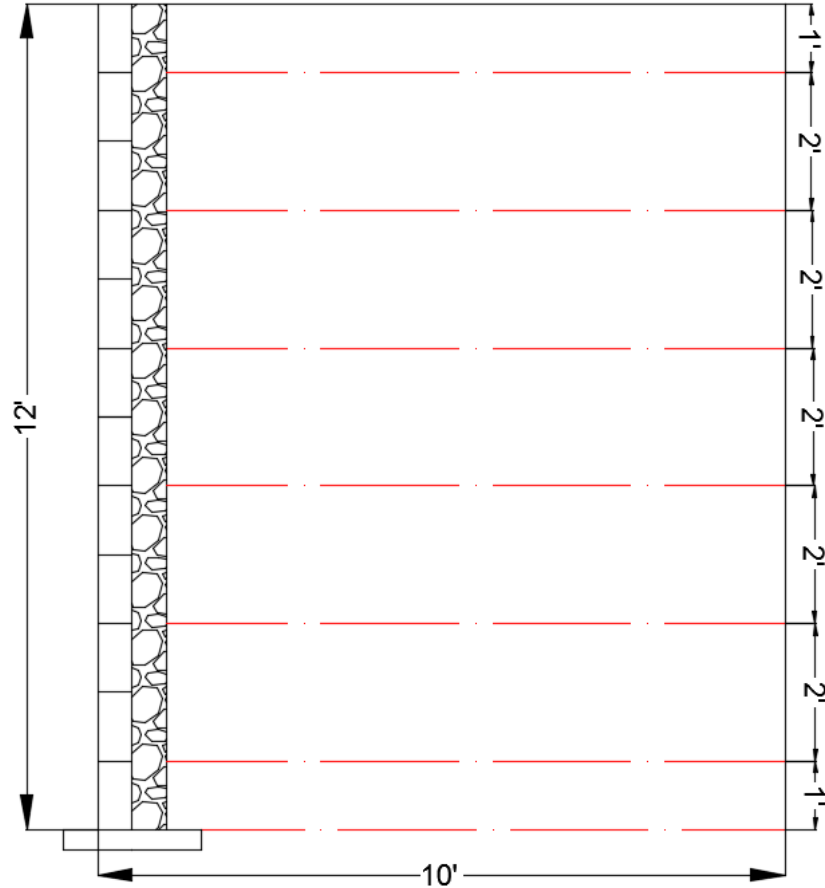
Figure 65: MSE Wall Final Dimensions

Table 13 - MSE Wall Factors of Safety

	Required	Calculated	Sufficient?
Sliding Factor of Safety	1.50	1.51	YES
Overturning Factor of Safety	$e < e_{max} = 2.5$	1.72	YES
Bearing Capacity Factor of Safety	3.0	3.4	YES

4.3.1 MSE Wall Cost

The estimated cost to construct the MSE wall was based on cost per cubic yard estimates (International Project Estimating Limited, 2017) and RSMeans data for building construction costs. Each component of the cost was broken down and evaluated, resulting in an overall cost of **$200,000** for the MSE wall. The individual cost of each item is shown in Tables 14 and 15 below. A quantity of 333.3 cubic yards was used for the volume of the backfill and excavation.

Table 14: MSE Wall Excavation and Backfill Costs

	Excavation	Construction
Labor	6.32 ($/yd^3$)	160.51 ($/yd^2$)
Equipment	5.62 ($/yd^3$)	91.68 ($/yd^2$)
Job Materials	0.11 ($/yd^3$)	2.18 ($/yd^2$)
Permanent Materials	6.17 ($/yd^3$)	641.51 ($/yd^2$)
Total	18.22 ($/yd^3$)	895.9 ($/yd^3$)

Item	Cost	Total Cost
Geosynthetic Reinforcement	0.79 ($/$ft^2$)	$4,148
Masonry Block Units	19.55 ($/$ft^2$)	$17,595
Gravel for Drainage	2.08($/$ft^2$)	$936
Leveling Pad	1400 ($/$yd^3$)	$11,667
Backfill and Excavation	895.9 ($/$yd^3$)	$163,946
Total		$200,000

4.3.2 MSE Wall Schedule

Construction begins with the preparation of the subgrade and pouring of a 1 ft-by-3 ft leveling pad for the MBW units. It will take one to two weeks for the site preparation and 28 days for the concrete to completely cure. Once the MBW units are placed, backfill is placed, compacted, and reinforcement is placed on top and pulled taut. This process continues simultaneously for each reinforcement layer until reaching the top. If this is completed in 20 foot segments of the MSE wall, the ideal construction time is 1-2 weeks per segment. The total construction time of the MSE wall is about 2-3 months.

4.4 Bridge Superstructure Design

4.4.1 Simple versus Continuous Span
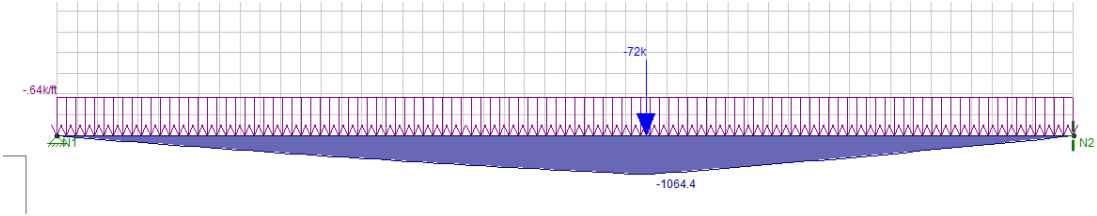
Figure 66 - Simple Span Loading
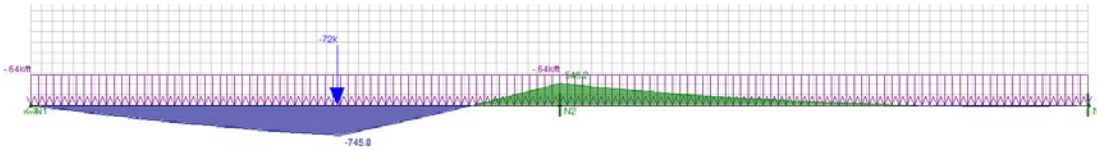
Figure 67: Continuous Span Loading

In the figures shown above, resulting moment diagrams are displayed for both simple and continuous span configurations of typical bridge loading.

Table 16 – Continuous and Simple Span Maximum Moment and Shear

Span Type	Maximum Positive Moment	Maximum Negative Moment	Maximum Shear
Simple	1064.4	0	57.8
Continuous	745.8	546.2	68.7

The table demonstrates that maximum moment in a simple span bridge under the same loading conditions is greater than that of a continuous span.

4.4.2 Determining the Critical Moment on the Span

Below is a table featuring several arbitrary load placements representing a trial-and-error approach to finding the location that would create a critical moment. Emboldened is the iteration that produced the global maximum in terms of absolute moment.

Table 17 – Maximum Load and Moment Location Based on Location

Concentrated Load Distance from Pin (feet)	Maximum Positive Moment (k-ft)	Maximum Negative Moment (k-ft)	Maximum Shear (k)
15	2272.2	2566.7	261.5
20	2415.8	2657.5	274.8
21	2412.7	2671.9	277.4
22	2397.3	2684.8	280.0
23	2369.6	2696.3	282.5
25	2277.8	2714.3	287.5
29	**2042.6**	**2729.3**	**297.0**
30	1987.9	2728.2	299.3
31	1934.2	2725.1	301.5
35	1739.6	2690.4	310.0

Shown above and below is the information and graphic depiction of the critical moment loading scenario and the numerical results that accompany it.
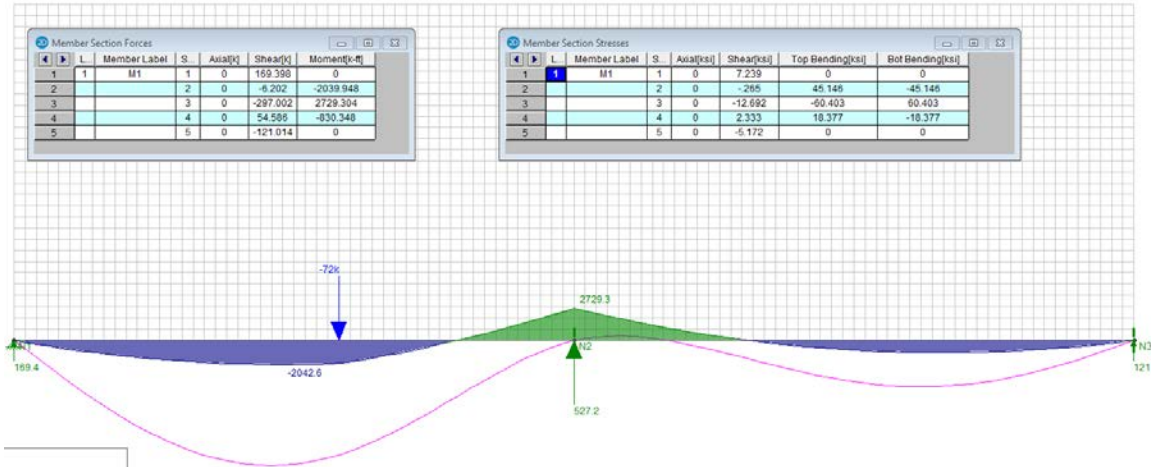
Figure 68: RISA Image of Critical Moment Loading

Included below is a graphical representation of the negative moment, which in this span, as opposed to positive moment, takes on a greater magnitude, vs. the distance from the pin joint where the design truck is placed.
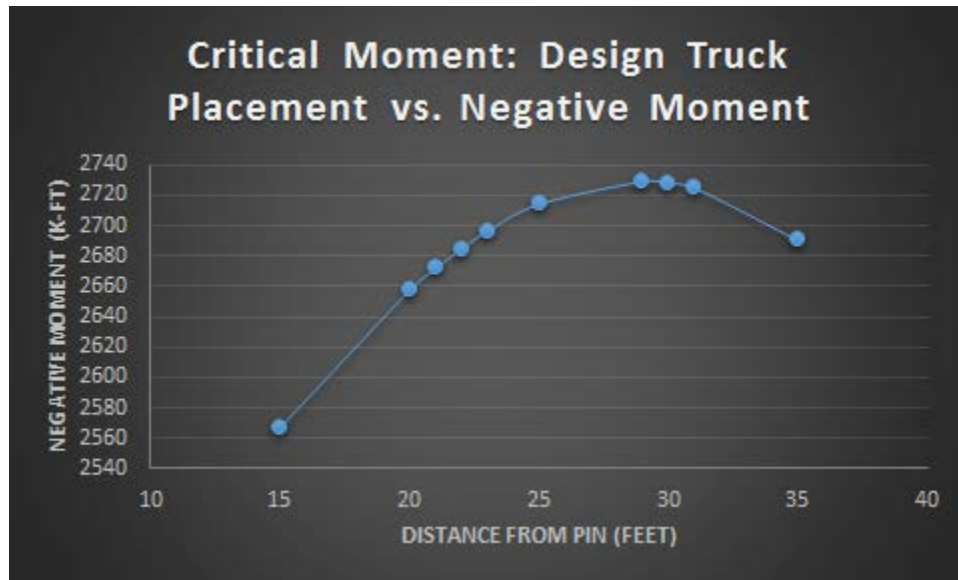
Figure 69: Critical Moment

With a moment of this value coupled with the deflection requirement of L/1000 as per AASHTO Specifications, it was determined that the required beam section for the non-composite superstructure alternative was a W40x215.

4.4.2.1 Non-Composite Cost Estimate

Table 18 - Non-Composite Basic Cost Estimate

Non-Composite	Dollars
Girder	187050
CIP Concrete	122100
Reinforcing Steel	25930
Traffic Barrier	8500
Total (roughly)	**340000**

The value of material cost of the non-composite superstructure, roughly $340,000 dollars, is $40,000 higher than the Composite Superstructure design, which effectively eliminated the Non-Composite Estimate from further consideration.

4.4.3 Concrete Slab Reinforcement
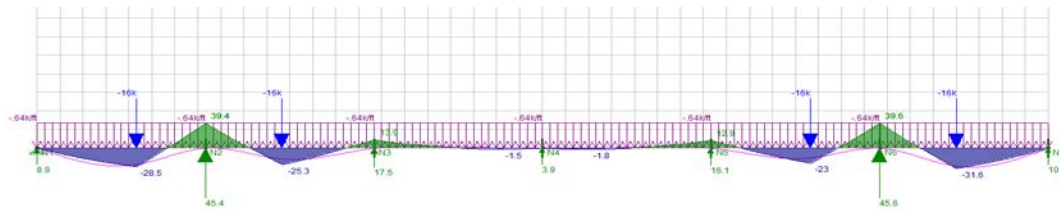
Figure 70: Slab Approximation Moment Distribution

With the above approximate visualization of loading on the slab, the moment result was used to calculate the required area of steel and spacing for the rebar design. For top and bottom rebar, the required area of top steel was 1.83 in^2 and the required area of bottom steel was 1.39

in^2. Therefore, the chosen design was 2 #8 bars on top and 2 #6 bars on the bottom with 2" of cover on top and 1.25" of cover on the bottom as per AASHTO specifications. Spacing was designated to be an even 8 inches throughout the entire slab, and the temperature and shrinkage reinforcement was designated to be every foot throughout the longitudinal section of the slab.
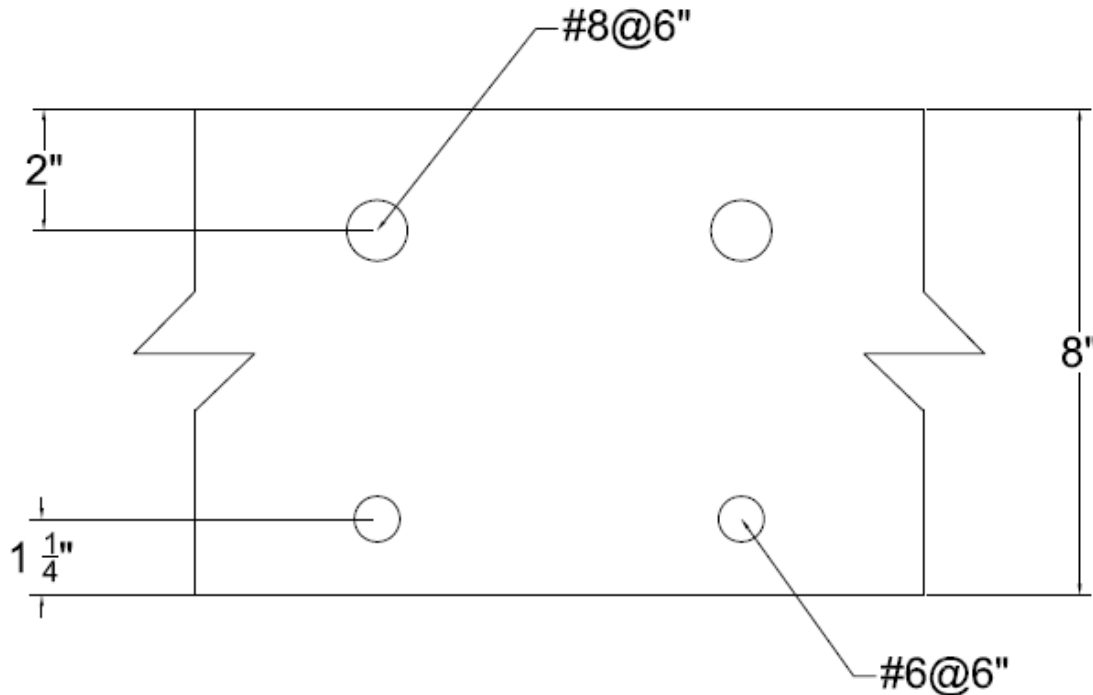
Figure 71 - Rebar Design Section

In terms of longitudinal reinforcement requirements, ACI 318-02, Section 7.12.2.1, lays out that the ratio of reinforcement area to gross concrete area shall not be less than 0.0014. To design conservatively, 0.0018 was the ratio used, resulting in 2 #3 bars for each 1-foot section of the longitudinal slab direction.

4.4.4 Composite Slab Design

The final design for the composite superstructure is shown in elevation and cross section views below.
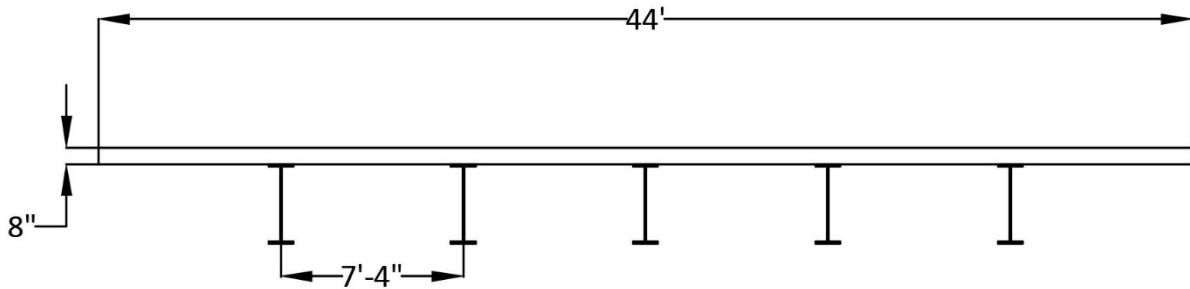
Figure 67: Cross Section View of Composite Slab
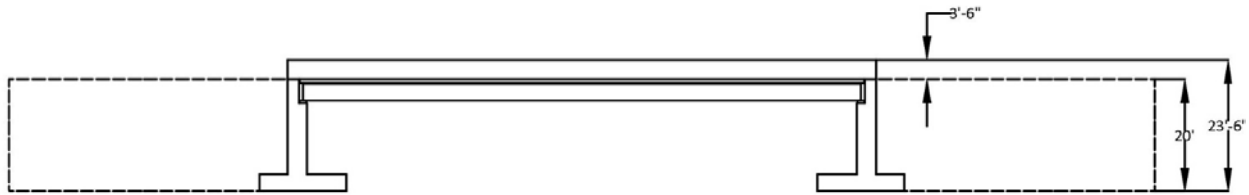
Figure 71: Elevation View of Bridge Design

The bridge designs shown above are the final design options, for a composite slab design. The composite slab design was calculated using the equations and steps given in the Methodology section of this report. The final beam chosen, based on unshored construction loading limits was a W-section with the W24x68 beam. The basic characteristics used to check if this beam was sufficient are below.

Table 19 – W24x68 W Section Properties

Size of Beam	Z_x	I_x
W24x68	177 in^3	1830 in^4

Under initial unshored construction the total loading was 1656.4 plf which included a

bridge deck for the shear studs to be installed into, and an allowable live load for construction,

but does not include a beam self-weight. The calculated moment was 517.6 kip-feet and

produced a total Z_x value was 138 in^3 and as seen in Table 19 above the Z_x for the beam was 177

in^3. This seems overdesigned, but because the beam self-weight was not included it allows for an

increase in the Z_x value due to an added load. The new load including the self-weight was 1738

plf and produced a moment of 543.1 kip-feet and a Z_x value of 144.8 in^3, which was still below

the W-sections beam 177 in^3. It still appeared overdesigned but the deflection still needed to be

calculated. The deflection from the final unfactored loading was 1109 plf and produced a

deflection of 0.586 inches. This value was just below the allowable per AASHTO of L/1000,

which is 0.6 inches. Other smaller beams explored passed the allowable Z_x but the maximum

deflection was larger than the allowable. This beam was sufficient for unshored construction.

The results listed above are outlined in Tables 20 and 21 below.

Table 20 – W24x68 Loading and Allowable Section Modulus

	Factored Load (excluding self-weight)	**Factored Load (including self-weight)**
W_u	1656.4 plf	1738 plf
Required Moment	517.6 k-ft	543.1 k-ft
Required Z_x	138 in^3	144.8 in^3
Beam Z_x	177 in^3	177 in^3
Sufficient?	YES	YES

	Unfactored Load (including self-weight)
W_T	1109.6 plf
I_x	1830 in^4
Calculated Deflection	0.586 in
Allowable Deflection	0.6 in

The next step was to test the W-Section post construction, when it is working as a composite beam with the slab. The loading no longer includes a variance for the concrete and instead includes the HL-93 loading. After using the Tables in the *AISC Steel Manual*, the initial composite beam chosen did not pass the deflection limits. The moment of inertia for the composite beam was 3153.4 in^4 and the deflection calculated was 2.05 inches, which is almost three times the allowable per AASHTO. This meant the moment of inertia for the composite section needed to be increased by a factor of three. The new beam chosen was a W40x167, and passed the required section modulus test and the deflection test for unshored construction. The results for these calculations and the composite beam deflections are outlined in Table 22 below.

Table 22 – W40x167 Allowable Section Modulus for Unshored Construction

	Factored Load (including self-weight)
W_u (plf)	1857.7
Required Moment (k-ft)	579
Required Z_x (in^3)	154.4
Beam Z_x (in^3)	693
Sufficient?	YES

Table 23 – W40x167 and Composite Calculated Deflections

Beam Type	W40x167	Composite W40x167 and 8" Slab
Point in Time	Unshored Construction	Service Loading
Loading Type	Unfactored with Self Weight	Unfactored with Self Weight
W_T	1208 plf	1125 plf
I_x	1830 in^4	18348 in^4
Calculated Deflection	0.586 in	0.297 in
Allowable Deflection	0.6 in	0.6 in
Sufficient?	YES	YES

The reason the composite total unfactored load is smaller than the unfactored load with just the beam is because for the service deflections the distribution factors are included in the load, as per AASHTO, because the entire live load is almost never on girder at the same time. The final step in the design for the composite beam was to determine the shear stud capacity based on the upper bound calculations detailed in methodology section. The shear stud capacity

(Q_n) was 21.55 kips per stud. The capacity was calculated at location 7, making the beam fully composite. The value was calculated using the equations outlined in the methodology section. The shear transfer stress was 2460 kips as given by the area of the W-section which was 49.2 inches multiplied by the yield stress of the steel used which was 50 kips per square inch. This was compared with the capacity and a number of shear studs was reached. The final number was 115 per half span of the bridge, resulting in 230 total studs on the bridge. The spacing was calculated by using the number of studs plus 1 compared to the total span of the bridge in inches. The spacing was 5.19 inches and it was rounded to 5 inches, and was within the limits set by the AISC manual. The minimum distance was 6 times the diameter of the stud and the max was 8 times the slab thickness, but not to exceed 36 inches. The minimum distance was 4.5 inches and the maximum was 36" inches. The value of 5 inches complies with these limits, so the shear stud design was completed.

4.4.4.1 Composite Superstructure Justification

In order to justify selecting a superstructure based on the three designs, namely, non-composite, composite, and prestressed superstructures, we decided to use cost as a measure to justify our selection. Through research into material costs of each of the different superstructures, it was found that a composite superstructure costs $40,000 less than both the non-composite and prestressed superstructures. By virtue of that fact, it was justifiable to select the composite superstructure for use in the final design.

4.4.5 Prestressed Beam

The prestressed structure was designed similar to the composite structure. The deck was still treated as composite, and the beam was simply prestressed concrete instead of a steel girder.

The prestressed beam's design was not calculated using a minimum section modulus because there are various shapes and sizes of typical beams that can be used for a bridge. The beam chosen for this instance, as mentioned before in the Methodology, was a double-tee shaped beam to fit the shorter span length. The double-tee beam chosen specifically was a size of 8DT32, as seen below, with prestressing steel 148-S.

8'-0" in. × 32"
Normalweight Concrete

Section Properties

	No Topping	2 in. Topping
A =	567 in.²	–
I =	55,464 in.⁴	71,886 in.⁴
y_b =	21.21 in.	23.66 in.
y_t =	10.79 in.	10.34 in.
S_b =	2615 in.³	3038 in.³
S_t =	5140 in.³	6952 in.³
wt =	591 lb/ft	791 lb/ft
DL =	74 lb/ft²	99 lb/ft²
V/S =	1.79 in.	

f'_c = 5000 psi
f_{pu} = 270,000 psi
$1/2$-in.-diameter regular strand
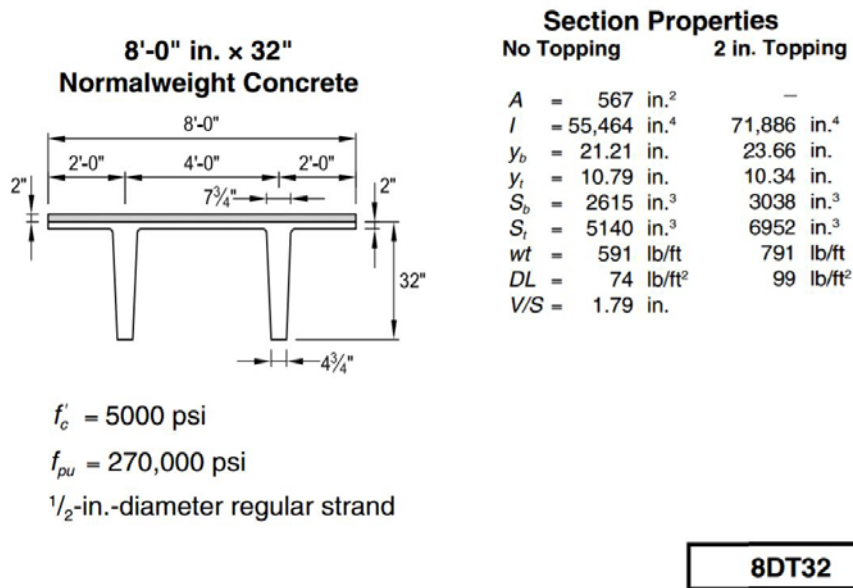
8DT32

Figure 72: Double-tee 8DT32 Section Properties (Prestressed Concrete Institute, 1982),

This beam was chosen based on its ability to handle a load of 199 psf over a span of 50 feet. It is purposefully close to 199 psf because the wet concrete was simply an estimation so 199 psf is close enough. These section properties were used to calculate the stresses for each fiber.

The important thing to notice is that the total moment, including the HL-93 loading, was applied to the composite section modulus for the girder and deck. This is important because it means the beam can be smaller without sacrificing overall area, and thus section modulus. The calculated stresses compared with limit stresses from above are outlined in Table 24 below. the Based on the table of calculated values the 8DT32 beam with 148-S prestressed strands and

topped with an 8" composite slab is sufficient for use in the superstructure. Each final fiber

stress, whether at transfer or under full loading is within the acceptable limits provided by PCI

handbook.

Table 24 – Top and Bottom Fiber Stresses at Different Points in Time

Time Point	Stress Type	Top Fiber Stress (psi)	Bottom Fiber Stress (psi)
At Transfer	**Total**	-521.1 (C)	-1211.8 (C)
	Allowable	-2200 (C)	-2200 (C)
Sufficient?		YES	YES
Construction	**Total**	-1087.8 (C)	-1547.15 (C)
	Allowable	-2000(C)	-2000 (C)
Sufficient?		YES	YES
Composite in service	**Total**	-1563.9 (C)	-31.7 psi
	Allowable	-2000 (C)	-2000 (C)
Sufficient?		YES	YES

4.5 Substructure

4.5.2 Abutment

4.5.2.2 Abutment Calculations

The abutment was treated as a cantilever retaining wall with different loading. The cantilever retaining wall only included the loading from Hl-93 and the soil, where the abutment carries both of the loadings along with the dead load from the bridge. The preliminary dimensions were assumed based on a similar design example, used in the FHWA manual. The dimensions finally used were a total length of 20 feet, with a stem length of 17 feet, and a footing depth of 17 feet. It has a toe length of 7 feet, and a heel length of 5 feet, as seen in Figure 68 below. The stem has a width of 3.5 feet and the footing is a total of 15.5 feet. The width of the abutment was 44 feet.
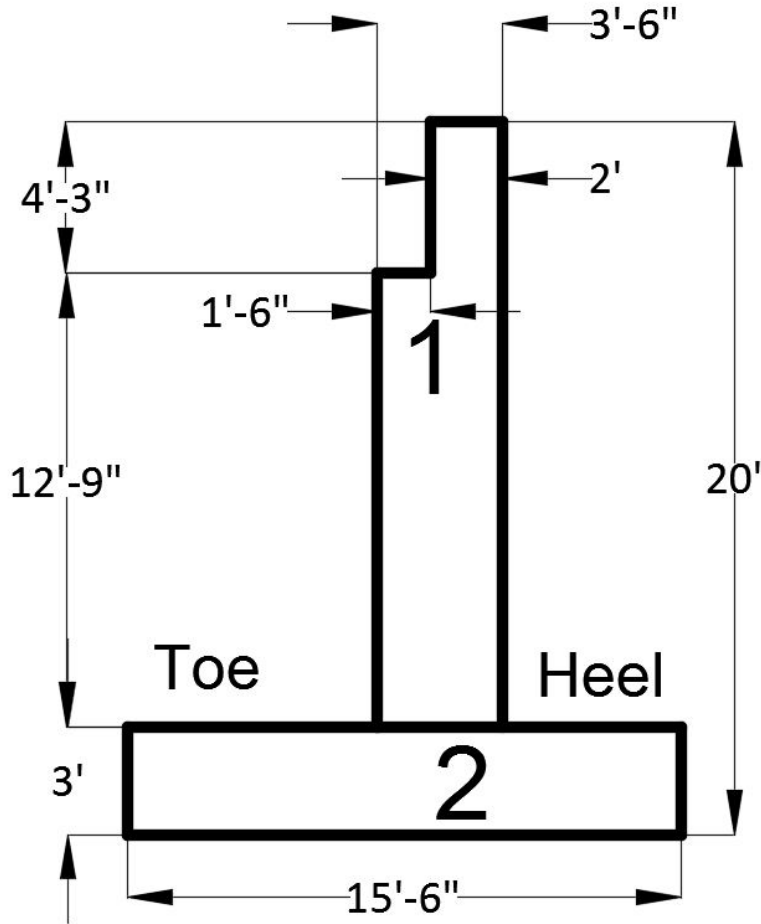
Figure 73: Integral Abutment Dimensions

The loading added to the abutment was the live load from the bridge per AASHTO and the combined dead load of the bridge superstructure, and the soil backpressure. An *Excel* Sheet, was used to calculate the retaining wall Factors of Safety. In accordance with the methodology section, three factors of safety were calculated through a combination of the vertical loads and the lateral forces. The three factors were Overturning, Sliding, and Bearing Capacity, with goal values of 3, 1.5, and 3 respectively. The first few dimensions tried were not able to handle the load and caused the factors to be very small and not close to these goals. The final dimensions above proved to satisfy these requirements.

Table 25 – Required Limit States for Abutment with Calculated States

	Sliding Factor of Safety	Overturning Factor of Safety	Bearing Capacity
Required	1.5	2	3
Calculated	2.83	2.95	36.89
Sufficient?	YES	YES	OK

The calculated values are much higher than the minimums to ensure the viability of the bridge and abutment structural, as well as the dimensions chosen forced such a high value. If the dimensions were reduced the minimums would not be met. Initially the values were below the minimums. To increase the value of the sliding factor of safety, the heel length was increased, and to increase the overturning factor the toe length was increased. Both of these adjustments also increased the bearing capacity, as did reducing the height of the stem.

The final step was to calculate the internal reinforcement for the abutment. The abutment, since it was treated as a retaining wall, follows the same design for reinforcement. The loading was defined from the dead and live loading from the bridge superstructure, and the lateral soil pressure. The live load due to surcharge was 2.99 kips per linear foot. The lateral earth pressure was 7.5 kips per foot, and the lateral earth pressure was 0.75 kips per square foot. The loadings were a result of the active condition constant, the height, and the soil unit weight. The active constant (K_a), was a value of 0.3 and the unit weight was 125 pounds per cubic foot; both values were from the soil report (LGCI, 2010).
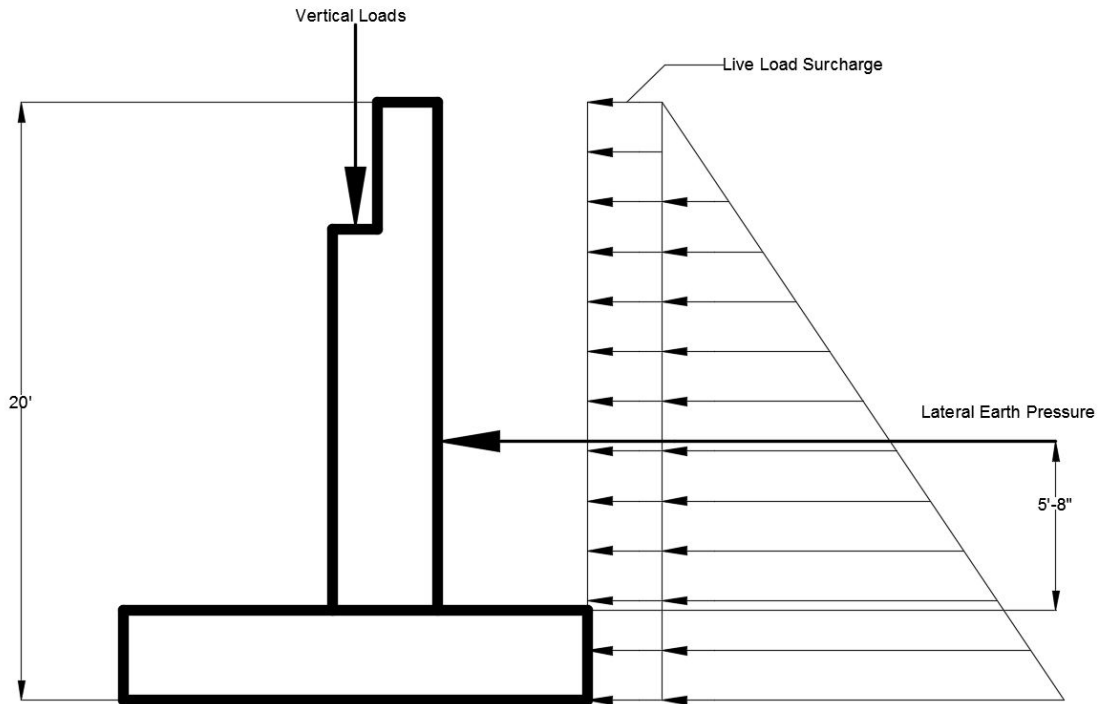
Figure 74: Abutment Loading

The cracking moment was calculated from the size of the stem. The reinforcement was assumed to #9 bars. The effective depth of 33 inches was calculated to be 33 inches, based on a 2.5 inch cover, and the diameter of the #9 bar. The effective depth was used to find the required amount of steel to prevent the concrete from cracking. The stress capacity required per bar was 0.13ksi. The ratio of reinforcement was calculated and used to find the required amount of steel per foot of the stem. The spacing was calculated to check the calculations and check crack control. The #9 bars were calculated to be spaced at 9 inches and the number of bars was proven to be sufficient, given the shear values and the moment values in the stem.

4.5.2.3 Abutment Cost

The estimated cost to construct the abutment was based on estimates from the Florida DOT and the adjusted costs from the MassDOT's weighted bid prices. The estimated volume of

concrete needed for the Substructure was estimated at 4500 cubic feet, based on the design of the

abutment. The cost, according to MassDOT is 1000 dollars per cubic yard. 4500 cubic feet is

about 165 cubic yards, putting the cost of concrete at $165000 in just materials, where man-

hours for a concrete according to MassDOT is around 100 per hour, for a base labor rate. The

estimated hours for the substructure are 10000 hours based on an estimate from a sample

estimate (International Project Estimating Limited, 2017). This puts the labor cost at $1,000,000.

This does not include the excavation costs. The final cost is the steel, which is priced at $2.00 per

pound, and there is an average of 200 inches cubed in the abutment because the #9 bars have a

diameter of 1 inch squared. This puts the cost of epoxy covered steel is $400. The labor was

already calculated. The total abutment cost is $1,065,400 just for the abutment.

Table 26: Total Cost of Abutment

Material	Quantity	Unit	Cost per unit	Cost
Concrete	165	Cubic Yards	$1000/cy	$165,000
Steel	200	Cubic Inches	$2.00/lb	$400
Labor	10000	Hours	$100/hr	$1,000,000
Total Cost				$1,065,400

4.5.2.4 Abutment Schedule

The time for an abutment to be built, varies, but according to a Colorado Department of

Transportation estimation, the abutment needs to occur after the site has been excavated and

graded. The abutment itself to construct should only take a month or two given the time frame of

installing the rebar and placing concrete however the superstructure wouldn't be able to put on

top of the abutment until after concrete has cured, which in this case would take a month on its own. The final step would be to backfill the abutment to ensure is it stabilized, this should only take a day or two of the material arrive while the concrete in curing. The time is estimated at 3 months, from start to completion.

4.5.3 Pier

The calculated load effects for all loads affecting the pier are shown below:

Load Type	Load (kips)
Pier Cap Dead Load	
Overhang	93
Interior	127.88
Total	313.88
Pier Column Dead Load	156.94
Pier Footing Dead Load	144.9
Weight of soil on footing	51.6
Live Load on Bearings(split evenly)	105.44
Braking Force	3.6
Wind Loads	
Attack angle 0 deg (transverse)	19.91
Attack angle 0 deg (longitudinal)	0
Attack angle 60 deg (transverse)	6.77
Attack angle 60 deg (longitudinal)	15.13
Vertical	44
Vehicles	
Atk Angle 0 (transverse)	5
Atk Angle 60 (transverse)	3.4
On substructure	
30 deg (T)	15.75
30 deg (L)	9.1
Temperature Load (same on all 6 bearings)	3.33

Figure 75: Calculated load effects for all loads affecting the pier

Figure 77 solely represents the many different iterations of varying loading categories that contribute to the overall force effects shown below.

Table 27 – Force Effects on Pier Segments

Pier Cap Force Effects	Magnitude
Flexure from Vertical Loads (str)	5659.7 k-ft
Shear from Vertical Loads (str)	797.7 k
Torsion from Horizontal Loads (str)	91.4 k-ft
Flexure from Vertical Loads (ser)	3952.2 k-ft
Pier Column Force Effects	**Magnitude**
Axial Force	1288.1 k
Transverse Moment	4793.3 k-ft
Longitudinal Moment	1019.9 k-ft
Factored Transverse Shear (Str III)	48.8 k
Factored Longitudinal Shear (Str V)	57.7 k
Pier Pile Force Effects	**Magnitude**
Axial Punching Shear	1895.4
Moment (T) Punching	2796.8
Moment (L) Punching	1457.9

The force effects are the main contributors to the design of the pier system. The above results pertain to the approximate effects of this project's load effects on the pier. The effects of the example are of larger magnitude, and therefore result in larger force effects which are used to specify and check the preliminary designs set forth by the FHWA example for pier cap and column reinforcement. Since the preliminary reinforcement designs of the example pass all of the design checks based on the applied force effects, they will be used as the final reinforcement of the pier elements for this project. The figures below display them:
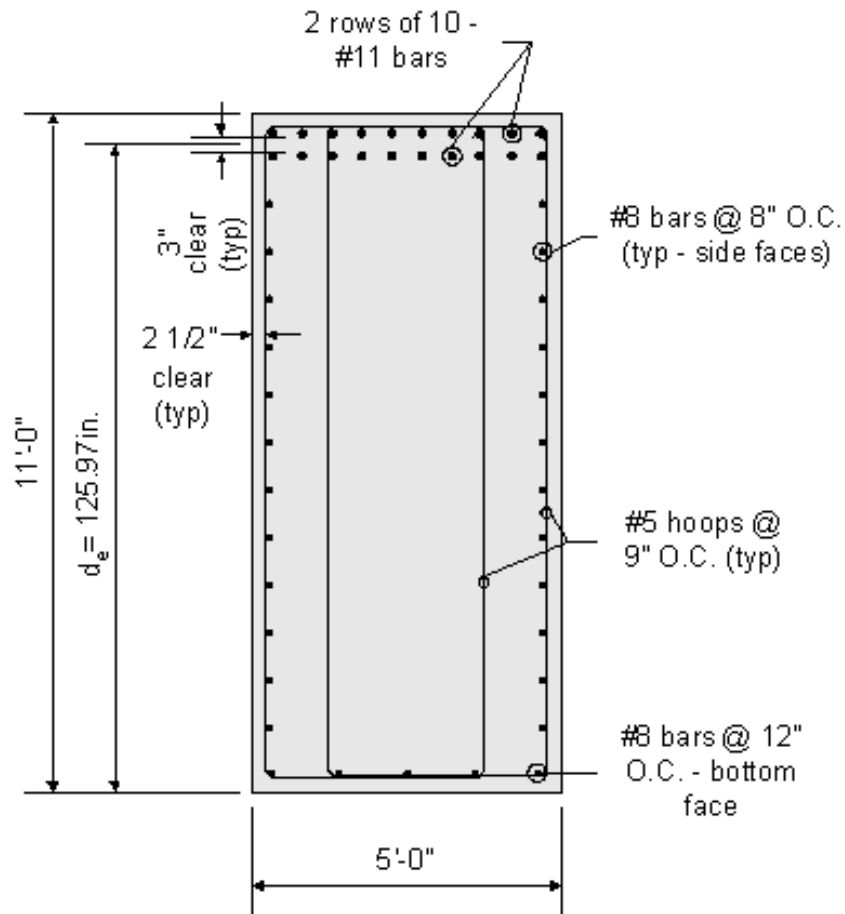
Figure 76 – Recommended Pier Cap Design (taken directly from "Bridges and Structures" USDOT, FHWA)
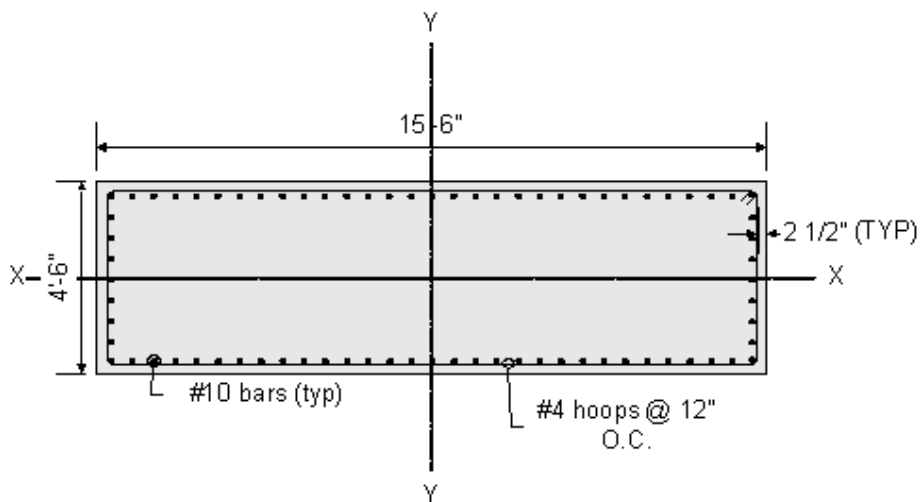
Figure 77 – Recommended Pier Column Design (taken directly from "Bridges and Structures" USDOT, FHWA)

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The results for the footing were found using the equations and methods in the methodology in section 3.0. The dimensions were assumed in order to calculate one and two-way shear values. The values calculated are listed in the table below.

Table 28 – Calculated and Allowable One and Two-Way Shear in Footing

	Calculated (kips)	**Allowable (kips)**	**Sufficient?**
One-Way Shear	319	430	YES
Two-Way Shear	901	2044	YES

All of the values were acceptable for a footing with the dimensions 23 feet by 12 feet. This allowed for a 4.75 foot radius from the end of the footing to the edge of the pier, which allowed for symmetry and for more reinforcement. The reinforcement was calculated to require 256 inches squared of rebar. This equals 324 bars at an area of 0.79 inches squared. The bar spacing was found to be 12 inches on center, using a #8 bar.

4.5.4 Pier Cost Estimate

Table 29 - Pier Cost Estimate Details

Quantities	Material	Volume (CF)	Weight (lb)	$
Pier Cap	Concrete	2092.5	313875	$ 77,500.00
Pier Column	Concrete	1046.25	156937.5	$ 38,750.00
Pier Footing	Concrete	966	144900	$ 35,777.78
Reinforcement				$ 13,200.00
				$165,227.78
			Labor	$ 26,043.39
			Total	$191,271.17

Using the example from the Federal Highway Administration, and adjusted below to reflect that engineering estimates are not as precise as what is displayed above, to find the appropriate volume of concrete and rebar for estimating purposes, the cost of the pier material

was calculated to be $165,000 with labor amounting to be $26,000. Thus, the total estimated basic cost of the pier becomes $191,000.

4.5.4 Wing Wall

The wing wall was based off the cantilever retaining wall design. The dimensions in figure 78 below were the final ones chosen for this design. The total height being 20 feet, a stem height of 17 feet, and a footing length of 14 feet with an embedment depth of 4 feet. This wing wall stretched across 50 feet of road on each side of the bridge.
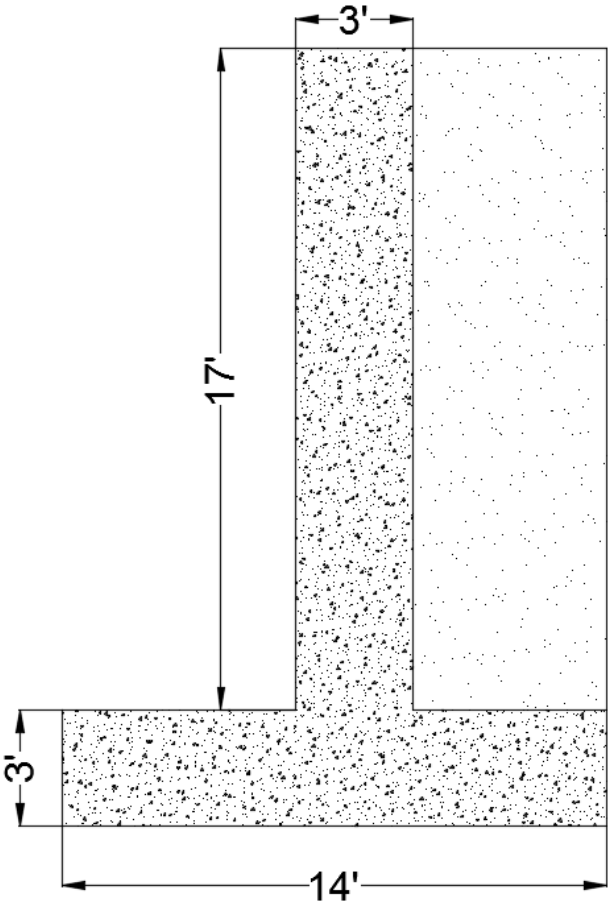
Figure 78: Wing-Wall Final Dimensions

The lateral earth pressure, surcharge load due to traffic, and the weight of the retaining wall and backfill soil were used in this wing wall design. The calculations done by hand can be found in Appendix G along with an excel spreadsheet that was used for iterative calculations. In accordance with the methodology section 3.6.2, three factors of safety were calculated to provide sufficient resistance against the three corresponding failure modes, which are provided below in Table 30. The dimensions in Figure 78 above were used when calculating these factors of safety.

Table 30 – Required and Calculated Wing Wall Factors of Safety

	Sliding Factor of Safety	**Overturning Factor of Safety**	**Bearing Capacity Factor of Safety**
Required	1.5	2	3
Calculated	2.44	2.47	5.39
Sufficient?	YES	YES	OK

These results are similar to the cantilever retaining wall results. The calculated sliding and overturning factors of safety were not much greater than the required ones, unlike the bearing capacity. The calculated factor of safety for bearing capacity for the wing wall was not as high as the cantilever retaining wall one, but was still much greater than the required factor of safety. The problem could be the high effective frictional angle and Terzaghi's bearing capacity method, similar to the cantilever retaining wall.

Table 31 below shows the steel reinforcement required for the final dimensions of the wing wall. The calculations done by hand can be found in Appendix G which were based off of the FHWA example for abutments and wing walls.

Table 31 – Wing Wall Reinforcement Selection

	Back Face Flexure Reinforcement	Front Face Vertical Reinforcement	Horizontal Temperature & Shrinkage Reinforcement	Footing Reinforcement
Wing Wall	#9 bars at 9-inch spacing	#5 bars at 9-inch spacing	#5 bars at 9-inch spacing	#9 bars at 9-inch spacing

4.5.4.2 Wing Wall Cost

The estimated cost to construct the wing wall was based on cost per cubic yard estimates (International Project Estimating Limited, 2017). The total cost for this design was calculated in two parts: one for the excavation and back fill cost, and another for the construction cost. Table 32 shows the unit costs for man-hours, labor, equipment, and job and permanent materials for the excavation and construction.

Table 32 – Unit Cost for Wing Wall Design

	Excavation	Construction
Labor	6.32 ($/cubic yard)	160.51 ($/square yard)
Equipment	5.62 ($/cubic yard)	91.68 ($/square yard)
Job Materials	0.11 ($/cubic yard)	2.18 ($/square yard)
Permanent Materials	6.17 ($/cubic yard)	641.51 ($/square yard)

The quantity of the design was determined to be 1037.04 cubic yards of volume for excavation purposes, and 377.8 square yards of surface area for construction purposes. The quantities for excavation and construction were multiplied by the unit costs from Table 32 above to produce Table 33 below, and summed to determine the total cost of the project.

Table 33 – Total Cost for Wing Wall Design

Item	Cost
Excavation and Backfill	$9,315
Construction	$202,461
Total (Rounded Up)	$220,000

4.5.4.3 Wing Wall Schedule

Once the substructure has been completed and the site excavated, the construction time for a wing wall varies according to the design geometry, and Department of Transportation (DOT) estimations and requirements. The pouring of concrete should approximately take several

weeks, or even two months including the installation of the steel rebar reinforcement. Backfill soil will not be able to be placed behind the wing wall until the concrete has cured for 28 days to achieve maximum compressive strength. With these given time estimates, the design of the wing wall were estimated to be 3 months from start to finish.

4.5.5 Foundation

The dimensions in Figure 81 are the final ones chosen to support the loads from the bridge design. The depth of the foundation was 42 feet, with a shaft diameter of 2 feet, and an underreamed shaft diameter.
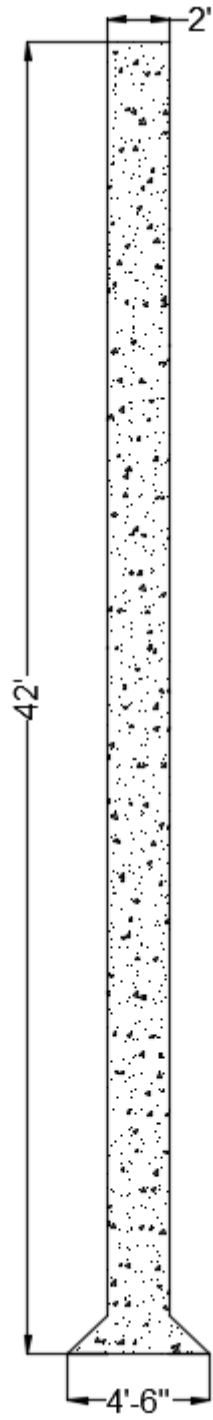
Figure 79: Deep Foundation Final Dimensions

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Table 34 shows the number bar and spacing used for the design. The reinforcement was chosen based on the define geometry and the determined axial loading from the pier results in section 4.10.1.

Table 34: Reinforcement Used in Drilled Underreamed Shaft Foundation

	Longitudinal Reinforcement	Spiral Reinforcement
Number Bar	16 # 10 bars	#6 bars at 4-inch spacing on center

The SPT test values from the soil profile, vertical effective stress, and surface contact area of the foundation were used for this design. The calculations done by hand can be found in Appendix J along with an excel spreadsheet that was used for iterative calculations. In accordance with the methodology section 3.7, the allowable axial compressive load was checked against the determined axial compressive load given from the pier calculations. The table below shows the toe bearing element, and side friction resistance element that were calculated to determine the allowable compressive load. The allowable was greater than the determined axial compressive force, making this foundation design sufficient.

Table 35: Calculated Allowable Downward Axial Compressive Load

$q'_t A_t = 9963.24 \ kips$
$\Sigma f_s A_s = 1518.7 \ kips$
$F.O.S. = 3$
$(P_a)_{downward} = 3827.06 \ kips > 2435 \ kips$
Sufficient Design

The estimated cost to construct the foundation was based on cost per cubic yard estimates (International Project Estimating Limited, 2017). The total cost for this design was calculated in two parts: one for the excavation and back fill cost, and another for the construction cost. Table 36 shows the unit costs for man-hours, labor, equipment, and job and permanent materials for the excavation, backfill, and construction.

Table 36: Unit Costs for Foundation Design

	Excavation & Backfill	**Construction**
Labor	6.32 ($/cubic yard)	466.19 ($/square yard)
Equipment	5.62 ($/cubic yard)	82.67 ($/square yard)
Job Materials	0.11 ($/cubic yard)	143.34 ($/square yard)
Permanent Materials	6.17 ($/cubic yard)	194.65 ($/square yard)

The quantity of the design was determined to be 14 cubic yards of volumes for excavation purposes, and 5.19 square yards of surface area for construction purposes. Table 37 below, and summed to determine the total cost of the project.

Item	Cost
Excavation and Backfill	$17,074
Construction	$4,649
Total Cost (Rounded Up)	$22,000

4.6 Total Cost of Bridge

Superstructure Total Cost

The total cost of the bridge comes from the different stages of the construction process. Construction is a large portion of the costs and if done sustainably can be the majority of costs required to keep the bridge functioning properly. These initial costs are permits, materials, and labor to put the structure in-place and are a one-time cost. These values will be added to the total cost of the initial construction which is outlined in the tables below. These values were taken from previous sections of this project. This begins with the cost of the superstructure.

The total cost of the superstructure was determined based on prices from different department of transportations' values. The values are listed in the table below.

Table 38- Material Cost Estimate – Composite Superstructure

Element	Cost	Citation
W40x167	$150300	WSDOT, 2015
Studs	$1380	FDOT, 2007
Traffic Barrier	$8500	FDOT, 2007
Cast In Place Slab	$122100	FDOT, 2007
Pedestrian Railing	$25932	FDOT, 2007
Labor	$200,000	
Total	$587,932	

Table 39- Composite Superstructure Detailed Outline

Quantities	Material	Volume (CF)	Weight (lb)	$
Girders (W40x215)	Steel	259.68	127243.2	$254,486.40
Slab	Concrete	2933.33	439999.5	$108,641.85
Parapets	Concrete	300	45000	$ 11,111.11
Reinforcement	Steel			$ 13,200.00
				$387,439.36
			Labor	$200,000.00
			Total	$587,439.36

After the cost of the superstructure was found, this needs to be factored in with labor costs. The labor cost for the superstructure was calculated using the values from Infrastructure Project Estimating. The average quantity of man hours for a project of this type was 10,000 hours. The rate for superstructure labor was an average of $200 per hour between the trades involved. This lead to an estimated labor cost of $200,000, as shown above. The total cost of the superstructure was determined to be roughly $588,000.

Site Work Cost Estimate

The site work estimate is detailed below and the values for unit costs were gathered from research on an estimating software and the examples they used. The software was called "Infrastructure Project Estimating" and it specializes in heavy civil site work. These values are based upon the unit cost per area of volume of work done. The areas and volumes were gathered from an estimated size of the site. The Excavation value of 3500cy was estimated based upon the abutment width. The approach slab volume was based on the examples used through the software. The grading and slope paving has to do with the area around the road, at a North-South distance of 350 feet and 50 feet in the East-West Direction. The volumes were estimated based on the dimensions used for designing the bridge, and includes a larger area than just the bridge to account for site cleanup and aesthetics.

Table 40 – Estimated Total Cost of Site Work

	Quantity	Labor	Equip	Materials	Total
Grading/Slope Paving	17500 sqft	$97825	$22225	$2800	$122,850
	Unit Cost (sqft)	*$5.59/sqft*	*$1.27/sqft*	*$0.16/sqft*	
Earth Excavation and Backfill	3500 cy	$188160	$331205	$1785	$521,150
	Unit Cost (cf)	*$53.76/cy*	*$94.63/cy*	*$0.51/cy*	
Total Cost					**$644,000**

Total Cost of Bridge Installation

The next step to determine an estimate for the entire bridge, including foundation, superstructure, and substructure. The values from the other sections needed to be combined. These costs as well as the total costs are shown in Table 41. The final cost of the bridge was $1,609,434.

Table 41 – Estimated Total Cost of Bridge by Element

Section of Bridge	Total
Foundation Piles	$21,723
Pier	$191.272
Abutment	$165,000
Superstructure	$587,439
Site Work	$644,00
Total Cost	$1,609,434

The total bridge construction cost listed above was compared to the estimate shown below. The second estimated was adapted from the Federal Highway Administration's numerical data on unit cost per square foot for bridges, specifically for Massachusetts. The unit cost, valued at $282 per square foot was taken from 2011. Multiplied by the superstructure's area, the cost resulted in the value shown above. This, coupled with the site work estimate, resulted in a total estimated cost of the bridge at $1,894,670.

Table 42 – Estimated Calculated Total Bridge Cost

	Cost
Bridge Construction	$1,240,800.00
Site Work	$644,000.00
Total	$1,894,670

The final table compares the estimate from the FWHA and the estimate established from this project. The table below shows the estimate for this project is lower than the FHWA estimate, by about 15%. This shows the estimate from this project was low, but within reasonable variation for a bid.

Table 43 – Cost Comparison of Bridge based on FHWA

Estimate	**Cost**
This Project	$1,609,434
FHWA Estimate	$1,894,670

5.0 Conclusions

North Lake Avenue has eroded down to one lane due to high rates of traffic flow and weathering conditions. Five-hundred feet of this road has been impacted causing impediment to travel time and discomfort for the local residents. This project proposes to construct an economical and sustainable solution for the current state of this road. Two engineering solutions were compared based upon essential design elements to choose the optimal option within the project constraints. The two solutions that were compared were the design of a bridge versus a retaining wall. The bridge was designed for complete reconstruction and replacement of the road. The superstructure, abutment, pier, and foundation of the bridge were all designed for this project to encapsulate all the parts of the bridge. Composite slab, non-composite slab, and prestressed-precast concrete slab were compared to choose the most economical option for the superstructure. The retaining wall was designed for the restoration of the embankment soil beneath the road. A cantilever retaining wall was compared to mechanically stabilized earth (MSE) wall to choose the more applicable solution. Cost, scheduling, and societal impacts were the decision factors when comparing these two options.

5.1 Results and Key Findings

The project management aspects of the design were completed once all the design work was fulfilled. The cost analysis for excavation, materials, and labor were calculated for both the bridge and retaining wall. Comparing the cost and schedule analysis proved to be the ultimate decision factor for this project. The table below summarizes the final results of the cost analysis.

Table 44: Comparison of the Retaining Walls

Retaining Wall	Cost	Schedule
Cantilever	$220,000	3 Months
MSE	$200,000	3 Months

Table 45: Comparison of the Bridges

Bridge	Cost	Schedule
Superstructure (Composite)	$587,493	2 Months
Abutments	$165,000	3 Months
Wing Walls	$330,000	3 Months
Pier	$191,271	2 Months
Foundations	$22,000	3 Months
Total	1,190,000	1 year

5.2 Comparison of Designs

5.2.1 Comparison of Cantilever Retaining Wall and MSE Wall

The design of both the cantilever retaining wall and MSE wall yield two solutions that offer different benefits to North Lake Ave. Each of the walls carry certain advantages and disadvantages in constructability and practicality, which has a direct impact on the final cost.

Cantilever retaining walls are beneficial in residential neighborhoods since they do not require the use of tiebacks and can be constructed in an open excavation. However, a larger excavation will be necessary to construct the footing of the retaining wall. Combining this factor

with the additional time to properly form the construction site increases the duration for the project schedule. A longer project schedule is not desired for both economic and social aspects. In a project of this statute, it is essential to provide a speedy and reliable solution to allow the residents to resume their daily activities.

MSE walls provide a cost effective structure to successfully restore the site of North Lake Ave. The duration of the schedule is shorter since the use of simple and repetitive construction techniques are used with minimal equipment needed. The construction crew does not require special skills which is an economical advantage. The majority of the costs associated with constructing an MSE wall are from importing suitable fill material to meet the backfill soil requirements.

5.2.2 Comparison of Bridges

In order to justify selecting a superstructure based on the three designs, namely, non-composite, composite, and prestressed superstructures, the cost was used as a measure for the optimal solution. Through research into material costs of each of the different superstructures, a composite superstructure costs $40,000 less than both the non-composite and prestressed superstructures. By virtue of that fact, it was justifiable to select the composite option for use in the final design of the superstructure.

5.2.3 Comparison of Retaining Wall and Bridge

Beam bridges are particularly advantageous in short spans. However, the construction materials and steel needed make beam bridges an expensive solution. The proposed bridge design for a 100 ft section of the road is impractical when compared to a retaining wall. The costs associated with replacing the entire road and risk of tampering with the underlying utilities

add an additional factor to the initial cost. Instead, the grade of the deteriorated area can be restored and supported with a retaining structure, successfully creating a two-lane road at a fraction of the price and time.

5.3 Recommended Design

An MSE wall is the most practical solution for North Lake Ave considering the site properties. Since this structure is between a lake and residential neighborhood, limited space is available to design a retaining structure. An MSE wall requires less space in front of the wall during construction, which is practical for North Lake Ave. The excavation costs are also lower since the dimensions of the MSE wall are smaller than the cantilever retaining wall, and do not require extra excavation for a footing. The shorter construction schedule allows traffic to be restored sooner and is beneficial for the residents. The initial cost to construct the MSE wall is $200,000 which will be constructed in 3 months.

5.3.1 Importance of Chosen Design

The main factor that was crucial in the decision of the MSE wall was the cost of the project, which was significantly less than the cost of the bridge. The MSE wall had far less material that was needed to construct it, which made it the more sustainable solution. The volume that the wall occupied was significantly lower than that of the bridge, meaning there was less site excavation, and ultimately less labor cost associated with it. Having less site to excavate evidently affected the construction schedule of the MSE wall, making the time needed to complete the wall a fraction of the time needed to complete the bridge. The bridge construction involved the driving of piles as deep foundations, which could cause local disturbances and

vibrations that could cause discomfort for residents and interfere with the local hospital

equipment. The MSE wall does not cause vibrations of this magnitude to affect the residents or

local community.

As this report demonstrates, retaining structures offer economical solutions compared to

bridges. Implementing economic, constructability, and social impacts in the design provide an

optimal solution to the issues currently facing North Lake Ave. MSE walls give great advantages

in cost and construction techniques compared to gravity retaining wall systems. A schematic of

the final MSE wall design is provided in Figure 82. This schematic as well as the technical

drawings shown in the results section give a sufficient representation of our presentation.
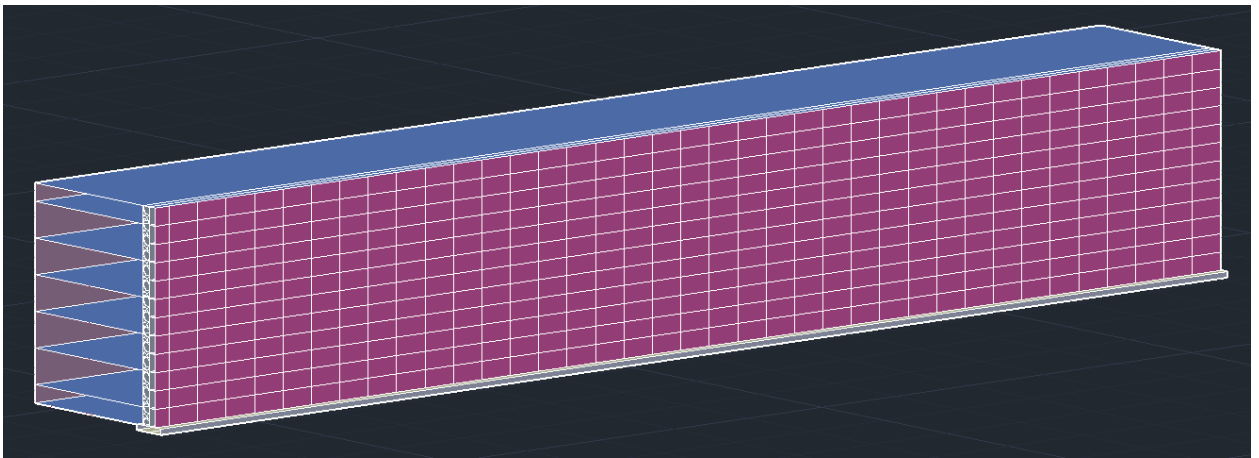
Figure 80: Final MSE Wall Design

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Appendix A

North Lake Ave Bridge Design

Major Qualifying Project Proposal

Submitted to the Faculty

of

Worcester Polytechnic Institute

By:

Jibreel Mustafa

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Date: October 12, 2016

Abstract

North Lake Ave is a road located in Worcester, MA that has been suffering from erosion for many years. Conceptual plans have been made to create a linear park and convert this road into a one-way. The design for a retaining wall and bridge will be compared based on safety, economic, constructability, environmental, societal and sustainability criteria. The design for the bridge includes the superstructure, substructure, and foundation, in compliance with AASHTO LRFD Bridge Design Specifications.

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Capstone Design

To ensure successful completion of this project, the ABET Capstone Design requirements must be entirely fulfilled. Since our project focused mainly on structural, geotechnical, and construction management aspects of civil engineering, we accomplished many of the design requirements. For the structural and geotechnical designs it was necessary to use real world data such as traffic records, boring logs and soil information for this specific site. This data combined with the social impact of the design and construction emphasized the importance of a universally accepted design pertaining to feasibility, performance, safety, and ascetics. Following the AASHTO LRFD Bridge Design Specifications ensured the safety of our design. The construction management component incorporated economical, constructability, and sustainability aspects into our design, which was essential in choosing the most feasible solution. The combination of these elements completes our capstone design requirements.

Constructability

The importance of constructability spans the entire project as it relates to the project's design and overall feasibility of construction. Different materials and orientations of bridge and retaining wall elements were selected based on the needs of the problem and were split into distinct activities. The materials and instructions for each activity are clearly laid out, and areas meant for storage were marked to allow distinct construction element retrieval. Regarding existing conditions, awareness of the unique steep hillside landscape warrants caution and understanding on the part of laborers and engineers in the field.

Social

The surrounding residential area, which is bordering the project area closely, and the traffic that is prominent on the road itself, warrant distinct and prominent social considerations for approaching the project. Consulting with the community and especially the residents whose properties border the worksite is a critically important step in designing the project and tailoring the construction to minimally affect their day-to-day life. Risking stonewalling from the residents on the project's progress could be a serious detriment to the restoration's cost and time of completion.

Economic

Economic constraints will be evaluated for project development in order to reduce construction costs. The economics of the project must come into play throughout the report and re-evaluated to preserve efficiency. The cost of a retaining wall construction was compared to the cost of a simple span bridge construction and erection in order to determine the most effective solution. The scope of the economic constraints includes materials, structural elements (concrete, steel, etc.), and construction management variables. These variables include cost of operation (engineers, construction workers, etc.), the construction plans, and project construction schedules.

Health and Safety

In the design of any construction involving human labor or occupancy, their health and safety are crucial factors to take into consideration in the project. In order to ensure safety of the occupants and integrity of the building, every structural element must be up to the required codes

and standards. Traffic load and member size restrictions were calculated based on the ASHTO LRFD Manual. The factor of safety for retaining walls and bridge foundation were determined using *Foundation Design: Principles and Practices. (Insert Bridge book/AISC night school)* was used in order to determine the factor of safety for a simple span bridge.

Environmental

Construction for both retaining walls and bridges will have environmental impacts that will be considered during the project management process. The excavation of the site will produce possible hazards on Quinsigamond Lake disrupting both traffic and wildlife. The effect that the construction of these structures will have must be taken into account throughout the project. Keeping these issues in mind will help reduce the impacts on the surrounding environment around the site.

Sustainability

Overall, the goal for this project is backed by the concept of sustainable civil infrastructure. The problem for North Lake ave was that the road was not built as sustainable as it could have been. The solution is make a structure with greater service life than what the road previously had. It is important to choose the optimal materials that will resist erosion effects in order to be a sustainable structure. An analysis on the life-cycle was performed on both retaining wall and bridge designs to determine the resistance to environmental deterioration throughout the structure's lifetime. This ensures sufficient sustainable materials were chosen.

1.0 Introduction

North Lake Ave is a road in critical condition overlooking Quinsigamond Lake in Worcester, MA. Since 2009 this road has been eroding at an increased rate due to heavy rainfall, poor soil conditions, and an increase in traffic. This severe erosion has caused the road to be diminished to one lane for the majority of the span, a loss of nearly 12 feet (Shulkin, J. 2009). A temporary solution was proposed in December 2009 to simply place jersey barriers along the right hand side of the road, and install traffic lights on both the northbound and southbound sides. Seven years later North Lake Ave is still subjected to one lane traffic causing major traffic delays and noise pollution during peak hours.

The city of Worcester has moved forward with a $75,000 traffic study analyzing the daily traffic on North Lake Ave (Shulkin, J. 2009). This traffic study is the first step towards replacing the temporary traffic lights and providing a permanent solution to repair this road. The proposed plan is to create a $3.3 million linear park along side Quinsigamond Lake and turn North Lake Ave into a one-way street (southerly). Secondary plans for a promenade along the two mile stretch of the road allowing access for bikers, joggers, and other pedestrians has been conceptualized alongside with the park (Kotsopoulos, N. 2009). The residents of North Lake Ave have expressed major opposition towards this project suggesting a one way street carries many negative impacts on the neighborhood such as increased vehicle speeds, traffic and noise. Creating a one-way southerly also inhibits access to I-290 East from North Lake Ave and redirects ambulance routes from UMASS Memorial Hospital.

The purpose of this Major Qualifying Project (MQP) is to conceptualize, design, and recommend an alternative design to combat the linear park design. This was completed by first designing the retaining wall structure necessary to support the volume of soil needed to create a

linear park and the traffic loads considering North Lake Ave as a one-way. The soil profile and parameters from the recent Burns Bridge project was referenced since it is located less than two miles from the site. The retaining wall design was then compared to an alternative bridge including the foundation, substructure, and superstructure design. The bridge design was accomplished by following the AASHTO LRFD Bridge Design Specifications. Various types of bridge designs were conducted and compared and the governing design was compared with the retaining wall. Each governing design was compared focusing on safety, economic, constructability, environmental, societal and sustainability criteria.

The following MQP report gives a comprehensive comparison of a retaining wall and bridge to improve the condition of North Lake Ave. The background includes information necessary to understand the design aspects of the design components as well as governing factors impacting the design. A methodology was provided outlining and describing the steps taken during each of the designs. The results obtained are shown in various sketches, renderings and comparative tables.

2.0 Background

 North Lake Ave quickly went from being a two-lane road functioning properly, to one of the most detrimental roads in Worcester, MA. The temporary traffic light and jersey barrier solution can not function as a permanent solution. Images taken from Google Maps and our own on-site photos compare the condition of North Lake Ave in the years 2007, 2011, and 2016 (Figure 1). The table provided below shows the constant degradation this road continues to suffer. The condition of the road will continue to worsen if it is kept in this condition. The proposed linear park design will incorporate a retaining wall system supporting the fill for the park as well as the one-way street traffic loads. The alternative bridge design will address the societal issue of having one lane and be designed and continue functioning as a two-lane road.

Figure 1 – North Lake Ave during 2007, 2011, and 2016 respectively

2.1 Introduction to Bridge Design

The major components in the superstructure include the deck, slab, and girders. Data such as projected traffic loadings coincide with the design of these elements. Similarly, the major components of the substructure include the piers and abutments, which are essential for transferring the loads to the foundations. Boring logs and the soil profile of the site are needed in the design of these elements as well as the foundation, which could either be piles or shallow. Scheduling all the activities needed to complete the construction defines this project with a beginning and end. With this information cost estimating becomes more clear and succinct. A brief overview of these components is given in the background to enhance the reader's knowledge of bridge systems, retaining walls, foundations, and soil parameters.

2.2 The Superstructure

The superstructure component of bridge includes the elements located above the substructure including the bridge deck, deck forms, structural members, cross frames, diaphragms, lateral bracing, bearings and other features such as the handrails, parapets, drainage, and wearing surface (Shaner, J. 2016). The bridge deck and steel girders are crucial components to the design of a bridge since they play a major role in transferring the traffic loads. The serviceability of these members must be designed with consideration of safety, sustainability and long-term use.

Figure 2 - Bridge Superstructure (Shaner, J. 2016)

2.2.1 Wearing Surface

The wearing surface is the top layer of the deck that includes the bituminous pavement for the road. This is intended to provide a smooth riding surface for the drivers as well as protect the deck from the weather. The thickness of the layer is dependent on the volume of traffic at the location.

2.2.2 Bridge Decks

The deck of a bridge does not only provide a location to place the wearing surface on, but is responsible for transferring the vertical vehicular loads throughout the superstructure as well as providing lateral stiffness to the superstructure of the bridge (Modjeski & Masters Inc. 2003).

A common deck form is a stay-in-place deck form or removable deck forms. The benefit of stay-in-place deck forms is moisture control and added strength to the deck. These elements are located directly above the stringers of the bridge and have the option to be designed compositely or non-compositely. A composite design is when a concrete slab is firmly connected to the steel beams providing longitudinal shear transfer between the two members (McCormack, Jack C. 2012). This is accomplished in bridges by using steel anchors to connect the reinforced concrete slab to the stringers (McCormack, Jack C. 2012). Composite designs provide increased strength and allow the bridge to act as a unit in resisting loads. The 1944 AASHTO Specifications approved the method of composite design and it has been incorporated in the majority of bridge deck designs since the early 1950's (McCormack, Jack C. 2012).

2.2.3 Cast-in-Place Concrete Slab

One of the most common types of concrete decks is cast-in-place concrete due to its low cost and constructability (CA.DOT. 2015). A layer of concrete is poured on site usually between 7 and 12 inches thick on top of the reinforcing steel (CA.DOT. 2015). Since concrete provides strength through compression, the reinforcing steel is beneficial in providing the necessary tensile requirements. As mentioned in section 2.2.2, the composite design between the deck and stringers benefits the strength of the deck allowing 33% - 50% more load to be supported (McCormack, Jack C. 2012). Some disadvantages associated with a cast-in-place concrete deck are cracking and rebar corrosion. This could potentially increase the money spent on bridge maintenance and damage the wearing surface (CA.DOT. 2015).

2.2.4 Prestressed-Precast Concrete

The second most common deck is precast concrete, which is prefabricated concrete slabs that are either reinforced with steel rebar or are prestressed (CA.DOT. 2015). These pre made panels are delivered to the construction site ready to be installed. This advantage expedites the construction schedule and has less of a social impact than other methods. Similar to a cast-in-place concrete deck, this could be constructed to be a composite member.

Normal concrete has a very low tensile strength, and thus cracks can develop in the early stages of loading. Prestressing fibers increase tensile and shear stress capacity at the midspan of the beam. A prestressed beam reacts more elastically, and has the ability to recover of cracking and deflection, but once the tensile strength of the concrete has been exceeded it acts exactly as a reinforced member.

The use of prestressed concrete can be utilized in the bridge deck, or superstructure of the bridge, wherever the use of concrete beams is used. Prestressed concrete comes in many varieties. Beams can be pretensioned, before the concrete is cast or post tensioned after it has been cast. Pre tensioning has an advantage in the manufacturability, as it is easier to mass-produce, and the tension is spread more evenly throughout the beam or slab. In post tensioned beams there is less curing time and objects can be cast in place, and will resist elastic shortening better.

The main things to consider when designing a prestressed beam are the shape, the size, and the loading most importantly. These specifics allow the beam to be designed accordingly. The beam can be designed according to the specific project's needs. The amount of design needed in prestress is also a flaw, the tendons and beams need to analyzed before tensioning, after the

concrete has been cured, and counting for the losses in prestress. The type of anchor needs to be evaluated, as well as the size and type of tendon being used.

2.2.5 I-Girders: Rolled Beams

The most common steel beams used are W-shapes that have parallel inner and outer flange surfaces, which give the beam the distinct "I" shape. Limited sizes and shapes are widely manufactured so it is essential to incorporate a size that is readily available. Custom made W-sections will cost more and will take longer to produce. These types of girders are useful for short span bridges under 200 feet, otherwise a girder with a longer web may be needed to span the longer distance (AISC 2016).

2.2.6 Cross Frames, Diaphragms, and Lateral Bracing

Cross frames and diaphragms provide torsional stability for steel girder bridges during construction and remain permanently fixed (Shaner, J. 2016). Lateral bracing provides lateral stiffness which decreases the lateral deflections from the horizontal forces on the bridge (Shaner, J. 2016)
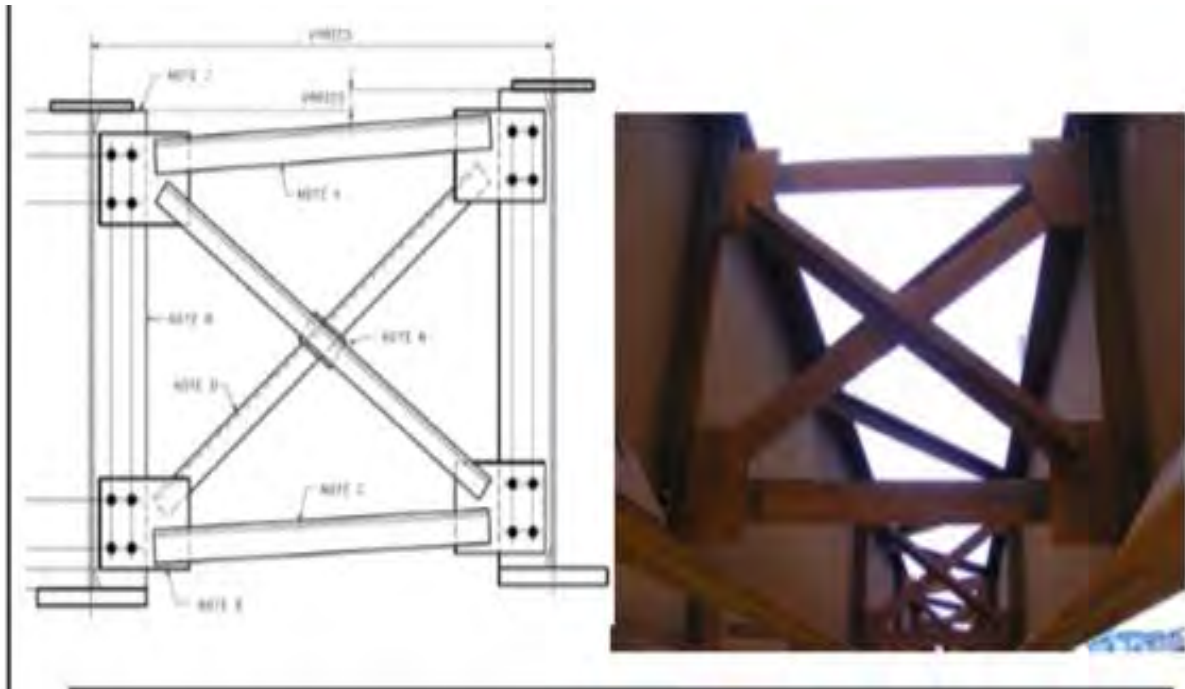
Figure 3 - Cross Frames (Shaner, J. 2016)

Figure 4 - Diaphragms (Shaner, J. 2016)

Figure 5 - Lateral Bracing (Shaner, J. 2016)

2.2.7 Bearings

Bearings can be considered as a part of the substructure or a component in and of itself. This is the component of the bridge that transfers the superstructure stresses through the substructure to the foundation. When designing the bearings for a bridge it is important to meet the certain requirements (Fu, G. 2013):

1. Ability to transfer vertical forces from the superstructure

2. Ability to accommodate horizontal translation along the bridges longitudinal axis due to thermal and load effects

3. Ability to accommodate rotation on the transverse axis of the bridge

4. Ability to function as a tie down system to secure the superstructure to the substructure to prevent uplift

In order to accommodate both steel and concrete girders, rollers (Figure 6) and elastomeric (Figure 7) bearings will be used for the design. These bearings allow translational and rotational movement to minimize the stresses given from the superstructure (Fu, G. 2013). The design of the bearings must focus on the maximum load carrying capacity and be able to withstand the translational and rotational stresses.
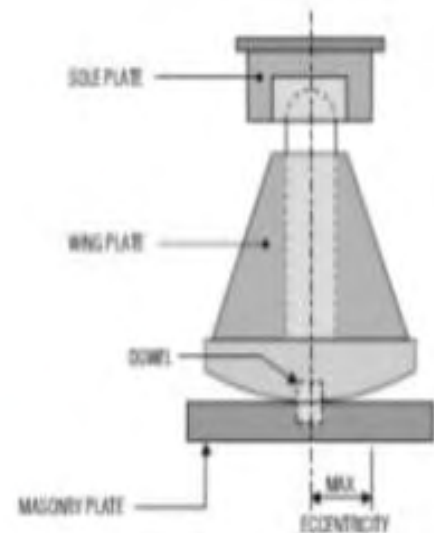
Figure 6 - Rocker Bearing (Shaner, J. 2016)
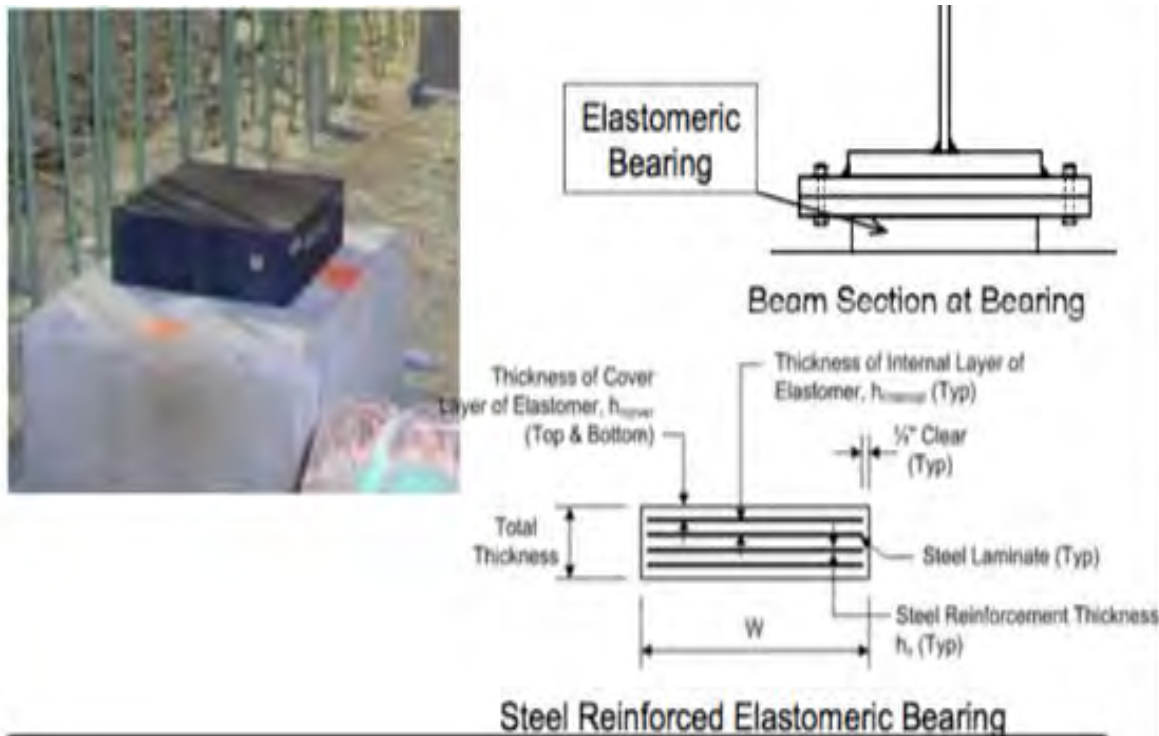
Figure 7 - Elastomeric Bearing (Shaner, J. 2016)

2.3 The Substructure

The basic definition of the substructure of a bridge is anything below the superstructure which includes: any abutments (end bents), piers (bents), pier caps (bent caps), or columns (FIG 20/22 AISC). Each of these elements are critical in the design of the bridge and need to be designed, like the superstructure, with sustainability, safety, and long term use.

2.3.1 Abutments (End Bents)

The abutment is where the roadway ends and the bridge begins. Its purpose is to support the loads of the superstructure and the soil pressures from the roadway embankments. Different

characteristics need to be considered when choosing an abutment type like bridge geometry (e.g length, clearances) anticipated loads, future maintenance, and constructability.

2.3.1.1 Conventional Abutments

This abutment type is characterized by a joint separating the bridge deck from the approach and backwall, expansion joints, wingwalls, and includes a bearing that separates it from the superstructure. A conventional abutment can be tall or "stub". Tall abutments can function as a retaining wall and do not require the use of a header slope. Stub abutments are usually capped at a nominal height and require a header slope of anywhere between 4:1 and 1:1. A stub abutment needs to combined with a retaining wall in front of it.

2.3.1.2 Integral and Semi-Integral Abutments

This type of abutment where the different features of the bridge: superstructure, abutment, and foundation are all integrated together. The superstructure is set on top of the abutment cap and a closure pour ensures the superstructure is cast into the abutment. A concrete pour isn't always used. Other methods like reinforcing structures, or anchors are also employed. Integral abutments offer no intentional moment relief. Since the foundation is integrated into the entire abutment, H beams or drilled shafts and spread footings are used to support the structure. The abutment can be similar to a conventional abutment, where the wall can be "stub" or tall. These types, like the conventional, also include wingwalls, and expansion joints. What classifies the type as integral or semi integral is the extent to which the superstructure and foundation is connected to the abutment. A semi-integral structure usually has some type of bearing to account for intentional moment relief.

2.3.2 Piers (Bents)

The basic pier elements include the pier cap, vertical support, intermediate struts, and intermediate bracing. All of these elements are most commonly fabricated from steel or concrete. There are a plethora of possibilities when designing because there are these different components to each pier . Each pier combination has its own benefits and risks; however the goal is to choose the right pier for the bridge that is being designed. The local site conditions, and vehicle traffic are important to consider, as well as, the aesthetics and proportions. Pier's should also be analyzed across both axes to ensure the loading capacity and moment behavior will change depending on the direction.

The pier caps are often integrated into the pier or superstructure. This can help improve efficiency in constructing or loading, it can also improve the aesthetics, and can improve clearances. As with the piers themselves there are many options when choosing the optimal pier cap. The pier cap choice comes down to the location, material, size and configuration or the piers. Often times, to alleviate the complications, an expansion joint is included below the superstructure.

2.4 Foundations

Designing foundations involves a few subdisciplines of civil engineering in order to fully understand how to go about determining dimensions, load resistance, and construction methods. Foundations are structural components that carry the loads from the structure to spreads those loads to the soil beneath and around it (Coduto, D. P. 2001). Foundations depend heavily on the soil properties and parameters from the geotechnical report. Lastly, foundations must be economically built for the sake of construction costs. The materials, methods, and any sort of construction constraints must be planned and designed for ahead of time (Coduto, D. P. 2001).

2.4.1 Driven Piles

Driven piles have been the preferred deep foundation for bridge design, especially for marine or near shore applications (Figure _). Driven piles are also environmentally friendly, leaving the construction site virtually clean and debris free. This deep foundation is driven to a required design depth for sufficient resistance against compression, tension, and lateral loads. Pre-drilling may be necessary if the driving needs to penetrate dense soil to the required depth (Baker, H. 2016). Driven piles are created to ensure sufficient quality, reliability, and strength to conform to ASTM standards. Driven piles maintain their shape and integrity during the driving or installation process and can be verified visually and dynamically.

Dynamic and static tests can determine adequate load carrying capacities and effects of hammer performance on the foundation. Usually driven piles are the most economical foundation option for many projects, and are the most structurally superior compared to other foundations. "The wide variety of materials and shapes available for driven piles can be easily fabricated or specified for high structural strength, allowing them to be driven by modern hammers to increased working loads thus requiring fewer piles per project, resulting in substantial savings in foundation costs," (Association, P. D. C. Benefits of Driven Piles. 2016). When driven into water, these foundations can immediately be ready for use, which reduces construction time of the project. "For bridges or piers, driven piles can be quickly incorporated into a bent structure allowing the bridge to pier itself to be used as the work platform for succeeding piles in top-down construction," (Association, P. D. C. Benefits of Driven Piles. 2016). "After the pile is driven into the site, the foundation can actually have increased load

carrying capacity as a result of the driving process. This phenomenon is called "setup" which can produce fewer or shorter piles, saving on construction costs such as time, labor, and materials" (Association, P. D. C. Benefits of Driven Piles. 2016). Driven piles are very adaptable according to structure type, site details, and budget constraints. These foundations can be either steel (tapered, shell, or sheet pile), concrete (square, cylinder, or sheet pile), or timber (Association, P. D. C. Benefits of Driven Piles. 2016).

2.5 Cantilever Retaining Wall

A cantilevered retaining wall is often the more economical choice of retaining walls' (Figure_). Cantilevered retaining walls are typically used for heights no more than 16 feet (Coduto, D. P. 2001). In order to have a successful retaining structure, both internal and external stability requirements must be met. External stability refers to the retaining wall staying fixed in the designed location. Internal stability, or structural integrity, ensures the retaining wall to be able to transfer internal forces to the soil underneath the wall without rupturing. Both of these requirements must be satisfied individually in order to have a sufficient retaining structure (Coduto, D. P. 2001). The dimensions of the retaining wall will come from these analyses.

Common Terminology

Ground Level

Back Fill

Construction Joint

Stem

Ground Level

Key

Drainage

Base Slab, Footing
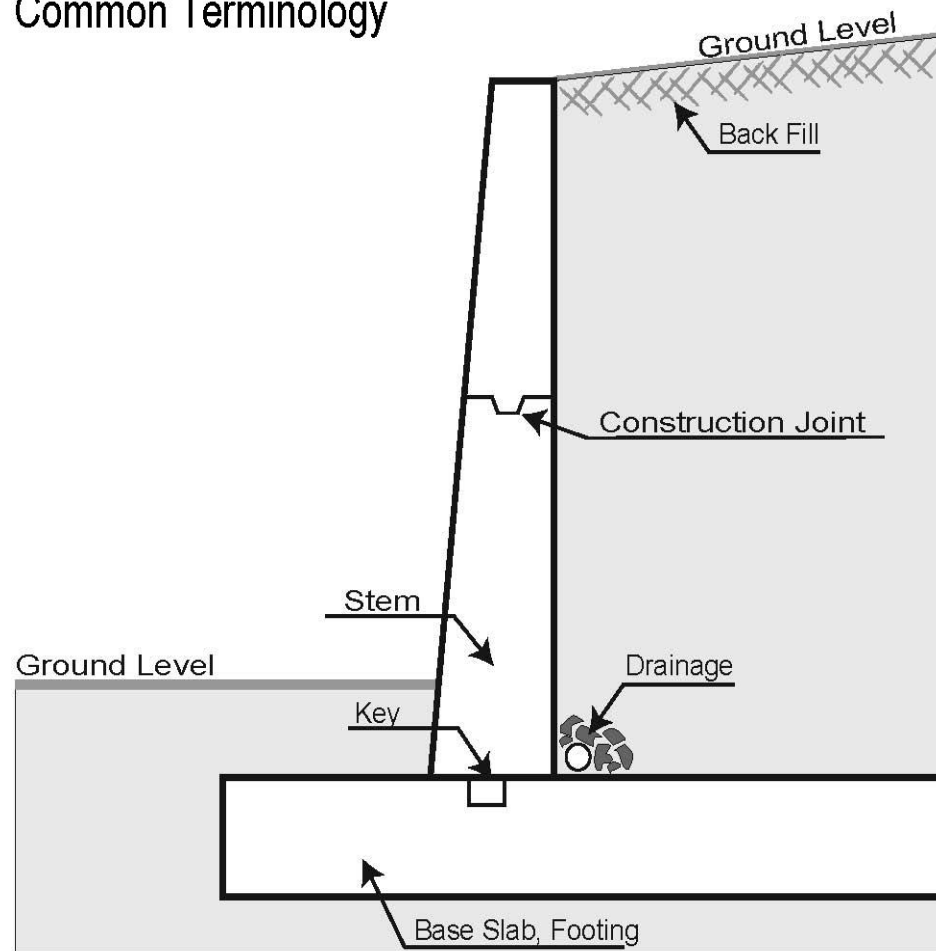
Figure 8 - Retaining Wall Design Calculations. 2012

2.5.1 External Stability

In terms of external stability, a cantilever retaining wall must not slide (Figure 9). A limit equation is used when evaluating the sliding stability. The factor of safety is taken into consideration for this limit equation, being the sum of the resistant forces (lateral earth pressure, sliding friction, etc.) divided by the driving forces (hydrostatic forces, seismic forces from backfill). A "true" factor of safety of 2.5 to 3.5 is the most suitable result for a cantilever retaining wall (Coduto, D. P. 2001).

Once sliding stability requirements are met, overturning stability must be taken into

consideration (Figure 10). The factor of safety equation is similar to the one for horizontal

sliding equation, but instead of the resistant and driving forces, it involves resistant and

overturning moments. The resistant moments must be summed together in one direction divided

by the overturning moments in the other direction (clockwise and counterclockwise). This means

the factor of safety is calculated depending on the location of the chosen point about which the

moments are taken (typically the toe of the footing). Typical overturning moments are the

horizontal component to the lateral earth pressure, hydrostatic forces acting behind the "wall-soil

unit," surcharge loads, and seismic forces from the backfill (Coduto, D. P. 2001). Typical

resistant moments are the vertical component to lateral earth pressure, the weight of the "wall-

soil unit," surcharge loads, and hydrostatic pressure acting on the front of the footing.

Overturning analysis neglects the normal force between the footing and the ground due to the

fact that this force has no moment arm (acts through the center of the overturning). This analysis also neglects the friction force for the same reason. The minimum factor of safety required for the overturning is 1.5 to 2.0 (Coduto, D. P. 2001).
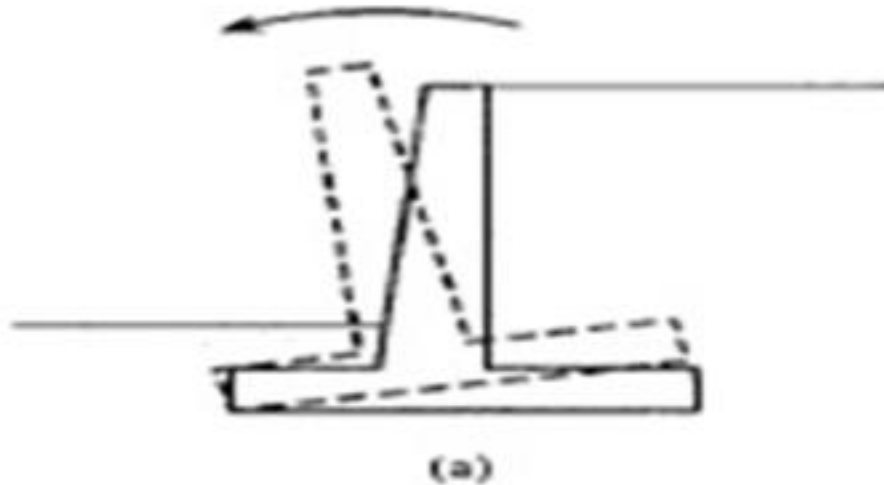
Figure 10 - Overturning Stability Failure (Retaining Wall Design Calculations, 2012)

2.5.2 Internal Stability

Once the external stability requirements are met, the retaining wall's internal stability (structural integrity) must be analyzed. The structural design must resist any applied loads with the sufficient factors of safety. The analysis of the internal stability begins with the stem, and then goes into the footing of the retaining wall. The footing is almost always made out of reinforced concrete (Coduto, D. P. 2001). Tall retaining wall stems are made of reinforced concrete, while shorter ones can use reinforced masonry. Reinforced concrete stems have much greater flexibility, and flexural strength, making it the most cost effective for tall retaining structures (Coduto, D. P. 2001).

2.5.3 Lateral Earth Pressure

"One of the first steps in the design of earth-retaining structures is to determine the magnitude and direction of the forces and pressures acting between the structure and the adjacent ground. The most important of these is the pressure between the retained earth and the back of the earth-retaining structure. This is called a *lateral earth pressure* because its primary component is horizontal. Another lateral earth pressure acts between the front of the foundation and the adjacent ground," (Coduto, D. P. 2001).

The coefficient of lateral earth pressure influences the lateral earth pressure acting on the retaining wall. The coefficient of lateral earth pressure is the ratio of horizontal to vertical effective stress at any point in the soil. Three soil conditions were defined as the at-rest condition, active condition, and passive condition (Coduto, D. P. 2001).

2.5.3.1 At-rest, Active, and Passive Conditions

For the at-rest condition, assume the retaining wall that resists flexural movements (rigid) and no translation or rotation (unyielding). Also making the assumption that there is no lateral strains within the ground will make the lateral stresses as if they were in a natural state (Coduto, D. P. 2001).

The active condition allows for very small movements that change the lateral earth pressure. In figure _ shows the transition from the at-rest condition to the active condition. This movement may be translational or rotational which reduces part of the horizontal stress, causing the Mohr's circle to expand (Figure _). The passive condition is the opposite of the active condition. Instead of the wall moving away from the backfill, the passive condition is when the retaining wall moves towards the backfill. The passive condition involves more movement than the active condition. "Notice how the vertical stress remains constant whereas the horizontal

stress changes in response to the induced horizontal strains. Engineers often use the passive pressure that develops along the toe of a retaining wall footing to help resist sliding," (Coduto, D. P. 2001).
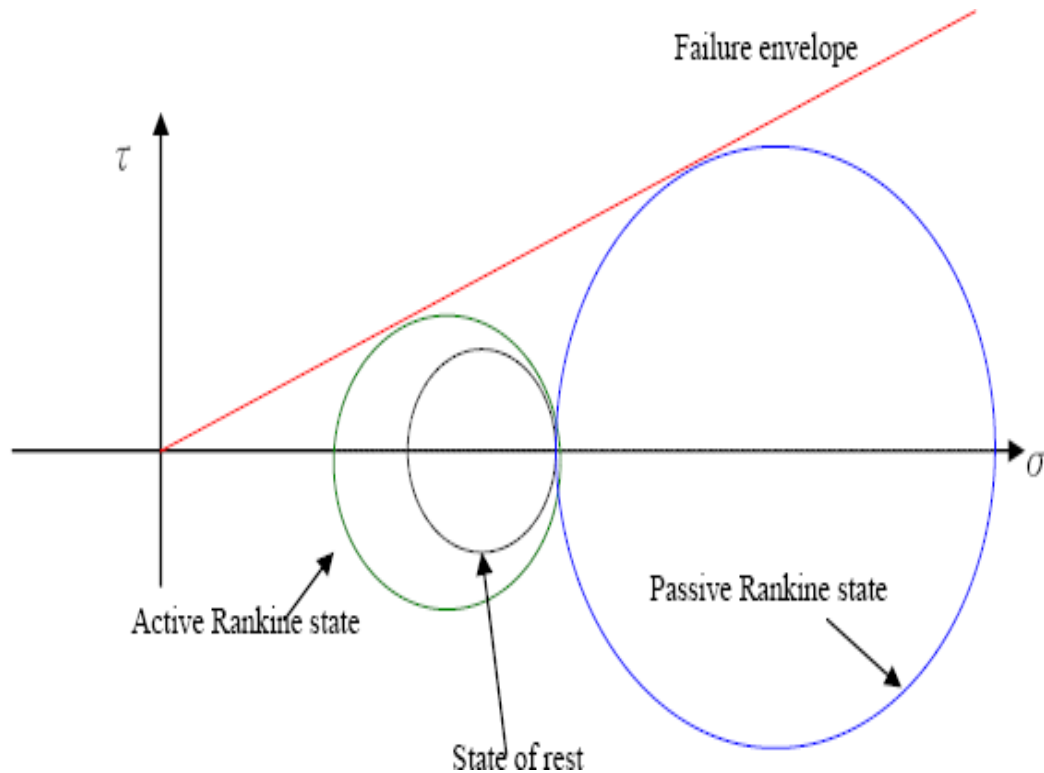
Figure 11 - Friction Angle and Failure Envelope

2.5.4 Retaining wall design Criteria

Retaining walls must abide by a criteria checklist before the process of designing the retaining wall. Four primary concerns must be met in order to meet the design criteria. Acceptable factors of safety for overturning and sliding must be met. The allowable soil bearing pressures should not be exceeded, and the structural integrity requirements should be within code allowable limits to be able to resist vertical and lateral loadings (Nielsen, H. 2013).

Before starting the retaining wall design, certain factors must be taken into consideration for this design criteria checklist. A soil investigation report with soil properties and parameters must

be established. Is there a property line condition or water table needed to be considered, and what building codes apply? Is there lateral restraint on the top of the wall? Is there a slab in front of the wall to restrain sliding or prevent erosion of the soil? Should the stem be reinforced concrete, masonry, or a combination of both? What is the slope of the backfill and how will the backfill be drained? Will there be any axial loading or seismic design required (Nielsen, H. 2013)?

Along with this design criteria checklist, the following values must be established/calculated in order to begin design process:

- Retained heights

- Embedment depth of footing required below grade *

- Allowable soil pressure (1,000 psf - 3,000 psf) *

- Passive earth pressure (150 pcf - 350 pcf) *

- Active earth pressure (30 pcf - 55 pcf) *

- Coefficient of friction (0.25 - 0.4)*

- Backfill slope (Do not exceed 2:1, Horizontal:Vertical, unless approved by geotechnical engineer)

- Axial loading on stem

- Surcharge loads

- Wind loads (If applicable)

- Seismic design (If applicable)*

- Soil density (110 pcf-130 pcf)

- Concrete (or masonry) allowable stresses

    - F'c: 2,000 psi-4,000 psi

    - Fy: 60,000 psi

- Unit side friction resistance (Fs): 24,000 psi

- Mobilized side friction resistance (F'm): 1,500 psi

- Fr: 145 psi-178 psi (Strength design)

- Unit Weight (γ)*

* These values are generally given in the geotechnical report. (Nielsen, H. 2013)

2.6 Project Management

In developing the design and constraints for solving the problems affecting North Lake Avenue, it is critical to assess how the design shall be brought to fruition, given the complexity of the landscape that is given. Much of the project, especially post design phase, will be the conceptual cost estimate and the creation of a scope of work in the form of activities and a work breakdown structure. Beyond the design of the actual bridge and retaining wall structure will be the actual implementation and building of the project, due to its relevancy in the solution of the engineering problem at hand. Therefore, when the design is close to completion, commencing on the use of Primavera, a scheduling application, and similar software applications to formulate a detailed plan for construction shall become a primary focus. In doing so, the hope is that the social, traffic, and complicated landscape issues may be resolved through reasonable planning and understanding.

The proximity of the road to the Quinsigamond Lake, the residences bordering North Lake Avenue, and the bottleneck of traffic at the location justify an in-depth look at the methodology with which we must go about designing a framework for building the project. Coordinating equipment and scheduling to minimize noise and disruption of traffic flow is a paramount consideration in the approach to this project. Communication of the plan to local

residents prior to commencement of work is also an important factor in building as their cooperation and our accountability to them is a large consideration at every point in the project.

2.7 Societal Impacts

2.7.1 Societal Impacts of Public Construction

Due to the extremely close proximity of the collapsed road to residential areas, construction cannot commence without disruption of access that residents will have to their homes, as the inevitable closure of the road will prevent street access on the North Lake Avenue side. One of the main problems that the project must address is how to effectively perform construction such that the residents are not heavily disturbed and that they are able to carry on with their day-to-day affairs in a way that is satisfactory to them.

In a study performed regarding road construction in China that impacted local residents, they determined that "inefficient communication is the most critical risk where public awareness plays a mediation role" (Wang, Han, et. al, 2016) Given that the road directly borders several properties, some of which depend on the road for partial to full access to parking and/or pathways to their home, disturbance of access to their home during paving or the installation of any bridge or retaining wall for some amount of time is virtually inevitable. Given this fact, it is necessary to approach this critical issue with the idea of efficient communication at the forefront of the project's mindset.

The nature of the repair of this road is ultimately a necessary thing, and formulating an understanding in the residents that are affected by this issue is thereby a necessity as well. In the approach to designing and constructing this project, designing a platform of open communication with the residents regarding the parameters of the project is just as necessary. The question underlying it all is therefore: "How do we develop a reasonable method of communication with

the public such that the project is able to progress without inhibition from the nearby residents?" This is a question we need to address within the scope of our project. Designing around this constraint will have to be a priority, given the fact that the creation and completion of road repairs on North Lake Avenue depends on it.

3.0 Methodology

3.1 Evaluation Methods

The deliverable of this project is to propose an alternative bridge design instead of the retaining wall design incorporating a linear park. The loading calculations for the soil and traffic will be following the *American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications 2012 Manual*. Based on the intended use for this bridge design, we determined this specifications manual as the most appropriate.

3.1.1 Criteria

We will be comparing cost, life expectancy, schedule, environmental impact, and ethics of the bridge versus retaining wall design. These categories were based on our team's capstone design requirements. The following subsections will be going into more detail on how our team evaluates each category.

3.1.1.1 Cost

We plan to evaluate the cost of the project based on the scope of work that all the elements of our designs will entail. For each design, including retaining walls, bridge superstructure, substructure, pile caps, abutments, girders, and other elements, we will gather total quantities of material, develop a system of activities, and generate total man-hours for the implementation of the project, forming a detailed estimate from this data.

Through use of software such as Primavera (and Timberline as it becomes available), we hope to develop a system of specific activities and develop a cost per square foot conceptual estimate in Uniformat of each trade and activity to provide a cohesive and intuitive means of setting the stage for building the project.

3.1.1.2 Life Expectancy

Our team considers life expectancy, or sustainability (as mentioned in our capstone design), of both the bridge and retaining wall. The life expectancy (in years), or life cycle, is however long the structure stands before needing a complete replacement. When evaluating this criterion, any major maintenance performed on the structures will be taken into consideration. The life expectancy of the bridge and retaining wall will be determined through case studies on similar projects. Noting the information on construction methods, and materials used from the case studies that our team will research. This will allow the group to compare how often certain structures are materials need to be replace.

3.1.1.3 Scheduling

The schedule of this project is an important logistical factor and necessary complement of the design in terms of real world application; knowing the breakdown and sequence of each specific activity is a critical element should this project to come to fruition. In terms of a comprehensive and detailed plan to construct the designs we create, generating a schedule of activities and using other case studies to determine the correct order and duration of activities would fulfill this aspect of the project. Using Primavera, a scheduling software for project management applications, we will compare this project to other similar ones and develop a sensible and coherent breakdown of activities for the construction of a bridge and retaining wall as it pertains to our unique design. Combined with a cost estimate, we hope to provide a realistic avenue for this project to be implemented given the opportunity.

3.1.1.4 Ethics

   The bridge and retaining wall design will be evaluated in terms of their ethics on calculating the loading that each structure can withstand. Design loads will be defined using the American Association of State Highway and Transportation Officials and Precast/Prestressed Concrete Institute design specifications. As powerful tools, these design specifications are necessary in the design process for the steel and concrete sections for our bridge design. These specifications will also assist in determining the design loads for both the bridge and retaining wall design. We will give more details on how our team will use these specifications in our sections on the superstructure and substructure.

3.2 Applying Loads

   The 2012 AASHTO LRFD Design Specifications loading traffic scheme HL-93 from section 3 of the AASHTO manual will be used to predict and produce the vehicular stresses imposed on this road. For the case of the retaining wall traffic loads will be considered for a one-way street, whereas the bridge design will incorporate two lanes.

3.3 Superstructure

   For each type of bridge (steel girder or prestressed, precast girder) there will be two different designs in which the governing design will triumph. Following the design of the girders are the deck design and the bearing design.

3.3.1 Steel Girder Design

Each component of the bridge will be designed with reference to the 2012 AASHTO LRFD Bridge Design Specifications. The AISC Steel Construction Manual will be implemented for the design of the steel girders. As previously stated in the introduction to the proposal, the design will be in accordance to the capstone design requirements.

3.3.2 Prestressed-Precast Concrete Girder Design

Once the span, and length of bridge have been designed and if the material of choice is concrete, then the girders can be designed. The design of the girders needs to be cost effective, safe, sustainable, and timely. The design will be made adequate to the established loads. The shape, reinforcing steel, and manufacturability need to be considered. For determining the prestress tendon size, spacing, and needed shear reinforcement, the shear stresses and moments need to be calculated. After the moments have been calculated the stresses in the beam can be determined, and will need to be compared to the ACI code to make sure the beam will not fail.

The design also needs to consider constructability. Can the beams be produced economically and efficiently? Some parts of the bridge may need to be over designed to ensure the beams are easier to produce. The method in which the tendons are tensioned also plays a role in constructability. If the beams are pretensioned all the tensioning is done in an offsite location. If it is post tensioned, the tension needs to be applied on site and the places for the tendons filled with grout. There needs to be ample space on the site to allow for this. If not then post tensioned beams will not work.

3.3.3 Bridge Deck Design

The deck slab design will be completed with reference from an example design flowchart given by FHWA. A different slab will have to be designed for both the steel girder and concrete girder bridges. AASHTO and PCI specifications will be considered for the design of the deck.

3.3.4 Bearing Design

The bearing design will differ for each bridge design produced. Once the superstructure is accurately designed, the dead load can be used to determine the sufficient bearing capable of withstanding these forces. As mentioned earlier in the background, rollers and elastomeric bearings will be analyzed and meet the following requirements:

1. Ability to transfer vertical forces from the superstructure

2. Ability to accommodate horizontal translation along the bridges longitudinal axis due to thermal and load effects

3. Ability to accommodate rotation on the transverse axis of the bridge

4. Ability to function as a tie down system to secure the superstructure to the substructure to prevent uplift

3.4 Substructure

The substructure is an integral piece of the bridge and the roadway around it. Each piece of the substructure: abutments, piers, pier caps, or columns needs to be evaluated according to cost, life expectancy, environmental restrictions, and social considerations.

3.4.1 Abutments

The abutments have many things to consider when being designed. The dead and live loading from the bridge and roadway needs to be taken into account. The shape and direction of the bridge also needs to be evaluated, and then the integration of a retaining wall needs to be considered for this project. How far the wall will run and if it will be used in conjunction with a bridge and if it could be integrated into the abutment wing walls, or how the slope will affect it. The shape and scope of the bridge will also dictate if the abutment can be integrated into the rest of the bridge. Specifically, the height of the abutment, preliminary proportions, the soil pressures acting on it need to be determined, as well as safety criteria, and then finally material and timeline.

3.4.2 Piers

The span of the bridge needs to be determined in order to determine how many piers will be needed. Once this has been decided the height, and the size of the caps can be designed. If the pier caps are going to be integrated, or made of steel of concrete and if they need to be reinforced. The footing depth needs to be considered, in relation to the frost level, groundwater table, and general conditions of placement. The size of the footing will be designed based on number of piers and loading from the bridge.

3.5 Retaining Wall Design

Once the retaining wall design criteria have been completed, the actual design process will begin. The first step is load computation such as soil pressures, axial, surcharge, or seismic loads. Once the loads are computed, the stem of the retaining wall must be calculated and

designed. Starting at the bottom of the stem is preferred because this is where the maximum shear and moment forces are. "Then, for economy, check several feet up the stem (such as at the top of the development length of the dowels projecting from the footing) to determine if the bar size can be reduced or alternate bars dropped. Check dowel embedment depth into the footing assuming a 90° bend (hooked bar)," (Nielsen, H. 2013). For the most part, the minimum thickness needed for the top of the retaining wall is six inches. The thickness through the stem varies the further you go down, and a thickness of eight inches is typically used for the bottom.

The next step is to calculate the overturning moments about the toe of the retaining wall's footing. The assumption that the footing width is approximately one-half or two-thirds the stem height is typically used for trial examples. Once the overturning moments are determined, calculating the resisting moments is necessary based upon the assumed footing width for the overturning moments. Once the moments are calculated, the external stability (Sliding and overturning) must be checked to have a sufficient factor of safety of 1.5 or more (Nielsen, H. 2013). If this factor of safety is not achieved, adjusting the depth of the footing or implementing a key on the bottom of the footing will assist in achieving the desired factor of safety. The next step is, "based upon an acceptable factor of safety against overturning, calculate the eccentricity of the total vertical load. Is it within or outside the middle-third of the footing width?" (Nielsen, H. 2013).

The soil pressure at both the toe and heel must be calculated to find the eccentricity. If the eccentricity is greater than the width of the footing divided by six, this is not recommended because it will be outside the middle third of the footing width. A triangular load distribution of the pressure will be the result since there cannot be tension between the soil and the footing. If

this condition is the result, then a consultation with the geotechnical engineer is recommended because this design will have a lower allowable soil bearing pressure (Nielsen, H. 2013).

The design of the footing of the retaining wall will strictly be based on the moments and shear forces, which might require steel rebar reinforcement. Check and review all calculations, and make sure all of the geotechnical requirements have been met throughout the design process. "Place a note on the structural sheets and on the structural calculations indicating that the backfill is to be placed and compacted in accordance with the geotechnical report," (Nielsen, H. 2013). The last step is to review the construction blueprints and specifications are in compliance with the retaining wall design (Nielsen, H. 2013).

3.6 Work Breakdown Structure

	Activity	Member	Delivery	ABET Requirement
	Proposal Progress	All	A-Term	
-- -- -- --	Retaining Wall Design	All	Early B-Term	Design, Health Safety, Environmental, Sustainability, Constructability
	External Stability Analysis	Ryan S	Early B-Term	
	Internal Stability Analysis	Jibreel M	Early B-Term	
	At-Rest condition	John C & Ryan C	Early B-Term	
-- -- -- -- -	Bridge Design	All	Mid B-Term	Design, Health Safety, Environmental, Sustainability, Constructability

	PreStressed Design	Ryan C & Ryan S	Mid B-Term	
	Steel Girder Design	Jibreel M & Ryan S	Mid B-Term	
	Foundation Design	All	Mid B-Term	
	Traffic Load Analysis	John C	A/B-Term Break	
-- -- -- -- -	Construction management	All	Mid to End B term	Constructability, Economic, Social
	Cost Analysis	Jibs M & Ryan S	End B term to Early C term	
	Project Schedule	John C & Ryan C	Mid to End B term	
	General Requirements/Conditions	Ryan C	Early B term	
	Potential Subcontractors	John C	Early B term	
	Draft Architectural Drawings	All	Mid to End B term	Design
	Activities For Project Completion	All	Early B term	
-- -- -- -- --	Introduction/Abstract	Jibreel M	A-Term	
-- -- -- -- --	Background	All	Mid B-Term	

-- -- -- -- --	Methodology	All	B-Term	Design, Health Safety, Environmental, Sustainability, Constructability
-- -- -- -- --	Results/Design	All	End of B-term --> Early C-Term	Design, Health Safety, Environmental, Sustainability, Constructability, Social
	Discussion/Conclusion	All	C- term	Social
-- -- -- -- --	MQP Finalized and Submitted	All	End of C-Term	

4.0 Deliverables

The completion of this project will provide a proposed alternative bridge design, instead of a retaining wall structure, that is most suitable for the North Lake ave residents and the city of Worcester, MA. We will have analyzed the evaluation process the team created to assess the categories that were chosen such as life cycle, cost, and scheduling. Well will have a write up of the hand-written calculations that have been checked using available softwares mentioned prior to this section. Lastly, our team will present graphical representations of important data for better and easier understanding and presentation.

5.0 Conclusions

5.1 Chosen Design

Based on the complex circumstances of the site, we expect the governing design to be a bridge with prestressed, precast concrete girders. This decision was based on the constructability of the material, allowing faster construction to mitigate negative societal impacts. The design of a retaining wall structure would be more economical and feasible than a bridge, however we considered the major societal impacts of each design.

5.2 Possible Problems

The problems we as a team will encounter are as follows:

- Not having access to Timberline for construction management calculations

- Calculating for every possible force the structure might encounter

- Finding well defined soil parameters/properties from the exact site

- Contacting MassDOT/The Beta Group (TBG) for information on the Burns Bridge

We as a team believe that not having access to Timberline will hinder construction management calculations such as material management, job costing, and item billing. To resolve this issue, the team will consult with advisors or other professors to use different software tools to achieve the same results as Timberline. To help resolve the issue on calculating every force in this design, our team will submit calculations several times to our advisors before the final draft of this MQP. A professional mindset and experience will assist in any mistakes made while ensuring we have sufficient and correct load calculations at the end of this process. We believe that well defined soil parameters will be difficult to get a hold of for North Lake ave, however an alternative

source will be used. The Burns State Bridge (about 2 miles away from the site) has already been constructed, leading the team to believe there is sufficient soil data from this project. This ties into getting into contact the MassDOT and The Beta Group for information on the soil for this project. Instead of emailing or making a phone call to these organizations, we will visit their offices in-person for direct contact and answers. Most of the design work is based upon this soil information and the soil mechanics involved in this site. To keep from inhibiting the design process, existing conditions will be assumed if needed and made note of in our final methodology if we do not obtain well-defined soil information.

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Appendix B

Cantilever Retaining Wall Calculations

- Assumptions : $\phi = 35°$, $\gamma = 125$ pcf $\mu = 0.55$ $\sigma_{sc} = 240$ psf

- Vertical effective stress σ_z' : $\sigma_z' = (125 \text{ pcf})(4 \text{ ft}) = \underline{500 \text{ psf}}$

- Lateral earth Pressure :
At rest : $K_o = 1 - \sin\phi = 1 - \sin(35) = \underline{0.43}$
Active Condition : $K_a = \tan^2(45° - \phi/2) = \tan^2(45° - 35°/2) = \underline{0.27}$
Passive Condition : $K_p = \tan^2(45 + \phi/2) = \tan^2(45 + 35°/2) = \underline{3.69}$

- Lateral Forces
• $F_{soils} = P_o/b = F_{P1} = \dfrac{\gamma h^2 K_o}{2} = \frac{1}{2}(125 \text{ lb/ft}^3)(12 \text{ ft})^2(0.43) = \underline{3870 \frac{lb}{ft}}$

• $F_{surcharge} = F_{P2} = \sigma_{sc} \cdot H \cdot K_o = (240 \text{ lb/ft}^2)(12 \text{ ft})(0.43) = \underline{1238 \text{ lb/ft}}$

• $F_{water} = F_{P3} = 0$ (gwt under footing)

- Moments (with respect to Point O)
• $M_{soils} = M_{P1} = (F_{P1})(\ell_1) = (3870 \text{ lb/ft})(4 \text{ ft}) = \underline{15.5 \text{ K-ft/ft}}$

• $M_{surcharge} = M_{P2} = (F_{P2})(\ell_2) = (1238 \text{ lb/ft})(6 \text{ ft}) = \underline{7.4 \text{ K-ft/ft}}$

• $M_{water} = M_{P3} = \underline{0}$

- $M_{result} = \Sigma M_O = M_{P1} + M_{P2} + M_{P3} = 12.5 \text{ K-ft/ft} + 7.4 \text{ K-ft/ft}$

$\therefore \boxed{M_{result} = 22.9 \text{ K-ft/ft}}$

- $F_{result} = \Sigma F = F_{P1} + F_{P2} + F_{P3} = 3870 \text{ lb/ft} + 1238 \text{ lb/ft} = \boxed{5.12 \text{ K/ft} = F_R}$

- $(F_R) h_r = M_R$ $\therefore$ $h_r = \dfrac{M_R}{F_R} = \dfrac{22.9 \text{ K-ft/ft}}{5.12 \text{ K/ft}} = \boxed{4.47 \text{ ft} = h_r}$
(above point O)

Cantilever Retaining Wall Calculations Cont.

Vertical Forces

- **Weights:**
- $W_{wall} = W_1 = \gamma_c \, bh = (150 \text{ lb/ft}^3)(1.2 \text{ ft})(10.8 \text{ ft}) = \underline{1.944 \text{ K/ft}}$
- $W_{footing} = W_2 = \gamma_c \, bh = (150 \text{ lb/ft}^3)(8.4 \text{ ft})(1.2 \text{ ft}) = \underline{1.512 \text{ K/ft}}$
- $W_{soil} = W_3 = \gamma_s \, bh = (125 \text{ lb/ft}^3)(6 \text{ ft})(10.8 \text{ ft}) = \underline{8.1 \text{ K/ft}}$
- $W_{surcharge} = W_4 = (\sigma_{sc})(\text{heel length}) = (240 \text{ psf})(6 \text{ ft}) = \underline{1.44 \text{ K/ft}}$

- **Moments due to weights (about Point O)**
- $M_{wall} = M_{W1} = W_1(x_1) = (1.944 \text{ K/ft})(1.2' + \frac{1.2'}{2}) = \underline{3.5 \text{ K-ft/ft}}$
- $M_{footing} = M_{W2} = W_2(x_2) = (1.512 \text{ K/ft})(8.4'/2) = \underline{6.35 \text{ K-ft/ft}}$
- $M_{soil} = M_{W3} = W_3(x_3) = (8.1 \text{ K/ft})(6'/2 + 2.4') = \underline{43.7 \text{ K-ft/ft}}$
- $M_{sc} = M_{W4} = W_4(x_4) = (1.44 \text{ K/ft})(6'/2 + 2.4) = \underline{7.8 \text{ K-ft/ft}}$

- $M_{result} = \Sigma M_W = M_{W1} + M_{W2} + M_{W3} = 3.5 + 6.35 + 43.7 = \boxed{53.6 \text{ K-ft/ft} = MR_1}$

  (No surcharge)

→ with surcharge load: $\underline{M_{R2} = 61.4 \text{ K-ft/ft}}$

- $(W_R)h_w = M_{R_1} \therefore (1.944 \text{ K/ft} + 1.512 \text{ K/ft} + 8.1 \text{ K/ft})h_w = 53.6 \text{ K-ft/ft}$

  $\therefore \boxed{h_w = 4.64 \text{ ft}}$ (to the right of Point O)

  $\boxed{W_r = 11.56 \text{ K/ft}}$

- **Factors of safety**

- Overturning: **Driving Moment:** $M_D = (F_R)(h_r) = (5.12 \text{ K/ft})(4.47 \text{ ft})$

  $\therefore \underline{M_D = 22.9 \text{ K-ft/ft}}$

- Resisting Moment: $M_R = (W_R)(h_w) = (11.56 \text{ K/ft})(4.64 \text{ ft})$

  $\therefore \underline{M_R = 53.64 \text{ K-ft/ft}}$

- $F.S. = \dfrac{M_R}{M_D} = \dfrac{53.64}{22.9} = \boxed{2.3} > 1.5 \quad \checkmark$

— Factors of safety: <u>Sliding</u>

• Driving Force: $F_R = 4.37$ K/ft

• Resisting Force: $F_w = 11.56$ K/ft.

• F.S. sliding $= \dfrac{B \tau + W_R}{F_R}$ $W_R = 11.56$ K/ft.

— $\tau = C_0 + \sigma' \tan(\delta)$

  → $\delta = \dfrac{2}{3} \phi = \dfrac{2}{3}(32°) = 21.33°$

  → $\sigma' = \mu \sigma$

  $\mu = 0.55$

  $\sigma = F/A = \dfrac{11.56 \text{ K/ft}}{(8.4 ft)(1 ft)} = 1.38$ ksf/ft.

  $\sigma' = 0.55 (1.38 \text{ ksf/ft}) = 0.76$ ksf/ft.

  → $C_0 = 0$ No cohesion

  $\tau = 0 + \left[0.76 \text{ ksf/ft} (\tan(21.33)) \right] = \underline{0.3 \text{ ksf/ft}}$

— F.S. sliding

  $F.S._{sliding} = \dfrac{B\tau}{F_R} = \dfrac{(8.4 ft)(1 ft)(0.3 \text{ ksf/ft}) + 11.56 \text{ K/ft}}{4.37 \text{ K/ft}}$

  $\boxed{F.S._{sliding} = 3.2 > 1.5}$

Bearing Capacity Calculations
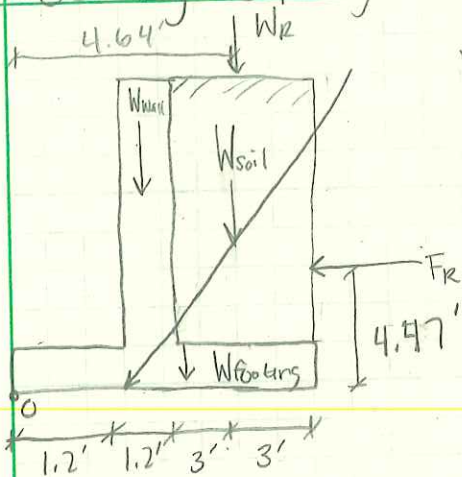
$$W_{wall} = W_1 = 1.944 \text{ K/ft}$$

$$W_{footing} = W_2 = 1.512 \text{ K/ft}$$

$$W_{soil} = W_3 = 8.1 \text{ K/ft}$$

$$F_R = 5.12 \text{ K/ft}$$

$$\Sigma F_x = F_R = -4.37 \text{ K/ft}$$

$$\Sigma F_y = W_R = 11.56 \text{ K/ft}$$

$$- R = \sqrt{(5.12)^2 + (11.56)^2} = 12.6 \text{ K/ft}$$

$$- \theta_x = \tan^{-1}\left(\frac{\Sigma F_y}{\Sigma F_x}\right) = \tan^{-1}\left(\frac{11.56}{-5.12}\right) = -66°$$

$$- \Sigma M_0 = (5.12 \text{ K/ft})(4.47 \text{ ft}) + (11.56 \text{ K/ft})(4.64 \text{ ft}) = 30.8 \text{ K-ft/ft}$$

$$\circlearrowleft +$$

$$- R\bar{a} = \Sigma M_0 \quad \therefore \quad \bar{a} = \frac{\Sigma M_0}{R} = \frac{30.8 \text{ K-ft/ft}}{12.6 \text{ K/ft}} = 2.44 \text{ ft}$$

$$- OE = \frac{\Sigma M_0}{\Sigma V} = \frac{30.8 \text{ K-ft/ft}}{11.56 \text{ K/ft}} = 2.66 \text{ ft}$$

$$- e = B/2 - OE = 8.4/2 - 2.66 = \underline{1.54} > 1.4 \qquad \times$$

$\rightarrow$ increase B

$$- B' = B - 2e = 8.4 - 2(1.54) = 5.32 \text{ ft}$$

$$- q_i = \left(\frac{W_F + P}{A_{foot}} - u_D\right)\left(1 \pm 6\frac{e}{B'}\right) = \left(\frac{11.56 \text{ K/ft}}{(8.4 \text{ ft})(1 \text{ ft})} - 0\right)\left(1 \pm 6\frac{1.54 \text{ ft}}{5.32 \text{ ft}}\right)$$

$$\boxed{q_{max} = 3.78 \text{ Ksf/ft}} \qquad \boxed{q_{min} = -1.01 \text{ Ksf/ft}}$$

Bearing Capacity Calculations Cont.

- $q_{ult} = c'N_c + \sigma_z' N_q + 0.5\gamma'B'N_\gamma$

$\gamma' = 125$ pcf

$c' = 0$

$\sigma_z' = 500$ psf

$B' = 5.32$ ft

$N_c = 44$

$N_q = 28.5$

$N_\gamma = 28.0$

$\Big\}$ From table 6.1 (Bearing Capacity Factors) for $\phi' = 32°$

- $q_{ult} = 0(44) + (500 \text{ lb/ft}^2)(28.5) + 0.5(125 \text{ lb/ft}^3)(5.32 \text{ ft})(28.0)$

$\therefore$ $\boxed{q_{ult} = 23.56 \text{ Ksf}}$

- Bearing Capacity $F.S = 3.0$

- $F.S = \dfrac{q_{ult}}{q_{allow}}$ $\therefore$ $q_a = \dfrac{q_{ult}}{F_S} = \dfrac{23.56}{3} = \underline{7.85 \text{ Ksf}}$

Max.

- $q_{allow} \gg q_{max}$ $\checkmark$

$F.S_{Bearing} = \dfrac{23.56}{7.85} = \underline{3}$

Bearing Capacity Calculations Cont.

Vesic's Method $\phi' = 32°$ $\gamma' = 125$ pcf $c' = 0$, $L = 100$ ft

$q_{ult} = c' N_c S_c d_c i_c b_c g_c + \sigma'_{zD} N_q S_q d_q i_q b_q g_q + 0.5 \gamma' B N_\gamma S_\gamma d_\gamma i_\gamma b_\gamma g_\gamma$

- $S_c = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) = 1 + \left(8.4 \text{ ft}/100 \text{ ft}\right)\left(\frac{23.2}{35.5}\right) = \underline{1.05}$

- $S_q = 1 + \left(\frac{B}{L}\right)\tan\phi' = 1 + \left(8.4 \text{ ft}/100 \text{ ft}\right)\tan 32° = \underline{1.05}$

- $S_\gamma = 1 - 0.4\left(B/L\right) = 1 - 0.4 \left(8.4'/100'\right) = \underline{0.97}$

- $d_c = 1 + 0.4K = 1 + 0.4(0.48) = \underline{1.19}$

- $d_q = 1 + 2K\tan\phi'(1-\sin\phi)^2 = 1 + 2(0.48)\tan(32°)(1-\sin(32))^2 = \underline{1.13}$

- $d_\gamma = 1$

- $K = D/B = 4 \text{ ft}/8.4 \text{ ft} = 0.48$

- $i_c = 1$ (The load acts perpendicular to the footing)

- $i_q = 1$

- $i_\gamma = 1$

- $b_c = 1$ (Base of the footing is level)

- $b_q = b_\gamma = 1$

- $g_c = g_q = g_\gamma = 1$ (Ground surface is level)

- $q_{ult} = 0 + (500 \text{ lb/ft}^2)(23.2)(1.05)(1.13)(1)(1)(1) + 0.5(125 \text{ lb/ft}^3)(8.4 \text{ ft})(30.2)(0.97)(1)$

∴ $\boxed{q_{ult} = 29 \text{ Ksf}}$

Internal stability Calculations

Stem thickness and reinforcement

- Nominal shear capacity [ACI 11.3.1.1, 11.3.2.1]

$$V_n/b = 2 \, bw \, d \sqrt{f'c}$$

bw = width of shear surface = 12 in/ft	
d = effective depth	
$f'c$ = 28-day Compressive strength of Concrete	

→ $V_n/b = 2(12 \text{ in/ft}) d \sqrt{4000 \text{ lb/in}^2}$

$ = 1518 d \text{ lb/ft}$

- Factored shear force per unit length of wall

$$V_u/b \leq 0.5\phi \, V_n/b \qquad \boxed{\phi = \text{resistance factor} = 0.85}$$

→ $V_u/b = W_r/b = 11,560 \text{ lb/ft} \leq 0.5 (0.85)(1518 d \text{ lb/ft})$

$$\therefore \quad \underline{d \geq 17.9 \text{ in}}$$

- Stem thickness: $T \geq d + d_b + Cover = 17.9 \text{ in} + 0.5 \text{ in} + 3 \text{ in} = 21.4 \text{ in}$

$$\therefore \quad \boxed{\begin{array}{l} T = 22 \text{ in} \\ d = 18.5 \text{ in} \end{array}} \text{ (increase stem width)}$$

- Required steel area per unit length of wall [ACI 11.7.4.1]

$$A_{vf}/b = \frac{V_u/b}{\phi \, f_y \, \mu}$$

f_y = yield strength of steel	
μ = coefficient of friction	

→ $A_{vf}/b = \dfrac{11,560 \text{ lb/ft}}{(0.85)(60,000 \text{ lb/in}^2)(0.55)} = \underline{0.412 \text{ in}^2/\text{ft}}$

- $A_s/b = \left(\dfrac{f'c \, b}{1.176 \, f_y}\right)\left(d - \sqrt{d^2 - \dfrac{2.353 \, M_u/b}{\phi \, f'c \, b}}\right)$

- $M_u/b = (W_r/b)(h)(\mu) = (11,560 \text{ lb/ft})(10.8 \text{ ft})(0.55)(12 \text{ in/ft}) = 824000 \dfrac{\text{in-lb}}{\text{ft}}$

Internal Stability Calcs Cont.

$$A_s/b = \left(\frac{(4,000 \text{ lb/in}^2)(12 \text{ in})}{1.176 (60000 \text{ lb/in}^2)}\right)\left(18.5 \text{ in} - \sqrt{(18.5 \text{ in})^2 - \frac{2,353 (824000)}{(0.9)(4000 \text{ lb/in}^2)(12 \text{ in})}}\right)$$

$$= (0.68 \text{ in})(18.5 \text{ in} - 17.244 \text{ in}) = \underline{0.854 \text{ in}^2/\text{ft}}$$

$$\rho = \frac{A_s}{bd} = \frac{0.854}{(12)(18.5)} = \underline{0.0038}$$

$\therefore A_s = A_{vf}/b + A_s/b = 0.412 + 0.854 = \underline{1.27 \text{ in}^2/\text{ft}}$

$\boxed{\text{Use } \#10 \text{ bars @ 9in O.C}}$ (Vertical steel Area)

$A_s/b = \left(\frac{1.27 \text{ in}^2}{9 \text{ in}}\right)(12 \text{ in}/1\text{ft}) = \underline{1.69 \text{ in}^2/\text{ft}} > 1.27 \text{ in}^2/\text{ft}$ ✓

$$\rho = \frac{A_s}{bd} = \frac{1.69 \text{ in}^2}{(18.5)(12)} = \underline{0.0076}$$

Longitudinal steel: $A_s = 0.0020 \, A_g = 0.002 \, (10.8 \text{ ft} \times 1.2 \text{ ft})(144 \text{ in}^2/\text{ft}^2)$

→ $A_s = 3.73 \text{ in}^2$ $\boxed{\text{Use } 9 \, \#6 \text{ bars} \quad (A_s = 3.96 \text{ in}^2)}$

Footing Reinforcement

- Minimum required footing thickness: $T \geq l_{dh} + 3 \text{ in}$

- required development length $l_{dh} = \dfrac{1200 \, d_b}{\sqrt{f'_c}}$ $\boxed{d_b = \text{bar diameter}}$

→ Use 0.7 modification factor for > 2in cover beyond end of hook

$$\overset{\#7}{l_{dh}} = \frac{1200 \, (0.875 \text{in})}{\sqrt{4000}} \times 0.7 = \underline{11.6 \text{ in}}$$

- $T \geq l_{dh} + 3 \text{in} = 11.6 + 3 = 14.6 \text{ in}$ → use $\underline{T = 15 \text{ in}}$

Internal stability Calcs Cont.

Heel extension; $W_{soil}/b = 8100$ lb/ft , $W_{footing}/b = 1512$ lb/ft
Shear Analysis

$V_u/b = 1.4D = 1.4(8100 + 1512) = \underline{13,457 \text{ lb/ft}}$

$V_n/b = 2b_w d\sqrt{f'_c} = 2(12\text{ in})d\sqrt{4000 \text{ lb/in}^2} = 1518d$

$\emptyset V_n/b = (0.85)(1518d) = 1290 d \text{ lb/ft}$

$V_u/b \leq \emptyset V_n/b \quad \therefore \quad d = \dfrac{13457}{1290} = 10.43 \text{ in} \rightarrow \underline{d = 10.5 \text{ in}}$

- Assume #8 bars ($d_b = 1.00$ in)

- $T \geq d + \dfrac{d_b}{2} + 3\text{in} = 10.5\text{ in} + \dfrac{1.0}{2} + 3 = \underline{14\text{ in}} < 15\text{ in} \quad \checkmark$

Heel extension - flexure:

$M_u/b = 1.4(8100 + 1512)\left(\dfrac{7.2 \text{ ft}}{2}\right)(12 \text{ in/ft}) = \underline{581,333 \text{ in-lb/ft}}$

$A_s/b = \left(\dfrac{f'_c b}{1.176 f_y}\right)\left(d - \sqrt{d^2 - \dfrac{2.353 M_u/b}{\emptyset f'_c b}}\right) = \left(\dfrac{(4000)(12)}{1.176(60,000)}\right)\left(10.5 - \sqrt{(10.5)^2 - \dfrac{2.353(581,333)}{(0.9)(4000)(12)}}\right)$

$\therefore \underline{A_s/b = 1.11 \text{ in}^2/\text{ft}}$

- min $A_s/b = 0.0018 A_g = 0.0018 \overset{T}{(15\text{in})} \overset{b}{(12\text{ in})} = \underline{0.324 \text{ in}^2/\text{ft}}$

- $\boxed{\text{Use } \#9 \text{ bars @ 9in O.C}} \quad (A_s/b = 1.33 \text{ in}^2/\text{ft})$

- Longitudinal steel: $A_s = 0.0018 A_g = (0.0018)\overset{T}{(15\text{ in})}(7.2\text{ ft})(12 \text{ in/ft})$

$\therefore \underline{A_s = 2.33 \text{ in}^2/\text{ft}}$

$\boxed{\text{Use } 13 \text{ } \#4 \text{ bars } (A_s = 2.60 \text{ in}^2)}$

2.0'

Vertical steel: #10 @ 9in O.C

Longitudinal steel: 9 #6 bars

#9 @ 9in O.C

13 #4 bars

10.8'

1.3'

standard 90° Hook

9.2'

A
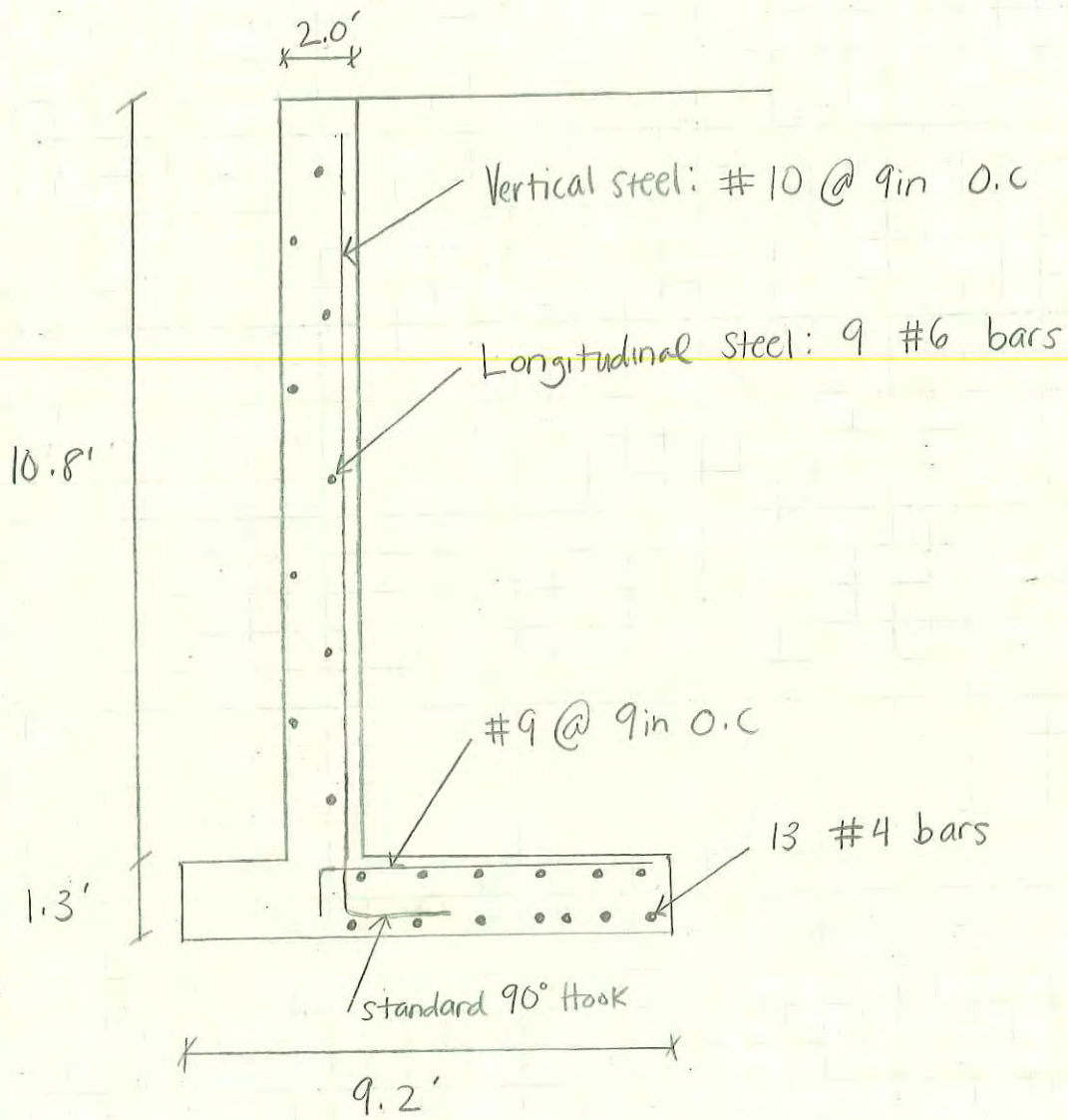
The Cantilever Retaining Wall rests on the soil at a depth of 3-7 ft.

- The SPT N value is 21 based upon the Table above.

- $N_{60} = \dfrac{E_m C_B C_s C_R N}{0.6}$ $\rightarrow$ $\left.\begin{array}{l} E_m = 0.6 \\ C_B = 1.0 \\ C_s = 1.0 \\ C_R = 0.75 \\ N = 21 \end{array}\right\}$ Based on Tables 4.3 & 4.4 in Coduto Textbook.

$N_{60} = \dfrac{(0.6)(1)(1)(0.75)(21)}{0.6}$

$\boxed{N_{60} = 15}$

- Corrected N_{60}

$(N_1)_{60} = N_{60} \sqrt{\dfrac{2000\ psf}{\sigma'_z}}$ $\rightarrow$ $N_{60} = 15$
$\sigma'_z = 1750\ psf$

$(N_1)_{60} = 15 \sqrt{\dfrac{2000\ psf}{1750\ psf}}$

$\boxed{(N_1)_{60} = 17}$

Settlement

- Equivalent modulus of Elasticity, $E_s = \beta_o \sqrt{OCR} + \beta_1 N_{60}$

$OCR = 1$

Table 4.1 Coduto — For Silty Sands: $\beta_o = 50,000\ psf$
$\beta_1 = 12,000\ psf$

$\therefore E_s = 50,000\ psf \sqrt{1} + (12,000\ psf)(15)$

$\therefore \underline{E_s = 230\ Ksf}$

- Peak Value of Strain Influence factor, $I_{\varepsilon P} = 0.5 + 0.1 \sqrt{\dfrac{2 - \sigma'_{zo}}{\sigma'_{zP}}}$

$q = 3780\ \ psf$

$\sigma'_{zo} = 500\ psf$

$\sigma'_{zP} = \gamma(D + B) = (125\ pcf)(4' + 8.4') = 1550\ psf$

- $I_{\varepsilon P} = 0.5 + 0.1 \sqrt{\dfrac{3780 - 500}{1550}} = \underline{0.64}$

- $I_{\varepsilon} = 0.2 + (z_f/B)(I_{\varepsilon P} - 0.2) = 0.2 + (3.6'/8.4')(0.64 - 0.2) = \underline{0.39}$

- Depth Factor, $C_1 = 1 - 0.5 \left(\frac{\sigma'_{zD}}{q - \sigma'_{zD}} \right) = 1 - 0.5 \left(\frac{500}{3780 - 500} \right)$

$$\underline{C_1 = 0.92}$$

- Secondary Creep factor, $C_2 = 1 + 0.2 \log \left(\frac{t}{0.1} \right)$ $t = 50 \text{ yr}$

$$\underline{C_2 = 1.54}$$

- Shape factor, $C_3 = 1.03 - 0.03 \frac{L}{B} \geq 0.73$

$$C_3 = 1.03 - 0.03 \, (75/8.4') = \underline{0.76}$$

Use L = 75'

$$\delta = C_1 C_2 C_3 \, (q - \sigma'_{zo}) \sum \frac{I_\varepsilon H}{E_s} \quad \ast \text{Thickness of soil layer}$$

on the spread sheet on the next page, the I_ε, H, & E_s for each soil layer are tabulated & $\sum \frac{I_\varepsilon H}{E_s}$ is given as $9.007 \ast 10^{-5}$

$$\delta = (0.92378)(1.53979)(0.762143)(3780 \, psf - 500 \, psf)(9 \ast 10^{-5})$$

$$\boxed{\delta = 3.84 \text{ in}}$$

Cantilever Retaining Wall Cost Analysis

- Based off of an example from infrastructurecost.com
  - Excavation & Backfill Cost
    - Quantity (yrd^3) = $75ft(16ft)(11.5ft)$ = $13800 ft^3 \left(\frac{1/3 yrd}{1 ft}\right)^3$ = $511.11 \, yrd^3$

    - Labor (6.32) = $511.11 \, yrd^3 (6.32 \, \$/yrd^3)$ = $\$3231$
    - Equipment (5.62) = $511.11 \, yrd^3 (5.62 \, \$/yrd^3)$ = $\$2873$
    - Job Materials (0.11) = $511.11 \, yrd^3 (0.11 \, \$/yrd^3)$ = $\$57$
    - Permanent Materials (6.17) = $511.11 \, yrd^3 (6.17 \, \$/yrd^3)$ = $\$3154$

      Total = $\$9,315$

- Construction Cost
    - Quantity (yrd^2) = $75ft(16ft + 11.5ft)$ = $2062.5 ft^2 \left(\frac{1/3 yrd}{1 ft}\right)^2$ = $225.83 \, yrd^2$

    - Labor (160.51) = $225.83 \, yrd^2 (160.51 \, \$/yrd^2)$ = $\$36248$
    - Equipment (91.68) = $225.83 \, yrd^2 (91.68 \, \$/yrd^2)$ = $\$20705$
    - Job Materials (2.18) = $225.83 \, yrd^2 (2.81 \, \$/yrd^2)$ = $\$635$
    - Permanent Materials (641.51) = $225.83 \, yrd^2 (641.51 \, \$/yrd^2)$ = $\$144873$

      Total = $\$202,461$

- Total Cost for Cantilever Retaining Wall

  Cost = $\$211,776$

  Use Cost = $\$220,000$

Excel Spreadsheets

	A	B	C	D	E	F	G	H	I	J
1	Cantilever Retaining Wall Design Calculations Including Surcharge Laterally									
2	For the At-Rest Condition									
3										
4	**Assumptions**			**Lateral Forces (lb/ft)**			**Moment Arm (ft)**		**Moments (kip-ft/ft)**	
5	Frictional Angle φ (degrees)	32		L.E.P. backfill(Fbfsoil)	7521.292		5.333333333		40.11355612	
6	Frictional Angle φ (Radians)	0.5585054		Surcharge (Fsc)	1805.11		8		14.4408802	
7	Unit Weight of Soil Υ (pcf)	125		Pore Water Pressure (Fw)	0		0		0	
8	Coefficient of friction μ	0.55								
9	Surcharge Load σ sc (psf)	240								
10	Unit Weight of Concrete (pcf)	150		**Vertical Forces (lb/ft)**			**Moment Arm (ft)**		**Moments (kip-ft/ft)**	
11				Weight of Soil (Wsoil)	8750		9		78.75	
12	**Dimensions**			Weight of Stem (Wstem)	5250		5.25		27.5625	
13	Total Height (ft)	16		Weight of Footing (Wfoot)	3450		5.75		19.8375	
14	Stem height (ft)	14		Weight of Surcharge (Wsc)	1200		9		10.8	
15	Footing height (ft)	2								
16	Toe length (ft)	4		**Resultant Forces&Moments**						
17	Heel length (ft)	5		Resultant Lateral Force Flr (	9.326402					
18	Stem length (ft)	2.5		Resultant Lateral Moment M	54.55444					
19	Footing length (ft)	11.5		Resultant Vertical Force Fvr	18.65					
20	Embedment depth (ft)	4		Resultant Vertical Moment	136.95					
21										
22	**Design Criteria**			**Resultant F&M locations from point O**						
23	Vertical Effective Stress σ'z (psf)	1750		Length of Flr horizontally h	5.849462					
24	Lateral Earth Pressures (L.E.P.):			Length of Fvr horizontally l	7.343164					
25	-At-Rest Ko	0.4700807								
26	-Active Ka	0.3072585								
27	-Passive Kp	3.2545883								
28				**Failure Mode**	**F.O.S.**					
29				Overturning	2.510337					
30				Sliding	2.429244					
31				Bearing Capacity	9.620261					
32										

Factor of Safety Calculation for Overturning failure

For the At-Rest Condition

Assumptions			**Resultant Forces&Moments**		
Frictional Angle φ (degrees)	32		Resultant Lateral Force Flr (k/ft)	9.326402	
Frictional Angle φ (Radians)	0.558505		Resultant Lateral Moment Mlr (k-ft/ft)	54.55444	
Unit Weight of Soil Υ (pcf)	125		Resultant Vertical Force Fvr (k/ft)	18.65	
Coefficient of friction μ	0.55		Resultant Vertical Moment Mvr (k-ft/ft)	136.95	
Surcharge Load σ sc (psf)	240				
Unit Weight of Concrete (pcf)	150				
Dimensions			**Factor of Safety (F.O.S.) Calculation**		
Total Height (ft)	16		Driving Moment Md (k-ft/ft)	54.55444	
Stem height (ft)	14		Resisting Moment Mr (k-ft/ft)	136.95	
Footing height (ft)	2				
Toe length (ft)	4		F.O.S. Overturning	2.510337	
Heel length (ft)	5				
Stem length (ft)	2.5				
Footing length (ft)	11.5				
Design Criteria					
Vertical Effective Stress σ'z (psf)	1750				
Lateral Earth Pressures (L.E.P.):					
-At-Rest Ko	0.470081				
-Active Ka	0.307259				
-Passive Kp	3.254588				

Factor of Safety Calculation for Sliding Failure
For the At-Rest Condition

	A	B	C	D	E
4	**Assumptions**			**Resultant Forces&Moments**	
5	Frictional Angle φ (degrees)	32		Resultant Lateral Force Flr (k/ft)	9.326402
6	Frictional Angle φ (Radians)	0.558505		Resultant Lateral Moment Mlr (k-ft/ft)	54.55444
7	Unit Weight of Soil Y (pcf)	125		Resultant Vertical Force Fvr (k/ft)	18.65
8	Coefficient of friction μ	0.55		Resultant Vertical Moment Mvr (k-ft/ft)	136.95
9	Surcharge Load σ sc (psf)	240			
10	Unit Weight of Concrete (pcf)	150		**Frictional Force Calculations**	
11	Interfacial Frictional Angle (degrees)	21.33333		Tao (ksf/ft)	0.348357
12	Interfacial Frictional Angle (radians)	0.372337		Vertical Stress (ksf/ft)	1.621739
13				Vertical Effective Stress (ksf/ft)	0.891957
14	**Dimensions**				
15	Total Height (ft)	16			
16	Stem height (ft)	14			
17	Footing height (ft)	2			
18	Toe length (ft)	4			
19	Heel length (ft)	5			
20	Stem length (ft)	2.5			
21	Footing length (ft)	11.5			
22					
23	**Design Criteria**			**Factor of Safety (F.O.S.) Calculation**	
24	Induced pressure (psf)	1750		Driving Force Fd (k/ft)	9.326402
25	Lateral Earth Pressures (L.E.P.):			Resisting Force Fr (k/ft)	22.65611
26	-At-Rest Ko	0.470081			
27	-Active Ka	0.307259		F.O.S. Sliding	2.429244
28	-Passive Kp	3.254588			
29					
30					

Factor of Safety Calculation for Bearing Capacity Failure
For the At-Rest Condition

	A	B		D	E		G	H
4	**Assumptions**			**Resultant Forces&Moments**			**Eccentricity Calculations**	
5	Frictional Angle φ (degrees)	32		Resultant Lateral Force Flr (k/ft)	9.326402		Combined Resultant (Flr&Fvr)Fr	20.85196
6	Frictional Angle φ (Radians)	0.558505		Resultant Lateral Moment Mlr (k-ft/ft)	54.55444		Angle of Fr (degrees)	28.64056
7	Unit Weight of Soil Υ (pcf)	125		Resultant Vertical Force Fvr (k/ft)	18.65		Sum of the Moments (k-ft/ft)	82.39556
8	Coefficient of friction μ	0.55		Resultant Vertical Moment Mvr (k-ft/ft)	136.95		Moment arm of Fr a (ft)	3.951454
9	Surcharge Load σ sc (psf)	240					Horizontal distance Fr is from O (ft)	4.417993
10	Unit Weight of Concrete (pcf)	150		**Resultant F&M locations from point O**			Eccentricity e (ft)	1.332007
11				Length of Flr horizontally hr (ft)	5.849462		B/6 Value (ft)	1.916667
12	**Dimensions**			Length of Fvr horizontally lr (ft)	7.343164			
13	Total Height (ft)	16						
14	Stem height (ft)	14		**Terzaghi's Method**			**Bearing Pressure Calculations**	
15	Footing height (ft)	2		Nc	44		Minimum Bearing Pressure qmin (ks	0.154895
16	Toe length (ft)	4		Nq	28.5		Maximum Bearing Pressure qmax (k	3.088583
17	Heel length (ft)	5		Ny	28		Ultimate Bearing Pressure qult (ksf)	29.71297
18	Stem length (ft)	2.5		Vertical Effective Stress σ'z (psf)	500			
19	Footing length (ft)	11.5		B' (ft)	8.835985		**F.O.S.**	9.620261
20				Unit Weight of Soil Υ (pcf)	125			
21	**Design Criteria**			Embedment Depth (ft)	4			
22	Vertical Effective Stress σ'z (psf)	500						
23	Lateral Earth Pressures (L.E.P.):							
24	-At-Rest Ko	0.470081						
25	-Active Ka	0.307259						
26	-Passive Kp	3.254588						
27								

Appendix C

MSE Wall Calculations

Step 1: Establish Project Requirements

$L_a = 3.6'$ $L_e = 6.4'$

$H = 12'$

$F_2 = 240$ psf

$F_1 = 2763$ psf

6"

4'

$L = 10'$

B
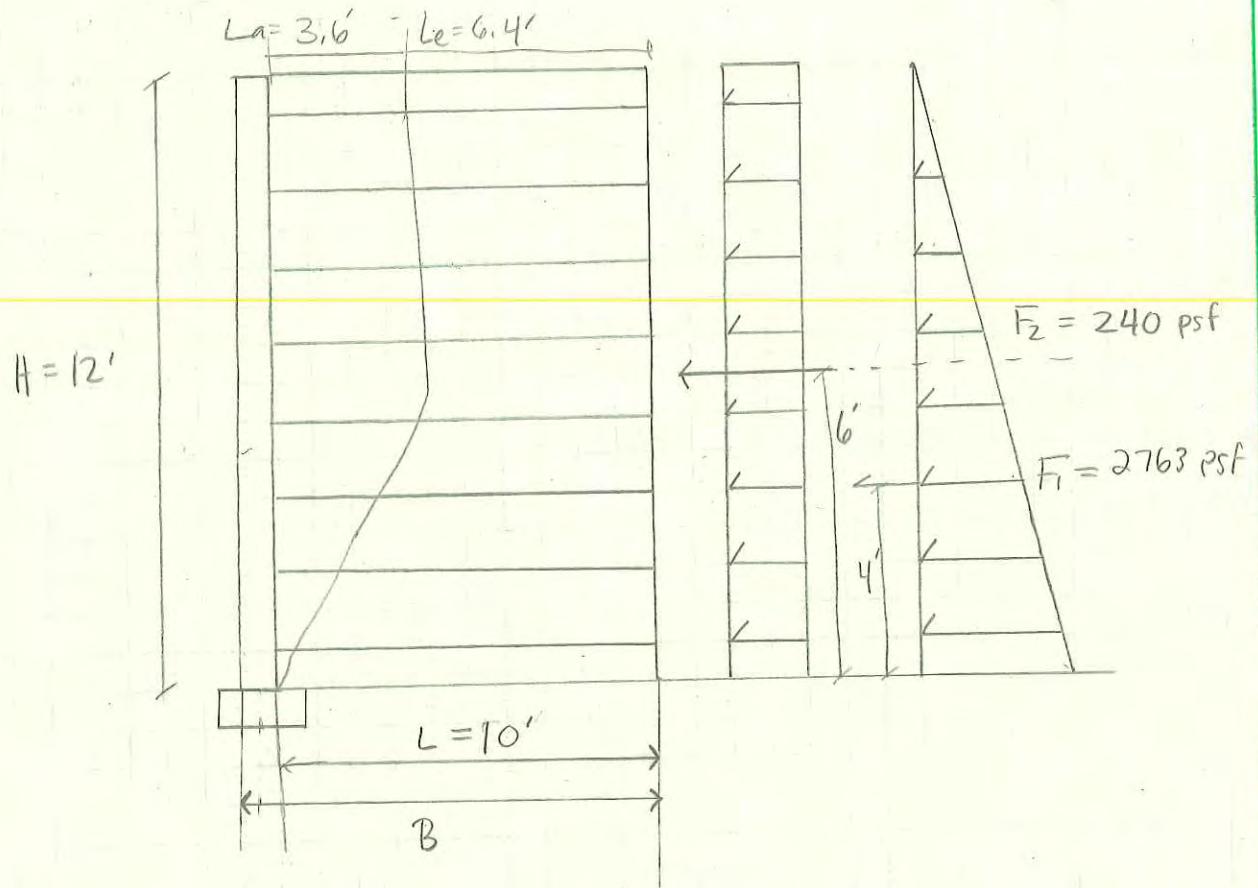
- Reinforcement length: $0.7H$

$\rightarrow 0.7H = 0.7(12\ ft) = \underline{8.4\ ft} \rightarrow$ Use $\underline{10\ ft = L}$

$\boxed{\emptyset'_b = \text{frictional angle of reinforced backfill}}$

- $K_{ab} = \tan^2\left(45 - \dfrac{\emptyset'_b}{2}\right)$

- $K_{ab} = \tan^2\left(45 - \dfrac{32°}{2}\right) = \underline{0.307}$

MSE Wall Calcs | Cont.

External Stability

• Sliding

- Nominal thrust per unit width: $F_1 = \frac{1}{2} K_{ab} \gamma_b H^2$

$\rightarrow F_1 = 0.5(0.307)(125 \text{ lb/ft}^3)(12 \text{ ft})^2 = \underline{2763 \text{ lb/ft}}$

- Surcharge load: $F_2 = K_{ab} q H = (0.307)(240 \text{ lb/ft}^2)(12 \text{ ft}) = \underline{884 \frac{lb}{ft}}$

- Resisting Force: $R_r = \phi_{EV} V_1 \mu \qquad \mu = 0.55$

- $V_1 = \gamma_r H L = (125 \text{ lb/ft}^3)(12 \text{ ft})(10 \text{ ft}) = \underline{15,000 \text{ lb/ft}}$

$\therefore R_r = (1.00)(15,000 \text{ lb/ft})(0.55) = \underline{8250 \text{ lb/ft}}$

- Driving force $P_d = \phi_{EH} F_1 + \phi_{LS} F_2 = 1.5(2763) + 1.5(884)$

$\therefore P_d = \underline{5471 \text{ lb/ft}}$

- $F.S_{sliding} = R_r / P_d = \dfrac{8250}{5471} = \underline{1.51} \checkmark$

• Overturning

$e = \dfrac{\phi_{EH} F_1 (H/3) + \phi_{LS} F_2 (H/2)}{\phi_{EV} V_1} = \dfrac{1.5(2763)(4) + 1.75(884)(6)}{1.0(15000)}$

$\therefore e = \underline{1.72 \text{ ft}} \leq e_{max} = L/4 = 2.5 \checkmark$

• Bearing Capacity

- Bearing capacity eccentricity, $e_b = \dfrac{\phi_{EH} F_1 (H/3) + \phi_{LS} F_2 (H/2)}{\phi_{EV} V_1 + \phi_{LS} q L}$

- $e_b = \dfrac{1.5(2763 \text{ lb/ft})(4 \text{ ft}) + 1.75(884 \text{ lb/ft})(6 \text{ ft})}{1.0(15000) + 1.75(240 \text{ lb/ft}^2)(10 \text{ ft})} = \underline{1.35 \text{ ft}}$

- Factored Vertical stress, $\sigma_{V-F} = \dfrac{\phi_{EV} V_1 + \phi_{LS} qL}{L - 2e_b}$

⁎ Assuming Meyerhof-type distribution

$\rightarrow \sigma_{V-F} = \dfrac{1.0 (15,000 \text{ lb/ft}) + 1.75 (240 \text{ lb/ft}^2)(10 \text{ ft})}{(10 \text{ ft}) - 2(1.35 \text{ ft})} = \underline{2630 \text{ lb/ft}^2}$

- Nominal Bearing resistance, $q_n = C_f N_c + 0.5 L' \gamma_f N_\gamma$

$\rightarrow$ for $\phi_f = 32°$; $N_c = 35.5$, $N_\gamma = 30.2$
(Table 10.6.3.1.2a-1 AASHTO 2007)

$\begin{array}{l} C_f = \text{Cohesion of foundation soil} \\ L' = \text{effective foundation width} \\ \quad = L - 2e_b \\ \gamma_f = \text{unit weight of foundation soil} \end{array}$

- $q_n = 0(35.5) + 0.5(10' - 2(1.35))(125 \text{ lb/ft}^3)(30.2)$

$\therefore q_n = \underline{13,779 \text{ lb/ft}^2}$

- Factored Bearing Resistance : $q_R = \phi q_n = 0.65(13,779) = \underline{8956 \text{ psf}}$

- $q_R \geq \sigma_{V-F}$ ✓

- $F.S_{Bearing} = \dfrac{8956}{2630} = \underline{3.4} > 3.0$ ✓

Internal Stability

- Extensible (Geosynthetic) Reinforcement

- MSE Wall with Level backfill and live load surcharge

$\rightarrow$ Horizontal stress : $\sigma_H = K_r \bar{\sigma}_V + \Delta \sigma_H$

$\begin{array}{l} K_r = K_{ab} = 0.307 \\ h_{eq} = 2 \text{ ft} \end{array}$

$\therefore \sigma_H = K_r (z + h_{eq}) \gamma_r \phi_{EV}$

$\sigma_H = 0.307 (12 \text{ ft} + 2 \text{ ft})(125 \text{ lb/ft}^3)(1.35)$

$\underline{\sigma_H = 725 \text{ lb/ft}^2}$

Labels on left (vertical):
S_1 | 12"
S_2 | 24"
S_3 | 24"
S_4 | 24"
S_5 | 24"
S_6 | 24"
S_7 | 12"

$L = 10'$
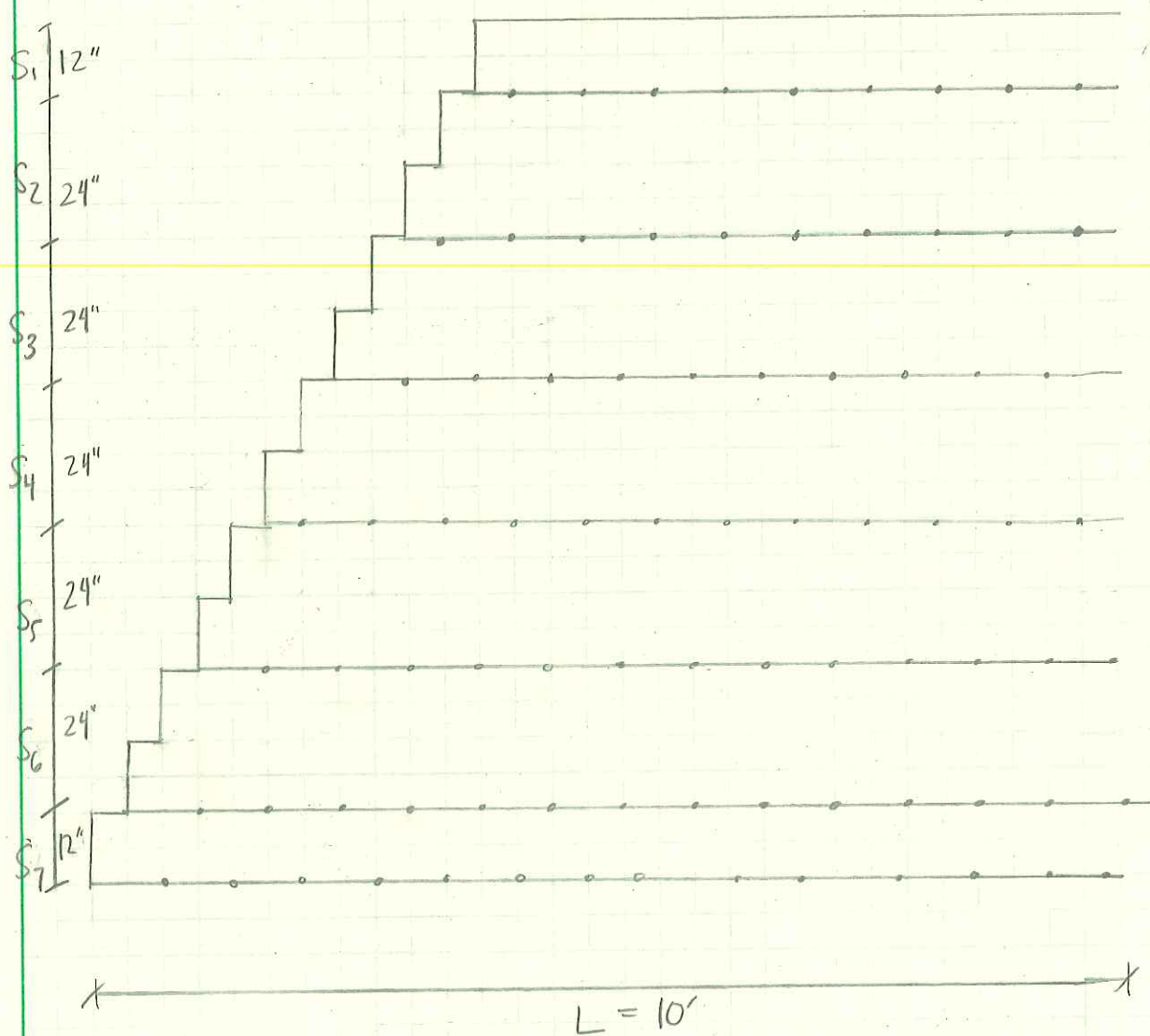
Maximum Tension, $T_{max} = \sigma_H S_v$ $\sigma_H = 0.307 (z_n + 2')(125 \, pcf)(1.35)$

$T_{max,1} = (155 \, lb/ft^2)(1 \, ft) = \underline{155 \, lb/ft}$

$T_{max,2} = (259 \, lb/ft^2)(2 \, ft) = \underline{518 \, lb/ft}$

$T_{max,3} = (362 \, lb/ft^2)(2 \, ft) = \underline{725 \, lb/ft}$

$T_{max,4} = (466 \, lb/ft^2)(2 \, ft) = \underline{932 \, lb/ft}$

$T_{max,5} = (569 \, lb/ft^2)(2 \, ft) = \underline{1139 \, lb/ft}$

$T_{max,6} = (673 \, lb/ft^2)(2 \, ft) = \underline{1346 \, lb/ft}$

$T_{max,7} = (725 \, lb/ft^2)(1 \, ft) = \underline{725 \, lb/ft}$

MSE Wall Calcs | Cont.

- Factored Tensile Resistance, $T_r = \phi T_{al}$

For TG-500 Geotextile grid
@ 6 in spacing

- $T_{al} = \dfrac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_D}$

$T_{ult} = 4.8 \; Ksf$

$\therefore T_{al} = \dfrac{4.8 \; Ksf}{(1.2)(1.6)(1.1)} = \underline{2.3 \; Ksf}$

$\rightarrow T_r = (0.9)(2.3 \; Ksf) = \underline{2.0 \; Ksf}$

- $T_{max} \leq T_r : 1.346 < 2.0$ ✓

- Pullout Failure : $\phi L_e \geq \dfrac{T_{max}}{F^* \alpha \sigma_v C R_c}$

- $F^* =$ Pullout resistance factor $= \frac{2}{3} \tan \phi = \underline{0.41}$

$\alpha =$ scale correction factor $= 1.0$

$\sigma_v =$ Nominal Vertical stress per layer of reinforcement $= \gamma_r Z_n$

$C = 2$

$R_c = 1.0$

$\phi = 0.9$

$L_e =$ length of embedment in Resisting zone

Layer	T_{max}	σ_v	ϕL_e (ft)	L_a (ft)	L (ft)
1	155	125	2.7	6.1	8.8
2	518	375	3.0	4.99	8.0
3	725	625	2.5	3.88	6.4
4	932	875	2.3	2.77	5.1
5	1139	1125	2.2	1.66	3.9
6	1346	1375	2.2	0.55	2.7
7	725	1500	1.1	0	1.1

$L_a = (H-z) \tan(45 - \phi/2)$

$L = L_a + L_e$

MSE Wall Cost

Construction Cost

- **Reinforcement** : Quantity = $(1 \text{ ft})(10 \text{ ft}) \times 7$ layers $= 70 \text{ ft}^2/\text{ft}$

  Total Reinforcement Area $= (70 \text{ ft}^2/\text{ft})(75 \text{ ft}) = \boxed{5250 \text{ ft}^2}$ of Reinforcement

- Geogrid Soil Reinforcement $4' \times 50' = 200 \text{ ft}^2/\text{unit}$

- Total Reinforcement Costs : $(5250 \text{ ft}^2)(0.79 \text{\$}/\text{ft}^2) = \boxed{\$4147.5}$

- **Masonary Block units** : 8" high $\times$ 18" wide $\times$ 20" deep

- Wall Face Area $= (12 \text{ ft})(75 \text{ ft}) = 900 \text{ ft}^2$

- 1 block unit $= (8 \text{ in} \times 1 \text{ ft}/12")(18 \text{ in} \times 1/12) = 1 \text{ ft}^2$ @ $\$19.55/\text{ft}^2$

- Total Cost of Masonary block units $= (900 \text{ ft}^2)(19.55 \text{\$}/\text{ft}^2)$

  $= \boxed{\$17,595}$

- **Gravel layer** $.5' \times 12' = (6 \text{ ft}^2/\text{ft})(75 \text{ ft}) = 450 \text{ ft}^2$

  Gravel fill $= \$2.08/\text{ft}^2$

  $\boxed{\text{Total} = \$936}$

Excavation + Backfill

Construction Costs :

Quantity $(\text{yd}^2) = 75 \text{ ft} \times (12 \text{ ft} + 10 \text{ ft}) \left(\frac{1/3 \text{ yd}}{1 \text{ ft}}\right)^2 = 183 \text{ yd}^2$ $(\$895.9 \text{\$}/\text{yd}^2)$

$= \boxed{\$163,946}$

Leveling Pad

Quantity $= (75 \text{ ft})(1 \text{ ft})(3 \text{ ft}) \left(\frac{1/3 \text{ yd}}{1 \text{ ft}}\right)^3 = \underline{8.33 \text{ yd}^3}$

$(1400 \text{\$}/\text{yd}^3)(8.33 \text{ yd}^3) = \boxed{\$11,667}$

$\boxed{\text{Total MSE Wall Cost} = \$200,000}$

Appendix D

Double Tee Beam Selection

$LL = \underbrace{25 \text{ psf}}_{\text{construction}} + \quad \quad + \underbrace{100 \text{ psf}}_{\text{Wet Concrete slab}} \quad \quad LL = 125 \text{ psf}$

$1.6(125) = 200 \text{ psf}$

Based on PCI Tables $\longrightarrow$ 8' × 32" Normal Weight
un topped

Span = 50'   safe load = 199 psf

8DT32

Not to Scale

32"   7¾"   7¾"   ↕2"

4¾"   4¾"

2'-0"   4'-0"   2'-0"
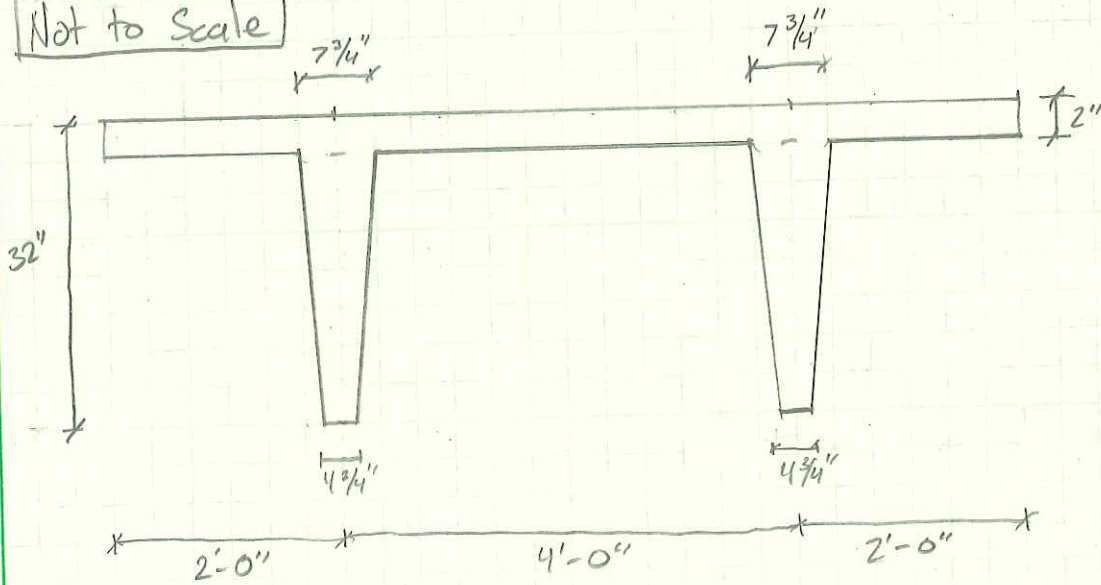
Section Properties

$A = 567 \text{ in}^2$ $f'_c = 5000 \text{ psi}$

$I = 55,464 \text{ in}^4$ $f_{pu} = 270,000 \text{ ksi}$

$y_b = 21.21 \text{ in}$ $y_e = 8.00 \text{ in}$

$y_t = 10.79 \text{ in}$ $y_c = 8.00 \text{ in}$

$S_b = 2,615 \text{ in}^3$ $r^2 = I_c / A_g = \frac{55464 \text{ in}^4}{567 \text{ in}^2} = 97.82 \text{ in}^2$

$S_t = 5,140 \text{ in}^3$

$W_t = 591 \text{ plf}$
or
74 psf

Strand orientation

148-S

14 ½" strands

Straight

10.79   C.G.   ↕8"

2"   21.21"
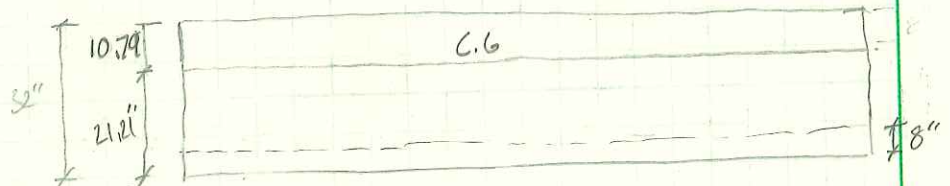
$V/s = 1.79 \text{ in}$ camber at erection = 1.3
                            long-time = 1.7

$f'_{ci} = 3500 \, psi$

Loads

$f_{ci} = 2100 \, psi$

$168 \, psf \longrightarrow 1.2 \, (lane + wet \, concrete + construction)$

$f_{pi} = 202.5 \, Ksi$

$168 \, psf * 8'$

$f_{py} = 243 \, Ksi \qquad f_{pe} = 162 \, Ksi \qquad 1344 \, plf \longrightarrow unshored \, construction$

$M_{max} = \dfrac{1.344 \, plf * 50'^2}{8} \qquad M_{max} = 420 \, K\text{-}ft$

$W_u = 1.2 \, (100 \, psf \times 8 \, ft) + 1.2(591 \, plf) + 1.6 \left(640 \, \tfrac{lb}{ft}\right) + 1.6 \, (25 \, psf \times 8')$

$W_u = 3.01 \, \tfrac{K}{ft}$

$M_u = \dfrac{3.01 \, \tfrac{K}{ft} \, (50')^2}{8} \qquad M_u = 940.625 \, K\text{-}ft$

$e = 8'' \qquad\qquad M_D = \dfrac{(591 \, plf)(50')^2}{8} \qquad\qquad M_D = 2216250 \, in\text{-}lb$

unshored live load

$M_{LL} = \dfrac{(25 \, psf)(50')^2}{8} \qquad\qquad M_{LL} = 5208.33 \, in\text{-}lb$

$M_{SD} = \dfrac{(100 \, psf \times 8')(50')^2}{8} \qquad M_{SD} = 3000000 \, in\text{-}lb$

required section Modulus

$\min S^t \geq \dfrac{(1 - .8)(2216250 \, in\text{-}lb) + .5208.33 \, in\text{-}lb + 3000000 \, in\text{-}lb}{.8 \, (212 \, psi) + 2000 \, psi}$

$f_c = 2000 \, psi \qquad f_{ti} = 3\sqrt{f'_c} = 212 \, psi \qquad f_t = 424 \, psi$

$S^t \geq 1405.58 \, in^3$

$\min S_b \geq \dfrac{(1 - .8)(2216250 \, in\text{-}lb) + 5208.33 \, in\text{-}lb + 3000000 \, in\text{-}lb}{424 \, psi - .8(-2.200 \, psi)}$

$\min S_b \geq 1396.3 \, in^3$

$148S - 8DT32$ checks out ✓

includes beam already ✓

449,58

Stresses at transfer

$$M_T = 2216250 \text{ in-lb} + 206000 \text{ in-lb} + 3000000 \text{ in-lb}$$

$$M_T = 3421650 \text{ in-lb}$$

$$A_{ps} = .153 \text{ in}^2 (14) \qquad A_{ps} = 2.142 \text{ in}^2$$

$$P_i = A_{ps} f_{pi} = 2.142 \text{ in}^2 (202.5 \text{ Ksi}) \qquad P_i = 433.76 \text{ kips}$$

$$P_e = .8 P_i = .8(433,755 \text{ lbs}) \qquad P_e = 347004 \text{ lbs}$$

Fiber stresses:

$$f^t = -\frac{P_i}{A_c}\left(1 - \frac{e_c C_t}{r^2}\right) - \frac{M_D}{S^t}$$

$$f^t = \frac{433.76^k}{-567 \text{ in}^2}\left(1 - \frac{8''(10.74'')}{97.82 \text{ in}^2}\right) \qquad f_t = -84.9 \text{ lbs (c)}$$

$$f_b = \frac{-433.76^k}{567 \text{ in}^2}\left(1 + \frac{8''(21.21'')}{97.82 \text{ in}}\right) \qquad f_b = -2092 \quad \text{(c)}$$

Dead Load Stress

$$f_t = \frac{M_D}{S_t} \qquad f_t = \frac{2216250 \text{ lb-in}}{5.140 \text{ in}^3} \qquad f_t = 431.2 \quad c$$

$$f_b = \frac{M_D}{S_b} \qquad f_t = \frac{2216250 \text{ lb-in}}{2615 \text{ in}^3} \qquad f_b = 847.5$$

@ Trasfer	Top	Bottom	Total
prestress	-84.9 lbs	-2059.32 lbs	Top = -521.1 lbs (c)
Dead	431.2 lbs	847.5 lbs	Bottom = -1211.82 (c)

@ Service $\qquad \gamma = \dfrac{f_{pe}}{f_{pi}} = \dfrac{162 \text{ Ksi}}{202.5 \text{ Ksi}} \qquad \gamma = .8$

$$f_t = \frac{-P_e}{A_c}\left(1 - \frac{e c_t}{r^2}\right) - \frac{M_D + M_{LL} + M_{SD}}{S_t}$$

$$f_b = \frac{-P_e}{A_c}\left(1 + \frac{e c_b}{r^2}\right) + \frac{M_D + M_{LL} + M_{SD}}{S_b}$$

$$f_t = -1087.8 \text{ lbs (c)} \qquad f_b = 216.3 \text{ (T) lbs}$$

MQP

Composite Structure

$$\frac{E_c \text{ (topping)}}{E_c \text{ (precast)}} = \frac{57,000\sqrt{3,000}}{57,000\sqrt{5000}} = .77$$

effective flange width = $8' = 96''$

modified flange width = $.77 \times 96'' = 73.92''$

$$C_b = \frac{(8'' \times 73.92'')(36'') + (567 in^2 \times 21.21 in)}{(8'')(65) + 560}$$

$$C_b = 30.85 in$$

$$I'_c = 55464 in^4 + 567(30.85 - 21.21)^2 + \frac{73.92''(8'')^3}{12}$$

$$+ \ 73.92 \times 8(5.15)^2 \qquad\qquad I'_c = 627993.35 in^4$$

$$r^2 = \frac{I'_c}{A} = \frac{627993.35 in^4}{567 + (8'' \times 73.92)} \qquad\qquad r^2 = 542.14 in^2$$

$$S_{cb} = \frac{I'_c}{Y_b} \qquad S_{cb} = 20356.35 in^3$$

$$S^t_c = 546081.14 \qquad\qquad C^t = 1.15 in$$

slab top $C^t = 1.15 in + 8'' = 9.15 in$

$$S^t_{cs} = \frac{627993.35 in^4}{9.15 in} \qquad\qquad S^t_{cs} = 68633.15 in^3$$

$$S_{cbs} = \frac{627993.35 in^4}{9.15 in - 2''} \qquad\qquad S_{cbs} = 87831.24 in^3$$

Stresses @ midspan $\qquad$ $W_T = 2231 \text{ lb/ft}$

$$f^t = -\frac{347004 \text{ lbs}}{567 \text{ in}^2}\left(1 - \frac{8''(10.79'')}{97.82 \text{ in}^2}\right) - \frac{8366250 + (72^k \cdot 29')}{546081 \text{ in}^3}$$

$$f^t = -612 \text{ psi}(.1176) - 61.2 \text{ psi} \qquad \boxed{f^t = -133.15 \text{ psi}}$$

$$f_b = -\frac{347004 \text{ lbs}}{567 \text{ in}^2}\left(1 + \frac{8''(21.21'')}{97.82 \text{ in}^2}\right) + \frac{8366250 \text{ in·lb} + (72^k \cdot 29')}{20356 \text{ in}^3}$$

$$f_b = -612 \text{ lbs}(2.7346) + 1641.88 \qquad \boxed{f_b = -31.70 \text{ psi}}$$

Appendix E

Composite Slab — unshored

Loads

$L = 50'$

$S = 88'' = 7.75'$

wet concrete = 145pcf + 10% = 106.33psf

maintenance/construction = 25psf

decking = 3psf

$$W_u = 1.6(106.33\,psf \times 7.75') + 1.6(25\,psf \times 7.75')$$
$$+ 1.2(3\,psf \times 7.75')$$
$$W_u = 1656.4\,plf$$

$$M_u = 517.6^{\,k\text{-}ft}$$

$$Z_x = 138\,in^3$$

$W24 \times 68 \longrightarrow Z_x = 177\,in^3$

$$W_u = 1656.4\,plf + 1.2(68plf) = 1738\,plf$$
$$M_u = 543.13^{\,k\text{-}ft}$$
$$Z_x = 144.83\,in^3 \checkmark$$

Deflection

$$\Delta_{max} = \frac{5\,W_T\,L^4}{384\,EI}$$

$W_T = 68\,plf + (106.33\,psf + 25\,psf + 3\,psf)\,7.75'$

$$W_T = 1109.06\,plf$$

$$\Delta max = \frac{1109.06\,plf\,(50')^4}{384\,(29000\,Ksi)(1830\,in^4)} \qquad \Delta max = .586$$

$$\frac{L}{1000} = .6'' \checkmark$$

Post-construction – unshored

$$W_u = 1.6(106\,psf + 25\,psf)\,7.75' + 1.2(3\,psf \times 7.75')$$
$$+ 1.2(68\,plf)$$

$$W_u = 1733.9 \; plf$$

$$M_u = 541.84 \; K\text{-}ft$$

$$b_e = 2 \times \tfrac{L}{8} \longrightarrow 12.5^{ft} \quad \underline{or} \quad \boxed{b_e = 2 \times \tfrac{S}{2} \longrightarrow 7.75'}$$

$$\text{Lower value governs}$$

$$a = \frac{50\,Ksi\,(23.73)}{.85(4\,Ksi)(88")} \qquad a = 3.966"$$

$$\phi M_n = .9(23.73")(50\,Ksi)(e = 17.882")$$

$$e = \frac{d}{2} + t_s - \frac{a}{2} \qquad e = 17.882"$$

$$\phi M_n = 19095.3 \; K\text{-}ft \; \geq \; 541.84 \; K\text{-}ft \; \checkmark$$

$$a = \frac{\Sigma Q_n}{.85\,f'_c\,b_e} \qquad a = \frac{1010^K}{.85(4\,Ksi)(88")} \qquad a = 3.38\,in$$

$$Y_2 = 8" - \frac{3.38"}{2} \quad Y_2 = 6.31" \qquad Y_2 = 6 \quad \phi M_n = 942^{K\text{-}ft}$$

$$@ \text{ location 7} \qquad Y_2 = 6.5 \quad \phi M_n = 951.^{K\text{-}ft}$$

$$\phi M_n = 942^{K\text{-}ft} + \frac{.31}{.5}(951^{K\text{-}ft} - 942^{K\text{-}ft})$$

$$\phi M_n = 948.2 \;^{ft\text{-}Kip} \; \geq \; 541.84 \;^{K\text{-}ft} \; \checkmark$$

I_c for W24×68 composite

$Y_2 = 6.31$ @ Location 7 $Y_2 = 6$ $I_c = 3110 \, in^4$

$Y_2 = 6.5$ $I_c = 3180 \, in^4$

$I_{LB} = 3110 \, in^4 + \left(\frac{.31}{.5}\right)\left(3180 \, in^4 - 3110 \, in^4\right)$

$$I_{LB} = 3153.4 \, in^4$$

Service Deflections

D + .5L

$W_T = (106 \, psf + 3 \, psf) \, 7.75' + \frac{640 \, plf}{2} + 68 \, plf$

$W_T = 1332.75 \, plf$

$\Delta_{max} = 2.049'' > \frac{L}{1000} = .6$ X   Fails

New beam ⟶ W40×167 $Z_x = 693 \, in^2$ $I_x = 11600 \, in^4$

$W_u = 1652.3 \, plf + 1.2 \, (167 \, plf)$

$W_u = 1852.7 \, plf$

$M_u = 578.97^{k-ft}$ $Z_x = 154.4 \, in^3 \le 693 \, in^3$ ✓

$W_T = 1208 \, plf$ $\Delta_{max} = .5'' \le .6''$ ✓

$b_e = 88''$ $a = 8.22''$ $\phi M_n = 51342.66 \, in\text{-}k$

$Y_2 = 3.87''$ $\phi M_n = 4278.6 \, k\text{-}ft$

ϕM_n @ TFL $= 4286.6^{k-ft}$

ϕM_n @ 7 $= 3598.8^{k-ft}$

I_{LB} @ 7 $= 18748 \, in^4$ $\Delta_{max} = .345 \, in \le .6''$ ✓

Shear Studs

$$Q_n = .5 A_{sa} \sqrt{f'_c E_c} \leq R_g R_p A_{sa} F_u$$

$$E_c = (145 pcf)^{1.5} \sqrt{4 ksi} \quad E_c = 3492 \qquad f'_c = 4 ksi$$

$$A_{sa} = \left(\frac{3/4}{4}\right)^2 \frac{\pi}{4} \quad A_{sa} = .442 \qquad R_g = 1.0 \quad R_p = .75$$

$$Q_n = .5(.442)\sqrt{4(3492)} \quad \text{or} \quad 1.0(.75)(.442)(65 ksi)$$

$$Q = 26.12^K \qquad \text{or} \quad \boxed{21.55^K}$$

$$N = \frac{(99.2 in^2)(50 ksi)}{21.55^K} \qquad N = 114.15 \longrightarrow 115 \text{ studs}$$

Service deflections

$$\text{unshored} \longrightarrow \frac{L}{1000} \qquad \frac{50' \times 12''}{1000} = .60'' \checkmark \qquad I_{LB}$$

LL service $\longrightarrow .6$

Total DL + LL $\longrightarrow .6$

$$W_T = 1125 \quad plf \longrightarrow \text{see previous page}$$

$$L = 50' \qquad I_{LB} = @ 7 \qquad Y_2 = 6.97'' \quad Y_2 = 6.5 \quad I_{LB} = 18200$$

$$Y_2 = 7 \quad I_{LB} = 18400$$

$$I_{LB} = 18200 in^4 + \frac{.37}{.5}(18400 - 18200)$$

$$I_{LB} = 18348 \ in^4$$

$$\Delta = \frac{5(1125 \ plf)\frac{1}{12''}(50' \times 12'')^4}{384(29000 ksi)(18348 \ in^4)} \qquad \Delta = .247'' \checkmark$$

Shear Studs

$$Q_n = .5 \, A_{sa} \sqrt{f'_c \, E_c} \leq R_g \, R_p \, A_{sa} \, F_u$$

$$E_c = (145 \, pcf)^{1.5} \sqrt{4 \, ksi} \qquad E_c = 3492 \qquad f'_c = 4 \, ksi$$

$$A_{sa} = \left(\frac{3''}{4}\right)^2 \frac{\pi}{4} \qquad A_{sa} = .442 \, in^2 \qquad R_g = 1.0 \qquad R_p = .75$$

$$Q_n = .5 \, (.442 \, in^2) \sqrt{4(3492)} \quad or \quad 1.0(.75)(.442 \, in^2)(65 \, ksi)$$

$$Q_n = 26.12^k \quad or \quad \boxed{21.55^k}$$

smaller value governs

$$N = \frac{49.2 \, in^2 (50 \, ksi)}{21.55^k} \qquad N = 114.15 \rightarrow 115 \text{ studs}$$

Stud Spacing

capacity = 21.55^k

shear transfer = $A_s \, F_y = 2460^k$

\# of studs = 115

AISC Spacing Limits

min $6d = 4.5''$

max $8 t_s = 64''$

spacing

$$\frac{100' \times 12''}{230 + 1} = 5.19''$$

$4.5'' = 6d \leq 5.19'' \leq 8 t_s = 64''$ ✓

Appendix F

Loading

Concrete = .150 Kcf

$f'_c = 4000$ ksi

$f_y = 60$ ksi

Reinforcing Steel cover req.

Backwall back = 2.5"

Stem back = 2.5"

Footing top = 2"

Footing bottom = 3"

Superstructure Data

S = 7.75'   Span = 100'

N = 6   Parapet height = 3'

      Parapet weight = .53 ^{k}/ft

W40 x 167

Superstructure dead $\longrightarrow$ $\dfrac{6 \times (167 \,plf \times 50')}{}$ = 50100 lbs

wingwall l = lwing wall $\longrightarrow$ 50'

abutment w = 44'

abutment h = 15'

wingwall h = 10'

$\dfrac{50,000 \, lbs}{44'} = 1.14'$

Backwall dead load = $\left[(1.25' \cdot 1.33') + (2.0 \times .76') + \dfrac{(2' \cdot 1.67') \cdot .33'}{2} + (1.67' \times 1.76')\right] \times 150 \, pcf$

$DL_{bw} = 1.71 \, ^k/ft$

Stem load   = 15' × 3.5' × W_c $DL_{stem} = 7.88 \, ^k/ft$

Footing DL   = 10.25' × 2.5' · W_c $DL_{ftg} = 3.84 \, ^k/ft$

Earth Dead Load = $(125 \, pcf = \gamma_s) \cdot 15' = 1.875 \, ^k/ft$
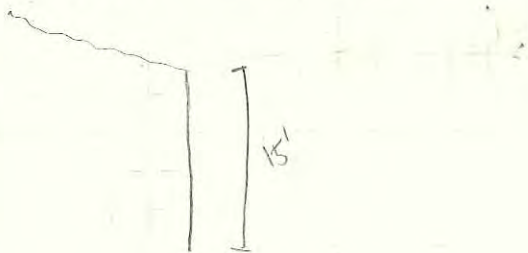
Dynamic Load allowance   IM = .33

multiple presence factors $m_1 = 1.20$ (1 Lane)

multiple presence factors $m_2 = 1.00$ (2 lanes)

multiple presence factors $m_3 = .85$ (3 lanes)

Backwall Live Load

$$R_{LLbw} = \frac{(6 \cdot 16^K \cdot (1 + IM) + 3 \cdot (.64 \, K/ft) \cdot 2.0')}{L_{abut}}$$

$$R_{LLbw} = 2.99 \, Klf$$

$V_{vehmax} =$ $V_{vehmin} =$

$V_{lanemax} =$ $V_{lanemin} =$

Other Loading

$h_{par} = 42 \, in$ $d_{web} = 38.6''$ $D_{tot} = \frac{93.13 \, in}{12''/ft} = 7.76'$

$t_{deck} = 8''$ $t_{bot \, flg} = 1.03''$

$t_{cslope} = 0 \, in$ $t_{haunch} = 3.5''$

$L = 60'$ $A_{wsuper} = D_{tot} * L_{wind}$ $A_{wsuper} = 388 \, ft^2$

Base wind $V = 100 \, mph$ $V_{DZ} = V_B$

Base wind pressure = $P_D = P_B \left(\frac{V_{DZ}}{V_B}\right)^2$ or $P_D = P_B \left(\frac{100 \, mph}{100 \, mph}\right)^2$

$$P_D = P_B$$

Wind total = $.05 \, Ksf \cdot D_{tot} = .05 \, Ksf \times 7.76' = .388 \, K/ft$

$$W_{s\,supertrans\,0} = A_{wsuper} \cdot .05 \, ksf$$
$$W_{s\,supertrans\,0} = 19.4^k$$

Wind load on abutment

$$A_{w\,sub(end)} = 3.5'(15') = 52.5 \, ft^2$$
$$W_{subtrans\,0} = 52.5 ft^2 (.04 \, ksf \cos(0))$$
$$W_{sub\,trans\,0} = 2.1^k$$

Earth Load

$$P = K_a \cdot y_s \cdot Z \qquad y_s = .125^{k}/cf \qquad K_a = .46$$
$$\qquad\qquad\qquad K_o = .5 \qquad K_p = 4.7$$
$$Z = 7'$$

$$P = .46 \cdot .125 \, kcf \times 7' \qquad P = .4025 \, ksf$$

Load acts at $\frac{h}{3}$ from bottom

$$R_{back\,wall} = \tfrac{1}{2} \cdot p \cdot Z \qquad R = \tfrac{1}{2}(.4025 \, ksf) \times 7' \qquad R_{chbw} = 1.41 \, klf$$

Bottom of stem lateral earth Load

$$K_a = .46 \qquad y_s = .125 \, kcf \qquad 7' + 15' = Z$$
$$P = K_a \times Z \times y_s \qquad P = 1.265 \, ksf$$

Lateral load at $\frac{h}{3}$

$$R_{stem} = \tfrac{1}{2} p \, h_{stem} = \tfrac{1}{2}(1.265 \, ksf)(22') \qquad P_{stem} = 13.915 \, k/s$$

Bottom of footing lateral earth load.

$$P = 1.41 \, kcf \qquad Z = 24.5' = 22' + 2.5'$$
$$R \cdot @ \, \frac{h}{3} \text{ of footing} \qquad R = 17.26 \, klf$$

Loads Due to temperature

Expansion $\varepsilon = 6.5 \times 10^{-6}$ in/in/°F   assumed set $T = 68°F$

moderate climate $-30° \longrightarrow 120°F$

$$\Delta_{exp} = \varepsilon \cdot \Delta_T \left(L_{span} \cdot 12 \tfrac{in}{ft} \right)$$

$$\Delta_{t\,rise} = 120 - t_{set} \qquad \Delta_{t\,rise} = 52°F$$

$$\Delta_{exp} = \varepsilon * \Delta_{t\,rise} \left(L_{span} \cdot 12 \tfrac{in}{ft} \right) \qquad \Delta_{exp} = .4056''$$

contraction $\Delta_{contr} = \varepsilon \cdot \Delta_t \cdot \left(L_{span} \times 12 \tfrac{in}{ft} \right)$

$$\Delta_{t\,fall} = t_{set} + 30° = 98°F$$

$$\Delta\,contraction = \left(6.5 \times 10^{-6} \right)^{in/in}/°F \, (98°F)(100' \times 12 \tfrac{in}{ft})$$

$$\Delta\,contraction = .7644''$$

$$H_u = G \cdot A \cdot \frac{\Delta}{h_{rt}}$$

assumed bearing properties $h_{rt} = 2''$

$L = 22'$

$G = .095 \, Ksi$

$A = 14in \times 15in \qquad h_{rt} = 3.5''$
$= 210.000 \, in^2$

$$H_{u\,rise} = G \cdot A \cdot \frac{\Delta_{exp}}{h_{rt}} \qquad H_{u\,rise} = .7644$$

$$H_{u\,rise\,total} = \frac{H_{u\,rise} \cdot 5}{L_{abut}} \qquad H_{u\,rise\,total} = \frac{2.77^k \cdot 5}{44'} = .315$$

$$H_{u\,fall} = 3.63^k$$

$$H_{u\,fall\,total} = \frac{H_{u\,fall} \cdot 5}{L_{abut}} \qquad H_{u\,fall\,total} = \frac{3.63^{k} \cdot 5}{44'} = .4125$$

$$D_{L\,bw} = 1.71 \, ^{k}/ft \qquad R_{EH\,bw} = 1.41 \, ^{k}/ft$$

$$R_{LL\,bw} = 2.98 \, ^{k}/ft \qquad R_{LS\,bw} = .91 \, ^{k}/ft$$

Dead load Effects

h_{ww} stem = 17'

Δl_{ww} stem = [3' × 17'] · W_c

Δl_{ww} stem = 7.65 ^{K}/ft

W_c = weight of concrete = 150 pcf

H_{ww} = Stem height

Deadload due to Superstructure = $\dfrac{50\,100 \text{ lbs}}{44'}$ = 1.14 klf

Lateral Earth Pressure

$P = Ka \cdot \gamma_s \cdot z$

$p = (Ka = .3) \cdot (\gamma_s = .125 \text{Ksf}) \cdot (20')$

$p = .75 \text{ Ksf}$

P = Earth Pressure
Ka = Earth Pressure coefficient
γ_s = Density of soil
Z = Wall height

Lateral Load due to Earth Pressure

H_{ww} = 20'

$R_{EH \text{ stem}} = \frac{1}{2} \cdot p \cdot H_{ww} = .5(.75\text{Ksf})(20') = 7.5$ ^{K}/ft @ $\dfrac{H_{ww}}{3}$

Loads due to Live Load Surcharge

$\Delta P = Ka \cdot \gamma_s \cdot h_{eq}$ $h_{eq} = 2'$

$\Delta P = .075 \text{ Ksf}$ $R_{ls \text{ stem}} = \Delta P \cdot H_{ww}$

$R_{ls \text{ stem}} = 1.5$ ^{K}/ft   Located at $\dfrac{H_{ww}}{2}$ from base

$R_{LL \text{ stem}} = 2.99 \text{ K/f}$ → according to FHWA located at $\dfrac{H_{ww}}{2}$

Design for flexure in the stem

- Assume #9 bars
- cracking strength

$d_{bar} = 1.128\,in$ (diameter of bar)
$A_{bar} = 1.0\,in^2$ (Area of bar)

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad f_r = .24\sqrt{f'_c}$$

$$M_{cr} = \frac{(.24\sqrt{4ksi})(46656\,in^4)}{18"} \qquad I_g = \frac{1}{12}(12")(36")^3$$

$$y_t = 18"$$

$$M_{cr} = 103.7 \frac{k\text{-}ft}{ft}$$

use 1.2 factor for M_{cr} to design abutment stem flexure reinforcement

$$1.2\,(M_{cr}) = 124.7 \frac{k\text{-}ft}{ft}$$

Effective depth of reinforcement

$t_{bw} = 36\,in$ (Thickness of stem)

cover = 2.5 in

$$d_e = t_{bw} - cover - \frac{d_{bar}}{2} = 36" - 2.5" - \frac{1.128\,in}{2}$$

$$d_e = 32.9\,in \longrightarrow d_e = 33\,in$$

required amount of steel reinforcement

$$\phi = .9 \quad b = 12\,in \quad f'_c = 4ksi \quad f_y = 60\,ksi$$

$$R_n = \frac{(1.2\,M_{cr}) \cdot 12\,in}{\phi \cdot b \cdot d_e^2} = \frac{(124.4\,\frac{k\text{-}ft}{ft})(12\,in/ft)}{(0.9)(12\,in)(33\,in)^2}$$

$$R_n = 0.13\,ksi$$

$$\rho = .85\frac{f'_c}{f_y}\left[1.0 - \sqrt{1.0 - \frac{2(R_n)}{.85\,f'_c}}\right.$$

$$\rho = .85\left(\frac{4ksi}{60\,ksi}\right)\left[1 - \sqrt{1 - \frac{2(.13\,ksi)}{.85(4\,ksi)}}\right. \qquad \rho = .00214$$

$$A_s = \rho \cdot \frac{b}{ft} \cdot d_e = .00214\left(\frac{12\,in}{ft}\right)(33\,in) \qquad A_s = .85\,in^2/ft$$

Required spacing $= \dfrac{A_{bar}}{A_s} = \dfrac{1.0 \, in^2}{.85 \, in^2/ft} = 14.1 \, in$

FHWA #9 bars at 9in spacing

$A_s = A_{bar} \left(\dfrac{12 \, in}{bar \, space} \right) = 1.0 \, in^2 \left(\dfrac{12 \, in}{9 \, in} \right)$ $A_s = 1.33 \, in^2/ft$

- Max reinforcement check

$T_s = A_s \cdot f_y = 1.33 \, in^2/ft \, (60 \, ksi)$ $T = 80 \, kip$

$a = \dfrac{T}{.85 f'_c b} = \dfrac{80^k}{.85 \, (4 \, ksi)(12 \, in)}$ $a = 1.96 \, in$

$c = \dfrac{a}{\beta_1} \longrightarrow \beta_1 = .85 - .05 \left(\dfrac{f'_c - 4000}{4000} \right)$

$\beta_1 = .85$

$c = \dfrac{1.96 \, in}{.85}$ $c = 2.31 \, in$

$\dfrac{c}{d_e} \le .42 \longrightarrow \dfrac{2.31 \, in}{33 \, in} \le .42$ $.07 \le .42 \checkmark \, ok$

- check crack control

$d_c = C_c + \dfrac{d_{bar}}{2}$ $C_c \rightarrow$ clear cover $> 2 \, in$

$d_c = 2 \, in + \dfrac{1.128 \, in}{2}$. $d_c = 2.56 \, in$

$A_c = 2 \cdot d_c \cdot bar \, space$ $A_c =$ concrete area

$A_c = 2 \, (2.56 \, in)(9 \, in)$ $A_c = 46.15 \, in^2$

$f_{sa} = \dfrac{Z}{(d_c \cdot A_c)^{1/3}}$ where $f_{sa} \le .6 f_y$ $f_{sa} =$ allowable reinforcement service load stress

$f_{sa} = \dfrac{130 \, k/in}{(2.56 in \cdot 46.15 in^2)^{1/3}}$ $Z = 130 \, k/in$ (salt exposure)

$f_{sa} = 26.5 \, ksi$ $.6 f_y = 36 \, ksi$

$26.5 \, ksi \le 36 \, ksi \checkmark$

$$n = \frac{E_s}{E_c} \qquad E_s = 24000 \text{ Ksi}$$

$$E_c = 3640 \text{ Ksi}$$

$$n = \frac{24000 \text{ Ksi}}{3640 \text{ Ksi}} \qquad n = 7.97 \longrightarrow 8''$$

$$M_{U \text{ stem service } \ell} = \left(R_{EH stem} \cdot \frac{H_{ww}}{3} \right) + \left(R_{es \, stem} \cdot \frac{H_{ww}}{2} \right)$$

$$= \left(7.5 \text{ k/ft} \cdot \frac{20'}{3} \text{ ft} \right) + \left(1.5 \text{ k/ft} + 2.44 \text{ k/ft} \right) \frac{20'}{2} \text{ ft}$$

$$M_{U \text{ stem service } \ell} = 94 \frac{K \cdot ft}{ft}$$

calculate f_s, actual stress in reinforcement

$$\rho = \frac{A_s}{\frac{b}{ft} \cdot d_e} = \frac{1.53 \text{ in}^2}{\frac{12 \text{ in}}{ft} \cdot 33 \text{ in}} \qquad \rho = .00336$$

$$K = \sqrt{(\rho \cdot n)^2 + (2 \cdot \rho \cdot n)} - (\rho \cdot n) = \sqrt{((8)(.00336))^2 + (2)(.00336) \, 8} - .00336(8)$$

$$K = .21$$

$$K \cdot d_e = .21 \cdot 33 \text{ in} = 6.93 \text{ in} \longrightarrow \text{N.A from front face of the abutment}$$

$$I_t = \frac{1}{3} \left(12 \text{ in/ft} \right) (K \cdot d_e)^3 + n \cdot A_s (d_e - K d_e)^2$$

$$I_t = \frac{1}{3} \left(12 \text{ in/ft} \right) (6.93 \text{ in})^3 + 8 (1.53 \text{ in}^2) \left[33 \text{ in} - 6.93 \text{ in} \right]^2$$

$$I_t = 8562.67 \text{ in}^4/ft$$

$$y = d_e - K \cdot d_e = 33 \text{ in} - 6.93 \text{ in} = 26.07 \text{ in}$$

$$f_s = \frac{n \left(M_{U \text{ stem service}} \cdot 12 \text{ in/ft} \cdot y \right)}{I_t} = \frac{8 \left(65 \frac{K \cdot ft}{ft} \right) (12 \text{ in/ft}) (26.07 \text{ in})}{8562.67 \text{ in}^4/ft}$$

$$f_s = 19 \text{ Ksi} \leq 26.5 \text{ Ksi} \checkmark$$

$$f_s \leq f_{sa}$$

Design for shear

$$V_{Uww\,stem} = R_{EH\,stem} + R_{LS\,stem} = 7.5\,{}^{k}/st + 1.5\,{}^{k}/st + 2.99\,{}^{k}/st + 1.14\,{}^{k}/st$$

$$V_{Uww\,stem} = 13.4\,{}^{k}/st$$

Nominal Shear Resistance

$$V_{n1} = V_c + V_s$$

$$V_{n2} = .25\,f'_c\,b_v\,d_v \qquad\qquad \text{lesser governs} \longrightarrow V_{n2}$$

$$V_c = .0816 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \cdot d_v \longrightarrow V_{n1}$$

* Per FHWA neglect V_s for this design

$$\beta = 2in$$

$$b_v = 12in$$

$$d_v = \max(d_e - {}^a/_2), (.9 \cdot d_e)(.72h)$$
$$\qquad\qquad\;① \qquad\quad ② \qquad ③$$

$$d_v = 32.02in$$

$$\boxed{① \; 35'' - \frac{1.96''}{2''} = 32.02in} \;\;\text{max}$$

$$② \; .9(33'') = 24.7in$$

$$③ \; .72(36'') = 25.92in$$

$$V_{n1} = 48.6\,{}^{k}/st \qquad\qquad V_{n2} = 384.24\,{}^{k}/st$$

$$V_n = V_{n1} = 48.6\,{}^{k}/st$$

$$\phi V_n = .9(48.6\,{}^{k}/st) \cdot \qquad \phi V_n = 43.7\,{}^{k}/st$$

$$\phi V_n \geq V_{U\,ww\,stem}$$
$$43.7\,{}^{k}/st \geq 13.4\,{}^{k}/st \;\checkmark$$

Temperature + Shrinkage

① $A_s \geq .11 \dfrac{A_g}{f_y}$

Use lesser value

② $\Sigma A_b = .0015 A_g$

$A_g = T_{bw} \cdot 12^{in}/_{ft} = 36\,in\,(12^{in}/_{ft})$ 　　　$A_g = 432^{in^2}/_{ft}$

① $.11 \dfrac{A_g}{f_y} = .8^{in^2}/_{ft}$ ⟶

② $.0015\,A_g = .65^{in^2}/_{ft}$ ⟶ 　$A_s \geq .65^{in^2}/_{ft}$

Try horizontal #5 bars for each face @ 9" spacing

$d_{bar} = .625\,in$

$A_{bar} = .31\,in^2$ 　　　$A_s = 2 \cdot \dfrac{A_{bar}}{2}\left(\dfrac{12in}{9in}\right) = .83^{in^2}/_{ft}$

$$.83\,in^2/_{ft} \geq .65^{in^2}/_{ft}$$

Footing Reinforcement

$T_{ft} \geq l_{dh} + 3in$

$l_{dh} = \dfrac{1200\,d_{bar}}{f'_c}$ 　　　　$l_{dh} = $ required development length

− use .7 modification factor for >2in cover beyond end of
　　　　　　　　　　　hook

− Try #7 bars ⟶ $d_{bar} = .875\,in$

$l_{dh} = \dfrac{1200\,(0.875in)}{\sqrt{4000}} \ast .7 = 11.6\,in$

$T \geq 11.6\,in + 3in = 14.6\,in \rightarrow \boxed{T = 15\,in}$

Heel Extension Shear analysis $W_{soil} = 12750^{lb}/st$, $W_{foot} = 6300plf$

$V_u/b = 1.4D = 1.4(20190 plf)$ $W_{LL} = 2.99^k/st$

$V_u/b = 28.3^k/st$ $W_{DL} = 1.14^k/st$

$V_n/b = 2b_w d_e \sqrt{f'_c} = 2(12in)(d_e)\sqrt{4ksi}$

$V_n/b = 1.518 d_e \; ^{k \cdot in}/st$

$V_u/b \leq \phi V_n/b \longrightarrow 28.3^k/st \leq .85(1.518)(d_e) \frac{^{k \cdot in}}{st}$

$d_e \geq 21.9'' \longrightarrow$ use $22'' = d_e$

Assume #8 bars $(d_{bar} = 1in)$

$T_{ft} \geq d_e + \frac{d_b}{2} + 3in = 22'' + \frac{1.0''}{2} + 3in$

$T_{ft} = 24.5 in \geq 15in$ $T_{ft} = 25in$

Heel Extension Flexural Analysis

$M_u = 1.4(W_{soil} + W_{footing} + W_{DL})\left(\frac{l_{stem} + l_{heel}}{2}\right)\left(\frac{12in}{st}\right)$

$M_u = 1.4(12750plf + 6300plf + 1140plf)\left(\frac{3'+6'}{2}\right)\left(\frac{12in}{st}\right)$

$M_u = 1526364^{lb \cdot in}/st$

$A_s/b = \frac{f'_c(b)}{(1.176)(f_y)}\left(d_e - \sqrt{d_e^2 - \frac{2.353(M_u/b)}{\phi f'_c b}}\right)$

$A_s/b = \frac{4000psi(12in)}{(1.176)(60000psi)}\left[21in - \sqrt{(21in)^2 - \frac{2.353(1526364^{lb \cdot in}/st)}{.9(4000psi)(12in)}}\right]$

$A_s/b = 1.42 \; ^{in^2}/ft$

Minimum amount of reinforcement

$A_s/b = .002 A_g = .002(T)(b) = .002(25in)(12in)$

$A_s/b = .324 \; ^{in^2}/ft$

Use #9 bars @ 9in spacing

$A_s = 2 \cdot \frac{A_{bar}}{2}\left(\frac{12in}{9in}\right) = 2 \cdot \frac{1.0in^2}{2}\left(\frac{12in}{9in}\right)$

$A_s = 1.42 \; ^{in^2}/st > .324 \checkmark$

Appendix G

Wing Wall Calculations
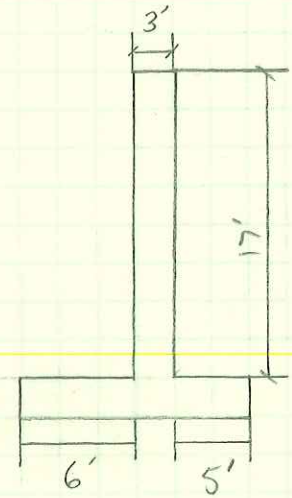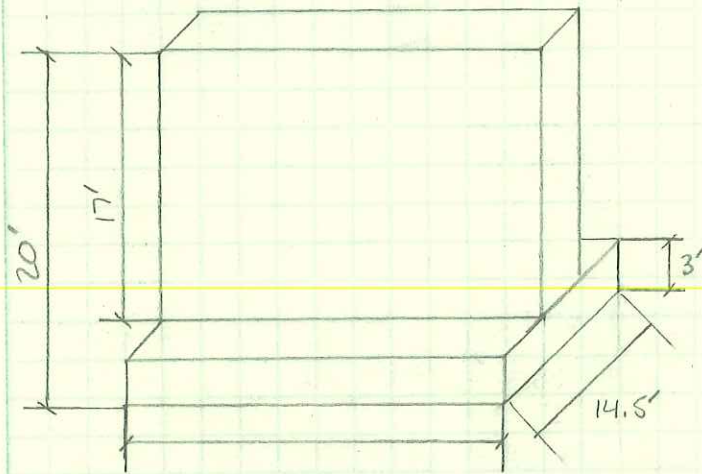
- Dead Load Effects

$h_{wwstem} = 17'$ $\longrightarrow$ H_{ww}; Height of stem

$DL_{wwstem} = [3' \cdot 17'] \cdot W_c$ $\longrightarrow$ W_c; Weight of concrete $= 150\,pcf$

$DL_{wwstem} = 51\,ft^2 \times 150\,pcf$

$\boxed{DL_{wwstem} = 7.65\,K/ft}$

- Lateral Earth Pressure

$p = K_a \cdot \gamma_s \cdot z$ $\longrightarrow$

$p = (0.3)(0.125\,kcf)(20')$

$\boxed{p = 0.75\,ksf}$

P; Earth Pressure
K_a; Earth Pressure Coefficient $= 0.3$
γ_s; Density of soil $= 0.125\,kcf$
z; Height of Wall $= 20\,ft$

- Lateral Load due to Earth Pressure

1. $H_{ww} = 20'$

$R_{EHstem} = (\frac{1}{2}) \cdot p \cdot H_{ww}$ $= (0.5)(0.75\,ksf)(20')$

$\boxed{R_{EHstem} = 7.5\,K/ft}$ $\rightarrow$ Located $H_{ww}/3$ from the base.
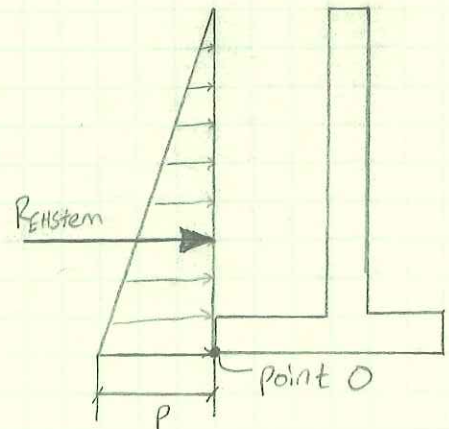
- Loads due to Live Load Surcharge

$\Delta P = K_a \cdot \gamma_s \cdot h_{eq}$ $\longrightarrow h_{eq} = 2'$

$\boxed{\Delta P = 0.075\,ksf}$ $\rightarrow$ $R_{LSstem} = \Delta P \cdot H_{ww}$

$\boxed{R_{LSstem} = (0.075\,ksf)(20') = 1.5\,K/ft}$ $\rightarrow$ Located $\frac{H_{ww}}{2}$ from base.

Wing Wall Calculations (cont.)

- ## Design for Flexure in stem

  - Assume #9 bars $\longrightarrow$ $d_{bar} = 1.128\,in$ (diameter of bar)

    $A_{bar} = 1.0\,in^2$ (Area of bar)

  - Cracking strength.

  $$M_{cr} = \frac{f_r\,I_g}{y_t} \qquad \longrightarrow \qquad f_r = 0.24\sqrt{f'_c}$$

  $$M_{cr} = \frac{(0.24 \cdot \sqrt{4\,ksi})(46656\,in^4)}{18''} \qquad I_g = \tfrac{1}{12}(12'')(36'')^3$$

  $$y_t = 18''$$

  $$\frac{M_{cr}}{ft.} = \frac{1244.2\ K\text{-}in}{ft.}$$

  $$\boxed{M_{cr} = 103.7\ \frac{K\text{-}ft}{ft}}$$

  - Use factor of 1.2 for M_{cr} to design the Wing Wall Stem flexure reinforcement.

  $$\boxed{1.2\,(M_{cr}) = 124.4\ \frac{K\text{-}ft}{ft}}$$
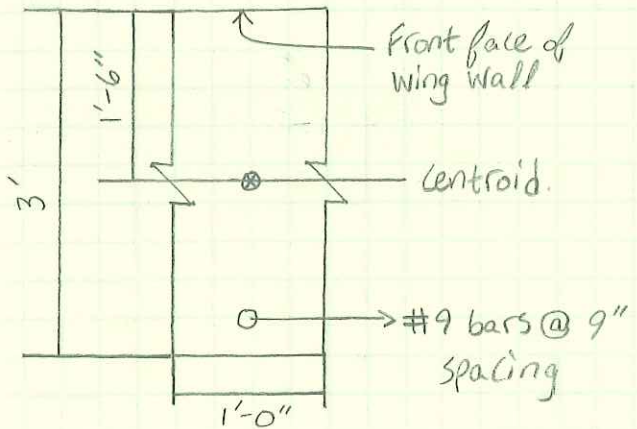
Front face of Wing Wall

centroid.

#9 bars @ 9" spacing

1'-6"

3'

1'-0"

  - Effective depth of reinforcement.

  $t_{bw} = 36\,in$ (Thickness of stem)

  $Cover = 2.5\,in$

  $$d_e = t_{bw} - Cover - \frac{d_{bar}}{2} = 36\,in - 2.5\,in - \frac{1.128\,in}{2}$$

  $$d_e = 32.9\,in \qquad \longrightarrow \qquad use \quad \boxed{d_e = 33\,in}$$

  - Required amount of steel reinforcement.

  $\phi = 0.9$
  $b = 12\,in$
  $f'_c = 4\,ksi$
  $f_y = 60\,ksi$

  $$R_n = \frac{(1.2\,M_{cr}) \cdot 12\,in}{\phi \cdot b \cdot d_e^2} = \frac{\left(124.4\ \frac{K\text{-}ft}{ft}\right)\left(12\ \tfrac{in}{ft}\right)}{(0.9)(12\,in)(33\,in)^2}$$

  $$\boxed{R_n = 0.13\,ksi}$$

Wing Wall Calculations (cont.)

- Required amount of steel reinforcement (cont.)

$$\rho = 0.85 \frac{f'_c}{f_y}\left[1.0 - \sqrt{1.0 - \frac{2 \cdot R_n}{0.85 f'_c}}\right] = 0.85 \frac{4 \text{ ksi}}{60 \text{ ksi}}\left[1.0 - \sqrt{1.0 - \frac{2(0.13 \text{ ksi})}{0.85(4 \text{ ksi})}}\right]$$

$$\boxed{\rho = 0.00214}$$

$$A_s = \rho \cdot \frac{b}{ft} \cdot d_e = 0.00214\left(\frac{12 \text{ in}}{ft}\right)(33 \text{ in})$$

$$\boxed{A_s = 0.85 \text{ in}^2/ft.}$$

- Required Spacing $= \frac{A_{bar}}{A_s} = \frac{1.0 \text{ in}^2}{0.85 \text{ in}^2/ft} = \underline{14.1 \text{ in}}$

✱ Use #9 bars @ 9" spacing to match abutment stem vertical bar spacing.

$$A_s = A_{bar}\left(\frac{12 \text{ in}}{bar\text{-}space}\right) = 1.0 \text{ in}^2\left(\frac{12 \text{ in}}{9 \text{ in}}\right)$$

$$\boxed{A_s = 1.33 \text{ in}^2/ft.}$$

- Maximum reinforcement check

$$T = A_s \cdot f_y = 1.33 \text{ in}^2/ft \, (60 \text{ ksi})$$

$$\boxed{T = 80 \text{ kip}}$$

$$a = \frac{T}{0.85 f'_c \cdot b} = \frac{80 \text{ K}}{0.85(4 \text{ ksi})(12 \text{ in})}$$

$$\boxed{a = 1.96 \text{ in}}$$

$$c = \frac{a}{\beta_1} \qquad \longrightarrow \qquad \beta_1 = 0.85 - 0.05\left(\frac{f'_c - 4000}{4000}\right)$$

$$\beta_1 = 0.85 - 0.05\left(\frac{4000 - 4000}{4000}\right) = 0.85 - 0$$

$$c = \frac{1.96 \text{ in}}{0.85} \qquad\qquad \beta_1 = 0.85$$

$$\boxed{c = 2.31 \text{ in}}$$

$$\frac{c}{d_e} \le 0.42 \qquad \longrightarrow \qquad \frac{2.31 \text{ in}}{33 \text{ in}} \le 0.42$$

$$\boxed{0.07 \le 0.42} \qquad \checkmark \qquad OK.$$

Wing Wall Calculations (cont.)

- Check Crack Control

$$d_c = C_c + \frac{d_{bar}}{2} \longrightarrow \quad C_c; \text{ Clear Cover should be no greater than 2in.}$$

$$d_c = 2in + \frac{1.128in}{2}$$

$$\boxed{d_c = 2.56 in}$$

$$A_c = 2 \cdot d_c \cdot bar\text{-}space \longrightarrow \quad A_c; \text{ Concrete Area}$$

$$A_c = 2(2.56in)(9in)$$

$$\boxed{A_c = 46.15 in^2}$$

$$f_{sa} = \frac{Z}{(d_c \cdot A_c)^{1/3}} \quad \text{where } f_{sa} \leq 0.6 f_y \longrightarrow \quad f_{sa}; \text{ Allowable reinforcement service load stress}$$

$$\qquad\qquad\qquad\qquad\qquad\qquad Z; 130 \text{ }^{k}/in \text{ (Exposure to salt)}$$

$$f_{sa} = \frac{130 \text{ }^{k}/in}{[(2.56)(46.15in^2)]^{1/3}}$$

$$\boxed{f_{sa} = 26.5 \text{ KSi}} \qquad 0.6 f_y = 36 \text{ KSi}$$

$$\boxed{26.5 \text{ KSi} \leq 36 \text{ KSi}} \quad \checkmark \text{ OK.}$$

$$n = \frac{E_s}{E_c} \longrightarrow \quad E_s = 29000 \text{ KSi}$$
$$\qquad\qquad\qquad\qquad E_c = 3640 \text{ KSi}$$

$$n = \frac{29000 \text{ KSi}}{3640 \text{ KSi}} =$$

$$n = 7.97 \longrightarrow \quad \text{use } \boxed{n = 8}$$

$$M_{ustm serv1} = \left(R_{EHstem} \cdot \frac{H_{ww}}{3}\right) + \left(R_{ESstem} \cdot \frac{H_{ww}}{2}\right)$$

$$M_{ustm serv1} = \left(7.5 \text{ }^{k}/ft \cdot 6.667ft\right) + \left(1.5 \text{ }^{k}/ft \cdot 10ft\right)$$

$$\boxed{M_{ustm serv1} = 65 \frac{k\text{-}ft}{ft}}$$

Wing Wall Calculations (cont.)

- Calculate f_s, actual stress in reinforcement.

$$\rho = \frac{A_s}{\frac{b}{ft} \cdot de} = \frac{1.33 \, in^2}{\frac{12 \, in}{ft} \cdot .33 \, in}$$

$$\rho = 0.00336$$

$$K = \sqrt{(\rho \cdot n)^2 + (2 \cdot \rho \cdot n)} - (\rho \cdot n) = \sqrt{[(0.00336)(8)]^2 + (2)(0.00336)(8)} - [(0.00336)(8)]$$

$$K = 0.21$$

$$K \cdot de = 0.21 * 33 \, in = \boxed{6.93 \, in} \longrightarrow \text{Location of neutral axis from front face of Wing Wall}$$

$$I_t = \frac{1}{3}\left(12 \, \frac{in}{ft}\right)(K \cdot de)^3 + n \cdot A_s (de - K \cdot de)^2$$

$$I_t = \frac{1}{3}\left(12 \, \frac{in}{ft}\right)(6.93)^3 + (8)(1.33 \, in^2)(33 \, in - 6.93 \, in)^2$$

$$\boxed{I_t = 8562.67 \, \frac{in^4}{ft}}$$

$$y = de - K \cdot de = 33 \, in - 6.93 \, in$$

$$\boxed{y = 26.07 \, in}$$

$$f_s = \frac{n \cdot \left(M_{ustmserul} \cdot 12 \, \frac{in}{ft} \cdot y\right)}{I_t} = \frac{8\left(65 \, \frac{K-ft}{ft}\right)\left(12 \, \frac{in}{ft}\right)(26.07 \, in)}{8562.67 \, \frac{in^4}{ft}}$$

$$f_s = 18.99 \, KSi \longrightarrow use \boxed{f_s = 19 \, KSi}$$

$$f_{sa} > f_s$$

$$26.48 > 19 \quad \checkmark \quad OK.$$

Wing Wall Calculations (cont.)

- ## Design for Shear

$V_{uww \, stem} = R_{EHstem} + R_{LSstem} = 7.5 \, ^{k}/_{ft} + 1.5 \, ^{k}/_{ft}$

$\boxed{V_{uww \, stem} = 9.0 \, ^{k}/_{ft}}$

- Nominal shear resistance

$V_{n1} = V_c + V_s$

$V_{n2} = 0.25 f'_c \, b_v \, d_v$ $\Biggr\}$ use lesser of the two.

$V_c = 0.0316 \cdot \beta \cdot \sqrt{f'_c} \cdot b_v \, d_v$

* Neglect V_s for this design

$\beta = 2$

$b_v = 12 in$

$d_v = max \underset{①}{(d_e - ^a/_2,} \, \underset{②}{0.9 \cdot d_e,} \, \underset{③}{0.72h)}$

$d_v = 32.02 in.$

① $33'' - \dfrac{1.96''}{2} = \boxed{32.02 \, in}$

② $0.9(33'') = 29.7 \, in$

③ $0.72(36'') = 25.92 \, in$

$V_{n1} = 0.0316 (2)(\sqrt{4 ksi})(12 in)(32.02 in) = 48.6 \, ^{k}/_{ft}$

$V_{n2} = 0.25(4 ksi)(12 in)(32.02 in) = 384.24 \, ^{k}/_{ft}$

$\boxed{V_n = V_{n1} = 48.6 \, ^{k}/_{ft}}$

$\phi V_n = 0.9 (48.6 \, ^{k}/_{ft})$

$\boxed{\phi V_n = 43.7 \, ^{k}/_{ft}}$

$\phi V_n > V_{uww \, stem}$

$43.7 > 9$ ✓ OK.

Wing Wall Calculations (cont.)

- <u>Shrinkage & temperature reinforcement</u>

① $A_s \geq 0.11 \dfrac{A_g}{b_y}$ ⎫
⎬ use lesser value.
② $\Sigma A_b = 0.0015 A_g$ ⎭

$A_g = T_{bw} \cdot \dfrac{12 in}{ft} = 36 in (12\ ^{in}/_{ft})$

$\boxed{A_g = 432\ ^{in^2}/_{ft}}$

① $0.11 \dfrac{A_g}{b_y} = 0.11 \dfrac{(432\ ^{in^2}/_{ft})}{(60\ KSi)} = 0.8\ ^{in^2}/_{ft}$

② $0.0015 (432\ ^{in^2}/_{ft}) = 0.65\ ^{in^2}/_{ft}$

$\boxed{A_s \geq 0.65\ ^{in^2}/_{ft}}$

- Try horizontal #5 bars for each face of
Wing Wall @ 9 in spacing

$d_{bar} = 0.625 in$
$A_{bar} = 0.31\ in^2$

$A_s = 2 \cdot \dfrac{A_{bar}}{2} \left(\dfrac{12 in}{9 in}\right) = 2\left(\dfrac{0.31\ in^2}{2}\right)\left(\dfrac{12 in}{9 in}\right)$

$\boxed{A_s = 0.83\ ^{in^2}/_{ft}}$

$0.83 \geq 0.65 \quad \checkmark \quad OK.$

- <u>Based on Wing Wall Design.</u>

- Back face flexure reinforcement: #9 bars @ 9 in spacing

- Front face vertical reinforcement: #5 bars @ 9 in spacing

- Horizontal temperature & shrinkage reinforcement: #5 bars @ 9 in spacing

Wing Wall Calculations (cont.)

Footing Reinforcement

$T_{ft} \geq l_{dh} + 3in.$

$l_{dh} = \dfrac{1200 \, d_{bar}}{\sqrt{f'_c}}$ $\longrightarrow l_{dh}$; required development length.

- use 0.7 modification factor for > 2in cover beyond end of hook.
- Try #7 bar. $\rightarrow$ $d_{bar} = 0.875$ in

$l_{dh} = \dfrac{1200\,(0.875in)}{\sqrt{4000}} * 0.7 = 11.6in$

$T \geq 11.6in + 3in = 14.6 \rightarrow$ use $\boxed{T = 15in}$

- **Heel Extension Shear Analysis.** ($W_{soil} = 12750 \, \frac{lb}{ft}$, $W_{footing} = 6300 \, \frac{lb}{ft}$)

$V_u/b = 1.4D = 1.4(12750 \, \frac{lb}{ft} + 6300 \, \frac{lb}{ft})$

$V_u/b = 26.7 \, \frac{K}{ft}$

$V_n/b = 2 b_w d_e \sqrt{f'_c} = 2(12in) d_e \sqrt{4000psi}$

$V_n/b = 1.518(d_e) \, \frac{K\cdot in}{ft}$

$V_u/b \leq \phi V_n/b \rightarrow 26.7 \, \frac{K}{ft} \leq (0.85)(1.518)(d_e) \, \frac{K\cdot in}{ft}$

$d_e \geq \dfrac{26.7 \, \frac{K}{ft}}{1.29 \, \frac{K\cdot in}{ft}}$

$d_e \geq 20.7in \rightarrow$ use $\boxed{d_e = 21in}$

- Assume #8 bars ($d_{bar} = 1.0in$)

$T_{ft} \geq d_e + \frac{d_b}{2} + 3in = 21in + \frac{1.0in}{2} + 3in$

$T_{ft} = 24.5in > 15in \rightarrow$ use $\boxed{T_{ft} = 25in}$

Wing Wall Calculations (cont.)

- Heel extension flexural Analysis

$$M_u/b = 1.4 \left(W_{soil} + W_{footing} \right) \left(\frac{l_{stem} + l_{heel}}{2} \right) \left(\frac{12 in}{ft} \right)$$

$$M_u/b = 1.4 \left(12750 \, \tfrac{lb}{ft} + 6300 \, \tfrac{lb}{ft} \right) \left(\frac{3ft + 6ft}{2} \right) \left(\frac{12 in}{ft} \right)$$

$$M_u/b = 1440180 \, \frac{lb \cdot in}{ft}$$

$$A_s/b = \frac{(f'_c)(b)}{(1.176)(f_y)} \left(d_e - \sqrt{d_e^2 - \frac{2.353(M_u/b)}{\phi f'_c \, b}} \right)$$

$$A_s/b = \frac{4000 \, psi \, (12 in)}{(1.176)(60000 psi)} \left[21 in - \sqrt{(21 in)^2 - \frac{(2.353)(1440180 \, lb\text{-}in/ft)}{(0.9)(4000 psi)(12 in)}} \right]$$

$$\boxed{A_s/b = 1.33 \, \tfrac{in^2}{ft}}$$

- Minimum amount of steel reinforcement.

$$A_s/b = 0.002 \, A_g = 0.002 \, (T)(b) = 0.002 \, (25 in)(12 in)$$

$$A_s/b = 0.324 \, \tfrac{in^2}{ft}$$

- use #9 bars @ 9in spacing

$$A_s = 2 \cdot \frac{A_{bar}}{2} \left(\frac{12 in}{9 in} \right) = 2 \cdot \frac{1.0 in^2}{2} \left(\frac{12 in}{9 in} \right)$$

$$A_s = 1.33 \, \tfrac{in^2}{ft} \qquad > 0.324 \qquad \checkmark \; OK.$$

Wing Wall Final Schematic

3'-0"

17'-0"

#5 bars @ 9" spacing

← 2½" →

#9 bars
@ 9" spacing

7'-0"

#5 bars @
9" spacing

3'-0"

14'
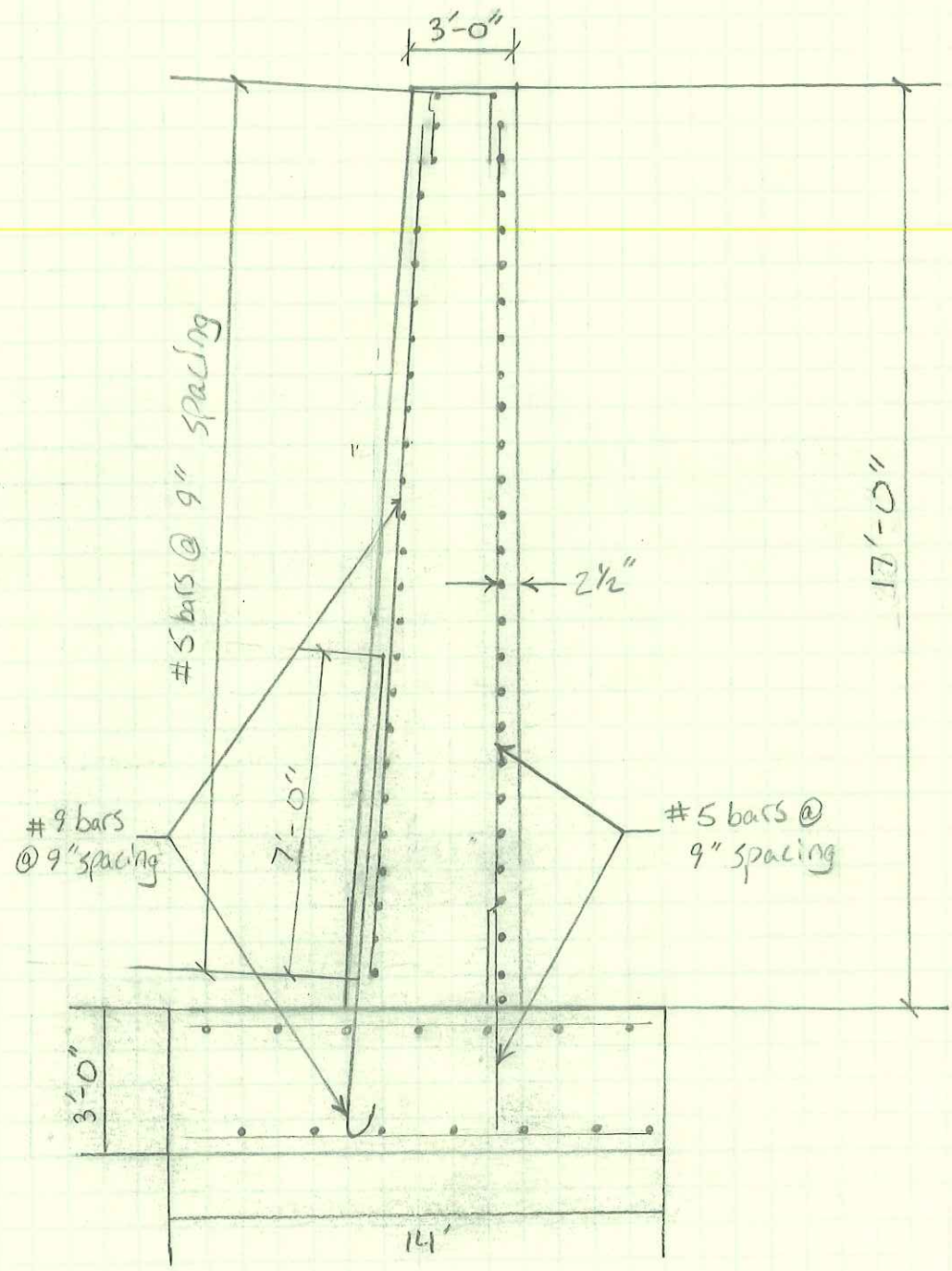
Wing Wall Cost Analysis.

- Based off of an example from infrastructurecost.com
  - Excavation & Backfill Cost (For 2-50ft Wing Walls)
    - Quantity (yrd^3) = 2(50ft)(20ft)(14ft) = 28000ft^3 $\left(\frac{1/3 \, yrd}{1ft}\right)^3$ = 1037.04 yrd^3
    - Man hours(0.13) = 1037.04 yrd^3 (0.13 \$/yrd^3) = \$ 135
    - Labor (6.32) = 1037.04 yrd^3 (6.32 \$/yrd^3) = \$ 6555
    - Equipment (5.62) = 1037.04 yrd^3 (5.62 \$/yrd^3) = \$ 5829
    - Job Materials (0.11) = 1037.04 yrd^3 (0.11 \$/yrd^3) = \$ 115
    - Permanent Materials(6.17) = 1037.04 yrd^3 (6.17 \$/yrd^3) = \$ 6399

    $$\text{Total} = \$ 19033$$

- Construction Cost (For 2-50ft Wing Walls)
  - Quantity (yrd^2) = 2(50ft)(20ft + 14ft) = 3400ft^2 $\left(\frac{1/3 \, yrd}{1ft}\right)^2$ = 377.8 yrd^2
  - Man hour (3.37) = 337.8 yrd^2 (3.37 \$/yrd^2) = \$ 1139
  - Labor (160.51) = 337.8 yrd^2 (160.51 \$/yrd^2) = \$ 54221
  - Equipment (91.68) = 337.8 yrd^2 (91.68 \$/yrd^2) = \$ 30970
  - Job Material (2.18) = 337.8 yrd^2 (2.18 \$/yrd^2) = \$ 737
  - Permanent Material (641.51) = 337.8 yrd^2 (641.51 \$/yrd^2) = \$ 216703

  $$\text{Total} = \$ 303770$$

- Total Cost for Wing Wall

  $$\text{Cost} = \$ 322803$$

  $$\boxed{\text{Use Cost} = \$ 330000}$$

Excel Spreadsheets

	A	B	C	D	E	F	G	H	I	J
1	Cantilever Retaining Wall Design Calculations Including Surcharge Laterally									
2	For the At-Rest Condition									
3										
4	**Assumptions**			**Lateral Forces (lb/ft)**			**Moment Arm (ft)**		**Moments (kip-ft/ft)**	
5	Frictional Angle φ (degrees)	32		L.E.P. backfill(Fbfsoil)	7521.292		5.333333333		40.11355612	
6	Frictional Angle φ (Radians)	0.5585054		Surcharge (Fsc)	1805.11		8		14.4408802	
7	Unit Weight of Soil Υ (pcf)	125		Pore Water Pressure (Fw)	0		0		0	
8	Coefficient of friction μ	0.55								
9	Surcharge Load σ sc (psf)	240								
10	Unit Weight of Concrete (pcf)	150		**Vertical Forces (lb/ft)**			**Moment Arm (ft)**		**Moments (kip-ft/ft)**	
11				Weight of Soil (Wsoil)	8750		9		78.75	
12	**Dimensions**			Weight of Stem (Wstem)	5250		5.25		27.5625	
13	Total Height (ft)	16		Weight of Footing (Wfoot)	3450		5.75		19.8375	
14	Stem height (ft)	14		Weight of Surcharge (Wsc)	1200		9		10.8	
15	Footing height (ft)	2								
16	Toe length (ft)	4		**Resultant Forces&Moments**						
17	Heel length (ft)	5		Resultant Lateral Force Flr (	9.326402					
18	Stem length (ft)	2.5		Resultant Lateral Moment N	54.55444					
19	Footing length (ft)	11.5		Resultant Vertical Force Fvr	18.65					
20	Embedment depth (ft)	4		Resultant Vertical Moment	136.95					
21										
22	**Design Criteria**			**Resultant F&M locations from point O**						
23	Vertical Effective Stress σ'z (psf)	1750		Length of Flr horizontally h	5.849462					
24	Lateral Earth Pressures (L.E.P.):			Length of Fvr horizontally I	7.343164					
25	-At-Rest Ko	0.4700807								
26	-Active Ka	0.3072585								
27	-Passive Kp	3.2545883								
28				**Failure Mode**	**F.O.S.**					
29				Overturning	2.510337					
30				Sliding	2.429244					
31				Bearing Capacity	9.620261					
32										

Factor of Safety Calculation for Overturning failure

For the At-Rest Condition

Assumptions			Resultant Forces&Moments	
Frictional Angle ϕ (degrees)	32		Resultant Lateral Force Flr (k/ft)	9.326402
Frictional Angle ϕ (Radians)	0.558505		Resultant Lateral Moment Mlr (k-ft/ft)	54.55444
Unit Weight of Soil Y (pcf)	125		Resultant Vertical Force Fvr (k/ft)	18.65
Coefficient of friction μ	0.55		Resultant Vertical Moment Mvr (k-ft/ft)	136.95
Surcharge Load σ sc (psf)	240			
Unit Weight of Concrete (pcf)	150			
Dimensions			**Factor of Safety (F.O.S.) Calculation**	
Total Height (ft)	16		Driving Moment Md (k-ft/ft)	54.55444
Stem height (ft)	14		Resisting Moment Mr (k-ft/ft)	136.95
Footing height (ft)	2			
Toe length (ft)	4		F.O.S. Overturning	2.510337
Heel length (ft)	5			
Stem length (ft)	2.5			
Footing length (ft)	11.5			
Design Criteria				
Vertical Effective Stress σ'z (psf)	1750			
Lateral Earth Pressures (L.E.P.):				
-At-Rest Ko	0.470081			
-Active Ka	0.307259			
-Passive Kp	3.254588			

Factor of Safety Calculation for Sliding Failure

For the At-Rest Condition

	Assumptions			Resultant Forces&Moments	
5	Frictional Angle φ (degrees)	32		Resultant Lateral Force Flr (k/ft)	9.326402
6	Frictional Angle φ (Radians)	0.558505		Resultant Lateral Moment Mlr (k-ft/ft)	54.55444
7	Unit Weight of Soil Y (pcf)	125		Resultant Vertical Force Fvr (k/ft)	18.65
8	Coefficient of friction μ	0.55		Resultant Vertical Moment Mvr (k-ft/ft)	136.95
9	Surcharge Load σ sc (psf)	240			
10	Unit Weight of Concrete (pcf)	150		Frictional Force Calculations	
11	Interfacial Frictional Angle (degrees)	21.33333		Tao (ksf/ft)	0.348357
12	Interfacial Frictional Angle (radians)	0.372337		Vertical Stress (ksf/ft)	1.621739
13				Vertical Effective Stress (ksf/ft)	0.891957
14	Dimensions				
15	Total Height (ft)	16			
16	Stem height (ft)	14			
17	Footing height (ft)	2			
18	Toe length (ft)	4			
19	Heel length (ft)	5			
20	Stem length (ft)	2.5			
21	Footing length (ft)	11.5			
22					
23	Design Criteria			Factor of Safety (F.O.S.) Calculation	
24	Induced pressure (psf)	1750		Driving Force Fd (k/ft)	9.326402
25	Lateral Earth Pressures (L.E.P.):			Resisting Force Fr (k/ft)	22.65611
26	-At-Rest Ko	0.470081			
27	-Active Ka	0.307259		F.O.S. Sliding	2.429244
28	-Passive Kp	3.254588			

Factor of Safety Calculation for Bearing Capacity Failure
For the At-Rest Condition

Assumptions			Resultant Forces&Moments			Eccentricity Calculations	
Frictional Angle φ (degrees)	32		Resultant Lateral Force Flr (k/ft)	9.326402		Combined Resultant (Flr&Fvr)Fr	20.85196
Frictional Angle φ (Radians)	0.558505		Resultant Lateral Moment Mlr (k-ft/ft)	54.55444		Angle of Fr (degrees)	28.64056
Unit Weight of Soil Y (pcf)	125		Resultant Vertical Force Fvr (k/ft)	18.65		Sum of the Moments (k-ft/ft)	82.39556
Coefficient of friction μ	0.55		Resultant Vertical Moment Mvr (k-ft/ft)	136.95		Moment arm of Fr a (ft)	3.951454
Surcharge Load σ sc (psf)	240					Horizontal distance Fr is from O (ft)	4.417993
Unit Weight of Concrete (pcf)	150		Resultant F&M locations from point O			Eccentricity e (ft)	1.332007
			Length of Flr horizontally hr (ft)	5.849462		B/6 Value (ft)	1.916667
Dimensions			Length of Fvr horizontally lr (ft)	7.343164			
Total Height (ft)	16						
Stem height (ft)	14		Terzaghi's Method			Bearing Pressure Calculations	
Footing height (ft)	2		Nc	44		Minimum Bearing Pressure qmin (ks	0.154895
Toe length (ft)	4		Nq	28.5		Maximum Bearing Pressure qmax (k	3.088583
Heel length (ft)	5		Ny	28		Ultimate Bearing Pressure qult (ksf)	29.71297
Stem length (ft)	2.5		Vertical Effective Stress σ'z (psf)	500			
Footing length (ft)	11.5		B' (ft)	8.835985		F.O.S.	9.620261
			Unit Weight of Soil Y (pcf)	125			
Design Criteria			Embedment Depth (ft)	4			
Vertical Effective Stress σ'z (psf)	500						
Lateral Earth Pressures (L.E.P.):							
-At-Rest Ko	0.470081						
-Active Ka	0.307259						
-Passive Kp	3.254588						

Appendix H

Top M_u = 39.6 k-ft

Bottom M_u = 31.6 k-ft

<u>Slab Constraints:</u>   (BDM-LRFD, Ch 3, Sec 2, Geometric Constraints)

8 in in slab thickness
2 in in top clear cover
1.25 in in bottom clear cover

<u>Top:</u> Use $d = 6''$, Assume $a = 1''$ $\phi = 0.9$

$$A_s = \frac{M_u}{\phi f_y \left(d - \frac{a}{2}\right)} = \frac{39.6 \text{ k-ft} \cdot 12''/\text{ft}}{0.9(60\text{ksi})\left(6 - \frac{1}{2}\right)} = 1.6 \text{ in}^2$$

$b = 12''$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.6 \text{ in}^2 (60\text{ksi})}{0.85(4\text{ksi})(12'')} = 2.353 \text{ in}$$

$$A_s = \frac{39.6 \cdot 12''/\text{ft}}{0.9(60\text{ksi})\left(6 - \frac{2.353}{2}\right)} = \frac{475.2}{0.9(60)(4.82)} = 1.83 \text{ in}^2$$

<u>Top Rebar $A_s = 1.83 \text{ in}^2$</u>
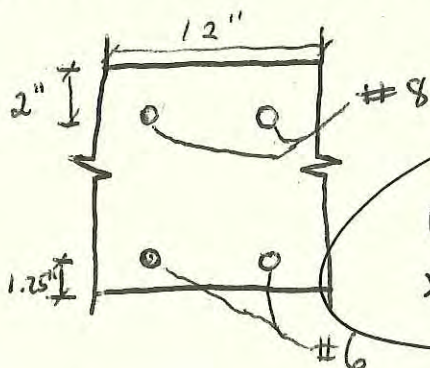
<u>Bottom:</u> Same constraints, but $d = 6.75$ in

$$A_s = \frac{31.6 \text{ k-ft} \cdot 12''/\text{ft}}{0.9(60\text{ksi})\left(6 - \frac{1}{2}\right)} = 1.28 \text{ in}^2$$

$$a = \frac{1.28 \text{ in}^2 (60\text{ksi})}{0.85(4\text{ksi})(12'')} = 1.88 \text{ in}$$

$$A_s = \frac{31.6 \text{ k-ft} \cdot 12''/\text{ft}}{0.9(60\text{ksi})\left(6 - \frac{1.88}{2}\right)} = 1.39 \text{ in}^2$$

<u>Bottom Rebar $A_s = 1.39 \text{ in}^2$</u>

Top: $1.83 \text{ in}^2 \Rightarrow$ Use 2 #8 bars ($A_s = 2.00 \text{ in}^2$) ✓

Bottom: $1.39 \text{ in}^2 \Rightarrow$ Use 2 #6 bars ($A_s = 1.50 \text{ in}^2$) ✓

Mueller - Breslau

Convert to Spreadsheet

$.0018 \, (A_s)$ secondary

- How do they treat vehicle loading surcharge?
- Spreadsheet deals w/ lateral
     - Use that to look at abutment facility
- traffic surcharge : bridge pushes on abutment
     - a help on over turning
- vertical load also helps overturning
- not so much overturning, but large <u>bearing</u> stress
     - Consider footing heavily
- Bridge DL helps w/ stability
- Large Vertical forces help w/ overturning + sliding
- Sort out what's going on under abutment

- - -

- Consider longitudinal effects from wheels on pier
     - combined axial + overturning

<u>Loads Due to Uniform Surcharge:</u>

Approach Slab and Roadway will cover the abutment backfill material, no uniform surcharge load will be applied.

<u>Loads Due to live load surcharge</u>

Loads due to live load surcharge → horizontal pressure increase

$$\Delta p = k \cdot \gamma_s \cdot h_{eq}$$

Bottom of backwall live load Surcharge load:

$k = k_a$

$\gamma_s = 0.120 \text{ kcf}$ (use average of loose and compacted gravel)

$h_{eq} = 3.6 \text{ ft}$ equivalent height of soil for vehicular loading based on 7 ft backwall height

$$\Delta_p = k \cdot \gamma_s \cdot h_{eq}$$
$$\boxed{\Delta_p = 0.130 \text{ ksf}}$$

Lateral load due to live load surcharge is:

$$R_{LS_{bw}} = \Delta_p \cdot h_{bkwll} \quad \Rightarrow \quad R_{LS_{bw}} = 0.91 \frac{k}{ft}$$

Bottom of abutment stem live load surcharge load:

$k = k_a$ $\quad \gamma_s = 0.120 \text{ kcf}$ (use average of loose + compacted gravel)

$h_{eq} = 2 \text{ ft}$ (equivalent height of soil for vehicular loading based on stem height)

$$\Delta_p = k \cdot \gamma_s \cdot h_{eq}$$
$$\boxed{\Delta_p = 0.072 \text{ ksf}}$$

The lateral load due to the live load surcharge load:

$$R_{LS_{stem}} = \Delta_p \cdot h_{stem}$$
$$\boxed{R_{LS_{stem}} = 1.58 \frac{k}{ft}}$$

Bottom of footing live load surcharge load:

$k = k_a$

$\gamma_s = 0.120 \text{ kcf}$ (use average of loose + compacted gravel)

$h_{eq} = 2 \text{ ft}$ equivalent height of soil for vehicular loading

$$\Delta_p = k \cdot \gamma_s \cdot h_{eq}$$
$$\boxed{\Delta_p = 0.072 \text{ ksf}}$$

The lateral load due to the live load surcharge is:

$$R_{LS_{ftg}} = \Delta_p \cdot (h_{stem} + t_{ftg})$$

$$\underline{R_{LS_{ftg}} = 1.76 \frac{k}{ft}}$$

Appendix I

Pier Design Example based on AASHTO LRFD Bridge Design Specs.

$$W_c = 0.150 \text{ kcf} \quad f'_c = 4.0 \text{ ksi} \quad f_y = 60.0 \text{ ksi}$$

Rebar Cover Reqs, Dimensional Aspects

$$S = 7.75 \text{ in} \qquad N = 6$$
Deck Overhang $= 3.9375 \text{ ft} = t_o$
$L = 100 \text{ ft} \qquad H_{par} = 3.5 \text{ ft}$

Pier Cap: $Cover_{cp} = 2.5 \text{ in}$
Pier Column: $Cover_c = 2.5 \text{ in}$
Footing Top Cover: $Cover_{fT} = 2.0 \text{ in}$
Footing bottom Cover: $Cover_{fb} = 3.0 \text{ in}$

Haunch Thickness: Iowa DOT $\Rightarrow H_{hnch} = 3.5 \text{ in}$ $t_{bf} = 1.03 \text{ in}$

Web Depth $\Rightarrow$ W40×167 $\therefore D_o = 40.0 \text{ in}$ $H_{brng} = 6.0 \text{ in}$

Superstructure Depth

$$H_{super} = H_{par} + \left(\frac{t_o + H_{hnch} + D_o + t_{bf}}{12 \text{ in/ft}} \right)$$

$$= 3.5 \text{ ft} + \left(\frac{\overset{9.0 \text{ in}}{\cancel{3.9375 \text{ ft}}} + 3.5 \text{ in} + 40 \text{ in} + 1.03 \text{ in}}{12 \text{ in/ft}} \right)$$

$$\Rightarrow \quad \boxed{\cancel{H_{super}}} \quad \boxed{H_{super} = 7.961 \text{ ft}}$$

Step 2: Select Optimum Pier Type

Hammerhead Pier + Preliminary Dimensions (NOT TO SCALE)
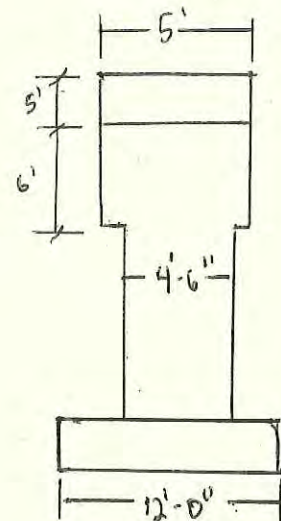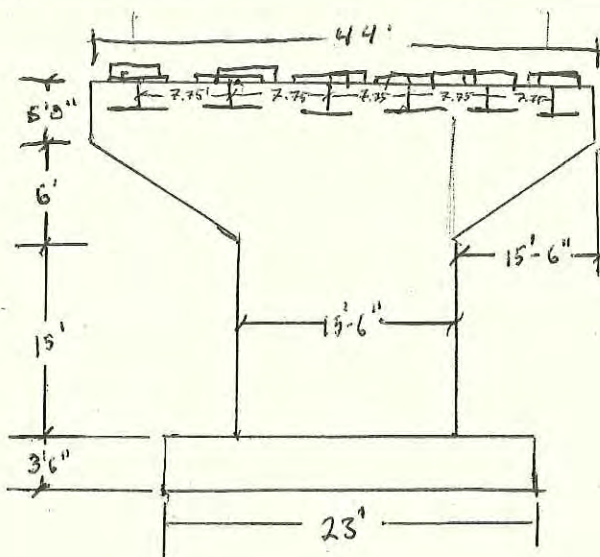
Step 4: Compute Dead Load Effects

USE RISA   REACTION FORCES

Exterior + Interior girder DL Reactions

Cobined point load Reaction force @ critical loading state
= $\boxed{527.2 \text{ K}}$
$\boxed{(\text{Includes Live Load})}$

Pier Cap Dead Load:

Overhang $\stackrel{=}{:} DL_{overhang} = (5ft \cdot 5ft \cdot 15.5ft) \cdot W_c + \frac{1}{2}(6ft \cdot 5ft \cdot 15.5ft) \cdot W_c$

$\boxed{DL_{ovrhg} = 93.00 K}$

Interior: $DL_{int} = (11ft \cdot 5ft \cdot 15.5ft) \cdot W_c \Rightarrow \boxed{DL_{int} = 127.88 \text{ k}}$

Total: $DL_{cap} = 2 \cdot DL_{ovrhg} + DL_{int} \Rightarrow \boxed{DL_{cap} = 313.88 \text{ K}}$

Pier Column Dead Load:

$DL_{col} = (15.5ft \cdot 4.5ft \cdot 15ft) \cdot W_c \Rightarrow \boxed{DL_{col} = 156.94 \text{ k}}$

Pier Footing Dead Load:

$DL_{ftg} = (3.5ft \cdot 23ft \cdot 12ft) W_c \Rightarrow \boxed{DL_{ftg} = 144.90 \text{ K}}$

Weight of soil on top of footing: $\gamma = 0.125 \text{ kcf}$

$EV_{ftg} = 0.125 \text{ kcF} \cdot (2ft) \cdot (23ft \cdot 12ft - 15.5ft \cdot 4.5ft)$

$\boxed{EV_{ftg} = 51.6 \text{ K}}$

Step 5: Live Load Effects

From above: Critical Loading Reaction   at   Pier

$= 527.2 \text{ K} \rightarrow$ Divide evenly across 5 bearings

$\therefore \boxed{R_{1-5_{bearing}} = 105.44 \text{ K}}$

Step 6: Compute other Load Effects   (Braking force, wind loads, temperature
                                             loads, and earthquake loads)

Braking Force

The greater of: (per lane):-   25% of the axle weights of the design trucks
                                5% of the axle weights of design truck + lane load
                                5% of the axle weights of design tandem + lane load

Use 25% of the design truck:

$$BRK_{trk} = 0.25 \cdot (32k + 32k + 8k) = 18.00 \, K$$

$$BRK_{ng} = \frac{18}{5} = \boxed{3.60 \, k = BRK_{BRG}}$$

Wind Load from Super-structure

$L_{span} = 100 \, ft$ width = 44 ft   Depth = $H_{super} - H_{par} = 7.961 - 3.5$

∴ Depth = 4.461 ft $\frac{L_{span}}{width} = \frac{100}{44 \, ft} = 2.27$ ok $\frac{L_{span}}{Depth} = \frac{100}{4.461} = 22.4$ OK

Ratios < 30 ∴ does not need to be investigated for
            aeroelastic instability.

$H_{super} = 7.961 \, ft.$

Tributary Length for Wind Load

$L_{windT} = \frac{100}{2} \, ft$ $L_{windT} = 50 \, ft$ (transverse)

$L_{windL} = 100 \, ft$

Transverse Wind Area : $A_{w \, superT} = H_{super} \cdot L_{windT} = 7.961 ft (50 \, ft) = \underline{398.1 \, ft^2}$

Longitudinal Wind Area : $A_{w superL} = 7.961 ft (100 \, ft) = \underline{796.1 \, ft^2}$

$V_B = 100 \, mph$, $P_D = P_B \rightarrow$ Design wind Pressure = Base wind pressure

$Wind_{total} = 0.050 \, ksf \cdot 7.961 \, ft = 0.398 \frac{k}{ft} > 0.30 \, klf$ __OK__

Wind Attack Angles

$0°$: $WS_{suptrns 0} = A_{w superT} \cdot 0.050 \, ksf => WS_{suptrns 0} = 19.91 \, K$

$60°$: $WS_{suptrns 60} = A_{w superT} \cdot 0.017 ksf = \cancel{20} \, 6.77 \, K$ $^{Long=0}$

$WS_{suptrns 60} = A_{w superL} \cdot 0.019 ksf = \underline{15.13 \, K}$

Transverse Wind Loads (Vertical Loads at Bearings)

$$M_{trns0} = WS_{suptrns0} \cdot \left(\frac{H_{super}}{2}\right) = 19.91K \cdot \left(\frac{7.961\,ft}{2}\right)$$

$$M_{trns0} = \cancel{} \rightarrow M_{trns0} = 79.3\,k \cdot ft$$

$I_{girders} = 950.63\,ft^2$ (example's recommendation)

$$RWS1\text{-}5_{trns0} = \frac{M_{trns0} \cdot 19.5\,ft}{I_{girders}} = \frac{79.3k \cdot ft \cdot 19.5\,ft}{950.63\,ft^2}$$

$$\boxed{RWS1\text{-}5_{trns0} = 1.63\,K}$$

$$RWS2\text{-}4_{trns0} = \frac{79.3k \cdot ft \cdot 7.75\,ft}{950.63\,ft^2} \Rightarrow \boxed{RWS2\text{-}4_{trns0} = 0.65\,K}$$

$$\boxed{RWS3_{trns0} = 0.0K}$$

* Reactions for other attack angles can be obtained by multiplying above reactions by ratio of the transverse load at the angle of interest to the transverse load at an attack angle of zero. (i.e. $WS_{suptrns0} = 19.91\,K$)

<u>Vertical Wind Load</u>: calculated by multiplying a 0.020 ksf vertical wind pressure by the out-to-out bridge deck width.

Width = 44.0 ft
L_{windT} = ~~max~~ 50.0 ft

∴ total wind load $\Rightarrow WS_{vert} = 0.02\,ksf \cdot (width)(L_{windT})$

$$= WS_{vert} = 0.02(44)(50) \Rightarrow \boxed{WS_{vert} = 44.0\,K}$$

<u>Resulting Moment</u>:

$$M_{wind\_vert} = WS_{vert} \cdot \frac{width}{4} \Rightarrow M_{wind\_vert} = 44.0K \cdot \frac{44\,ft}{4} \Rightarrow \boxed{M_{wind\_vert} = 484\,K \cdot ft}$$

<u>Bearing Reactions</u>

$$RWS_{vert\,1} = \frac{-WS_{vert}}{5} + \frac{M_{wind\_vert}(19.5\,ft)}{950.63\,ft^2 \,(I_{girders})} = \frac{-44.0K}{5} + \frac{484(19.5)}{950.63\,ft^2} = \boxed{1.13\,K}$$

$$RWS_{vert\,2} = \frac{-44}{5} + \frac{484(7.75)}{950.63\,ft^2} = \boxed{-4.85\,K}$$

$$RWS_{vert\,3} = \frac{-44}{5} = \boxed{-8.8\,K}$$

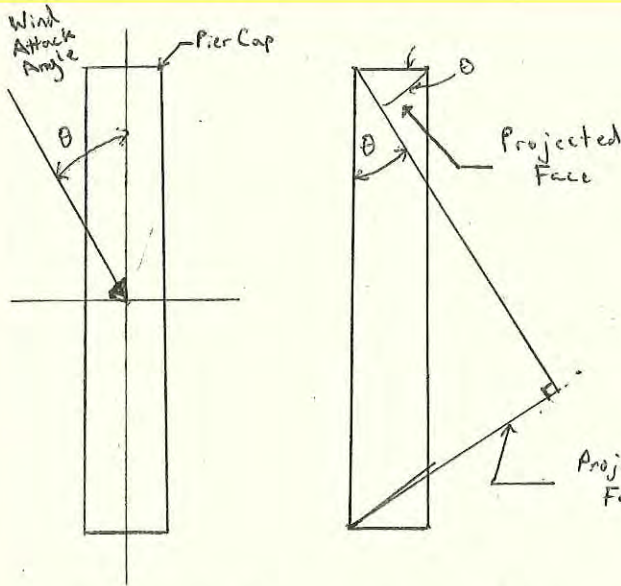$$RWS_{vert\,4} = -8.8K - \frac{484(7.75)}{950.63\,ft^2} = \boxed{-12.75\,K}$$

$$RWS_{vert\,5} = -8.8K - \frac{484(19.5\,ft)}{950.63\,ft^2} = \boxed{-18.73\,K}$$

Wind Load on Vehicles:

$0° => (0.10 \, k/ft)$ $60° => (0.034 \, Klf)$

$WL_{trans\,0} = L_{wind\,T} \cdot (0.10 \, k/ft) = 50(0.10 \, k/ft) => \boxed{WL_{trans\,0} = 5 \, K}$

$WL_{trans\,60} = L_{wind\,L} \cdot (0.034 \, k/ft) = 100(0.034 \, k/ft) => \boxed{WL_{trans\,60} = 3.4 \, K}$

Wind Load on Substructure:

(wind loads acting directly on substructure units calculated from base wind pressure of 0.040 ksf

Calculation Scenario, Attack Angle of 30°.

Component Areas

$A_{cap\,1} = (11.0 \, ft) \cdot (5.0 \, ft) => A_{cap\,1} = 55.00 \, ft^2$

$A_{cap\,2} = (11.0 \, ft)(46.5 \, ft) => A_{cap\,2} = 511.50 \, ft^2$

Projected Area of pier cap:

$AP_{cap} = A_{cap\,1} \cdot \cos(30 \, deg) + A_{cap\,2} \cdot \sin(30°)$

$AP_{cap} = 47.63 + 255.75$

$AP_{cap} = 303.38 \, ft^2$

Component Areas of the Pier Column

$A_{col\,1} = (15 \, ft - 2 \, ft)(4.5 \, ft) \quad A_{col\,1} = 58.50 \, ft^2$

$A_{col\,2} = (15 \, ft - 2 \, ft)(15.5 \, ft) \quad A_{col\,2} = 201.50 \, ft^2$

Projected Area of Pier Column:

$AP_{col} = A_{col\,1} \cdot \cos(30°) + A_{col\,2} \cdot \sin(30°) \longrightarrow AP_{col} = 151.41 \, ft^2$

$\therefore$ Total Wind Force: $WS_{sub30} = 0.040 \, ksf \cdot (AP_{cap} + AP_{col})$

$\boxed{WS_{sub30} = 18.19 \, K}$

Solve for Transverse + Longitudinal Components:

$\boxed{WS_{sub\,30T} = 15.75 \, K}$

$\boxed{WS_{sub\,30L} = 9.10 \, K}$

Temperature Loading: Assume Total Force of 20 K, equally
divided amongst the bearings, on top

$TU_1 = 4.0 K$
$TU_2 = 4.0 K$
$TV_3 = 4.0 K$
$TV_4 = 4.0 K$
$TU_5 = 4.0 K$

Step 7: Analyze and Combine Force Effects

Initial Rectification: Assumption Regarding Superstructure
Dead Load Reactions

$\rightarrow R_{DCE} = 80 K$ $R_{DWE} = \cancel{39.20} K$ (Exterior girder) } Largely
15 k
$R_{DCE} = $ "   15 k   " (Interior girder) } Approximated

Also, Pier Self-Weight, shown again below:

$DL_{cap} = 313.88 K$ $DL_{stg} = 144.90 K$
$DL_{col} = 156.94 K$ $EV_{stg} = 49.50 K$

Load Factors: Reference Table 8-16 in Fhwa pier design

Multiple Presence Factors
(2 lanes => $m_2 = 1.00$)

Flexure from vertical loads

$FVt_{cap\_flexstr1} = 1.25 \cdot R_{DC1} + 1.50 \cdot R_{DW1} + 1.75 \cdot R_{u\_c} \cdot m_2$

$= 1.25 (80) + 1.50 (15) + 1.75 \cdot (105.44 K) => \boxed{FVt_{cap\_flexstr1} = 307.02 K}$

$ArmV4_{cap} = \cancel{} 3.875 ft$

$FV5_{cap\_flexStr1} = 1.25 \cdot (80) + 1.50 (15) + 1.75 (105.44) = 307.02 K$

$ArmV5_{cap} = 11.625 ft$

$M_{u\,cap\_str1} = (307.02 K)(3.875) + (307.02 K)(11.625 ft)$

$+ 1.25\, DL_{ovhg} \left(\frac{15.5 ft}{2}\right)$ $[DL_{ovhg} = 93.0 K]$

$\boxed{M_{u\,cap\_str1} = 5,659.7 ft \cdot K}$ (Flexure)

Approximation for the purposes of this design,
simplification of force effects:

Using ratio of example moment effect and our
designs approximation:

$$\frac{5,659.7}{10706 \ (fhwa)} \Rightarrow \underline{0.529} \quad \text{of example results}$$

∴ Shear from vertical loads

$$V_{ucap\_str1} = 1509 K (0.529) \Rightarrow \boxed{797.7 K = V_{ucap\_str1}}$$

Torsion from horizontal loads:

$$172.77 \ ft \cdot K \cdot (0.529) \Rightarrow \boxed{T_{ucap\_str1} = 91.4 \ ft \cdot K}$$

Flexure from vertical loads (SERVICE 1)

$$7471 \ ft \cdot K \cdot (0.529) \Rightarrow \boxed{M_{ucap\_ser1} = 3952.2 \ ft \cdot K}$$

Pier Column Force Effects

<u>Axial Force</u>: $2435 K (0.529) \Rightarrow \boxed{Ax_{col} = 1288.1 \ K}$

<u>Transverse moment</u>: $9061 \ ft \cdot K (0.529) \Rightarrow \boxed{M_{ut\ col} = 4,793.3 \ ft \cdot K}$

<u>Longitudinal moment</u>: $1928 \ ft \cdot K (0.529) \Rightarrow \boxed{M_{ul\ col} = 1,019.9 \ ft \cdot K}$

From Strength III, the factored transverse shear in the column is:

$$92.28 K (0.529) \Rightarrow \boxed{V_{ut\ col} = 48.8 \ K}$$

For Strength V, the factored longitudinal shear in the column is:

$$109.09 K (0.529) \Rightarrow \boxed{V_{ul\ col} = 57.7 \ K}$$

<u>Pier Pile Force Effects</u>: Look ahead to Design Step 10

<u>Pier Footing Force Effects</u>:

$$3583 K (0.529) \Rightarrow \boxed{Ax_{col\_punch} = 1895.4 \ K}$$

$$5287 \ ft \cdot K (0.529) \Rightarrow \boxed{M_{ut\ col\_punch} = 2796.8 \ ft \cdot K}$$

$$2756 \ ft \cdot K (0.529) \Rightarrow \boxed{M_{ul\ col\_punch} = 1457.9 \ ft \cdot K}$$

Step 8: Design Pier Cap

Strength 1

$$M_{ucap-str1} = 5,659.7 \, ft \cdot K$$

$$V_{ucap-str1} = 797.7 \, K$$

$$T_{ucap-str1} = 91.4 \, ft \cdot K$$

Service 1

$$M_{ucap-ser1} = 3952.2 \, ft \cdot K$$

↓ Continue into Rebar Design ↓
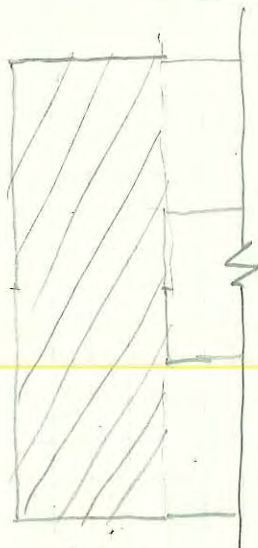
$$4.5 \quad M_u = (6.7\,Ksf)\left(\frac{12' - 4.5'}{2}\right)^2\left(\frac{.12'}{2}\right)$$

$$M_u = 565.31\ K\text{-}ft$$

$$C = T$$

$$C = .85 f'_c\, ba \cdot \quad T = A_s f_y$$

$$a = \frac{A_s f_y}{.85 f'_c\, b}$$

$$\frac{l-c}{2}$$

$$a = \frac{A_s (60\,ksi)}{.85(4\,ksi)(12')} = .123\, A_s$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) \qquad \phi M_n = \phi A_s (60\,ksi)\left(31.5'' - \frac{.123 A_s}{2}\right)$$

$$\phi M_n \longrightarrow M_u \qquad \qquad .125.6 = \left(31.5'' - \frac{.123 A_s}{2}\right) A_s$$

$$.151 = 31.5'' A_s - \frac{.123 A_s^3}{2} \qquad A_{s\,req} = 256\,in^2$$

$$\rho_1 = 0.0018 (12')(12'')(36\,in) = 9.33$$

$$A_{smin} < 9.53\,in^2 \le 256\,in^2$$

1 #8 bar @ 1' spacing   324 bars

$$\varepsilon_1 = \frac{\varepsilon_c}{c}(d - c) \qquad c = \frac{a}{\beta_1} \qquad a = .28 A_s$$

$$c = \frac{.1\,(324)\,(0.79\,in^2)}{.85} = 30\ in$$

$$\varepsilon_1 = \frac{.003}{30\,in}(31.5\,in - 30\,in) = .00015 \le 0.06 \ \checkmark$$

$$\phi V_c = \phi \, 2\sqrt{f'c} \; b_w d \qquad \phi V_c = \phi \, 2\sqrt{4000 psi} \, (12')(31.5'')$$

$$\phi V_c = 430^{K}$$

$$\phi V_c = 430^{K} \geq V_u = 318^{K} \qquad \text{One-Way} \checkmark$$

$$h = 31.5''$$

2 way Shear

$$b_o = 4(c+d) \qquad b_o = 4(4.5' + 31.5'') \qquad b_o = 342 \, in$$

a) $V_c = 4\lambda\sqrt{f'c}$ \qquad b) $V_c = \left(2 + \dfrac{4}{\beta}\right)\lambda\sqrt{f'c}$ \qquad $\beta = \dfrac{23'}{12'}$

$V_c = 4(1.0)\sqrt{4000}$ \qquad $V_c = \left(2 + \dfrac{4}{2}\right)1\sqrt{4000}$

$V_c = 253 \, psi$ \qquad $V_c = 253 \, psi$

$$\boxed{V_c = 253 psi} \longleftarrow \text{governs}$$

$$V_c = \left(\dfrac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'c} \qquad V_c = \left(\dfrac{40\,(31.5 in)}{342''} + 2\right)\sqrt{4000}$$

$$V_c = 359 \, psi$$

$$V_c = \underline{253 psi\,(342 in)(31.5 in)} \qquad V_c = 2725.6^{K}$$

$$\phi V_c = 2044.2$$

$$V_u = q_u\left[a^2 - (c+d)^2\right] \qquad V_u = 4 ksf\left[12' \times 23' - (4.5' + 31.5'')^2\right]$$

$$V_u = 901$$

$$\phi V_c = 2044.2 \geq 901 = V_u \qquad \text{Two-way} \checkmark$$

Rectangular Spread Footing

Soil bearing pressure = 4 ksf

$f'_c = 4000 \, psi$

$f_y = 60000 \, psi$

$\lambda = 1 \quad 130 \, pcf$

Loading

D + L 1407 K on Column

$1407 K + (Column = 15' \times 15.5' \times 4.5') \, 150 \, pcf = 1564^K$

Footing Area = 23' × 12'

soil self weight = concrete self weight $\longrightarrow$ neglect

Area = 276 ft²   Depth = 36" = T

23' = L

12' = B
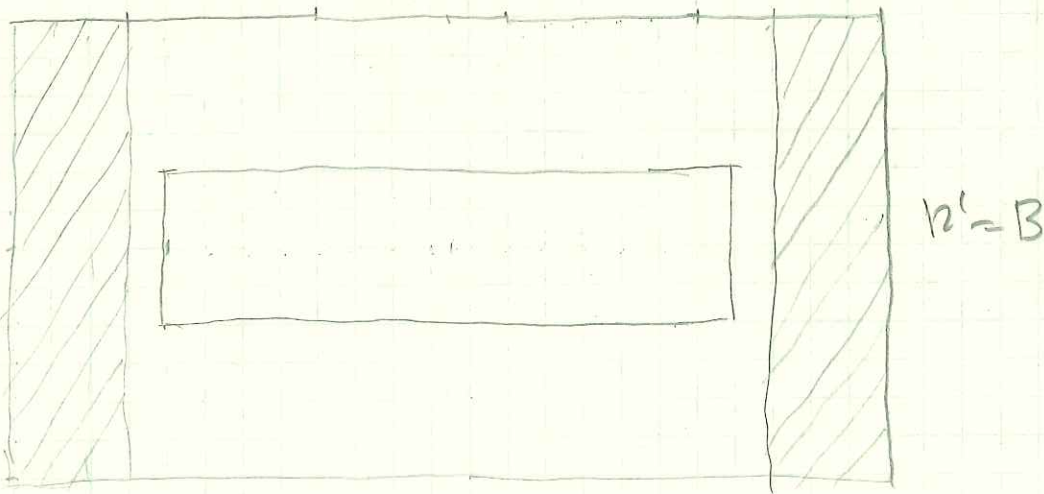
Assume #8 Bars $\phi = 1''$ Area = .79 in²

$d = 27'' = 36'' - 3'' \, Cover - 1'' - \frac{1}{2}'' = 31.5''$

$\phi V_n \geq V_c \quad V_n = V_c + V_s$ [assume $V_s = 0$]

$$\phi V_n \geq V_u = \left[\frac{l}{2} - \frac{c}{2} - d\right] b q_u$$

$$V_u = \left(\frac{23'}{2} - \frac{4.5'}{2} - \frac{31.5''}{12''}\right)(4 \, ksf/ft)(12')$$

$$V_u = 318 \, K/ft$$

Appendix J

Deep Foundation Design.

Drilled underreamed ("Bell") shaft. (42 ft deep)

- <u>Reinforcement.</u>

- Minimum shaft diameter. (B)

$$B = \sqrt{\frac{3.86 P}{f'_c}} \qquad \rightarrow \qquad P\text{(compressive Load)} = 2435 \text{ Kips}$$
$$f'_c \text{ (28-day compressive strength)} = 4000 \text{ psi}$$

$$B = 1.53 \text{ ft.} \rightarrow \boxed{\text{use } B = 24 \text{ in}}$$

- underreamed diameter (B$_s$)

$$\frac{B_s}{B} < 3.$$

$$\boxed{\text{Try } B_s = 54 \text{ in}} \qquad \rightarrow \qquad \frac{54}{24} = 2.25 < 3. \checkmark \quad \text{OK.}$$

- Find γ.

spiral steel — Longitudinal steel

I 3in.

γB

B
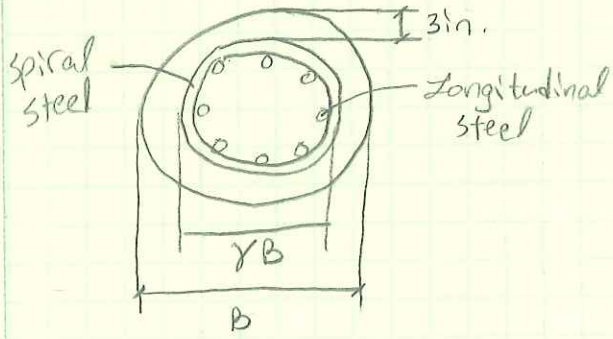
- Assume # 8 bars for longitudinal steel
- ½ in spiral reinforcement.
- 3 in clear cover (required by ACI)
- Diameter of longitudinal steel circle is approximately B-7.8 in.

$$\gamma = \frac{B-7.5}{B} = \frac{24 \text{ in} - 7.5}{24 \text{ in}}$$

$$\boxed{\gamma = 0.69}$$

- $P = 2435$ Kips } From Pier calculations.
  $M = 9061$ Kip-ft. }

$$A = \frac{\pi (B)^2}{4} = \frac{\pi (24 \text{ in})^2}{4} = 452 \text{ in}^2$$

$$\frac{P}{A} = \frac{2435 \text{ Kip}}{452 \text{ in}^2} = 5.38 \text{ Kip/in}^2$$

$$\frac{M}{AB} = \frac{9061 \text{ Kip-ft} \times 12 \text{ in/ft}}{(452 \text{ in}^2)(24 \text{ in})} = 10.01 \text{ Kip/in}^2$$

using Figures 12.6-12.9 in Coduto, D. P. (2001)

$$\gamma = 0.6 \rightarrow \rho = 0.055$$
$$\gamma = 0.75 \rightarrow \rho = 0.038.$$

$$\rho = 0.69 + \frac{(0.69 - 0.6)}{(0.75 - 0.6)} (0.038 - 0.055)$$

$$\boxed{\rho = 0.045}$$

Deep Foundation Design (cont.)

$$(A_s)_{req'd} = \frac{\rho \cdot \pi B^2}{4} = \frac{0.045 \pi (24 in)^2}{4} = 20.4 \, in^2$$

— Use # 10 bars

- spiral reinforcement

$$A_c = \pi \frac{(24 in - 6 in)^2}{4} = 254 \, in^2$$

$$\rho_s = 0.45 \left(\frac{A}{A_c} - 1 \right) \frac{f'_c}{f_y} = 0.45 \left(\frac{452 \, in^2}{254 \, in^2} - 1 \right) \frac{4000}{60000}$$

$$\rho_s = 0.023$$

- Volume of steel per turn = $18 \pi A_s = 56.5 A_s$
- Volume of core per turn = $\frac{\pi 18^2 \rho}{4} = 254 \rho$ $P = pitch.$

$$\rho_s = \frac{56.5 A_s}{254 \rho} = 0.023$$

- For # 6 bars; $A_s = 0.44 \, in^2$, $P = 4.2 \, in$

— Reinforcement recommendations

use 24 inch diameter shaft w/ 16 longitudinal #10 bars
& # 6 bars at 4 inches on center.

Deep Foundation Design (cont.)

- Net Toe Bearing Resistance

$$q'_t = 1200 \, N_{60} \leq 60,000 \, ^{lb}/ft^2$$

$$q'_t = 1200(29) \leq 60000 \, ^{lb}/ft^2$$

$$\boxed{q'_t = 34800 \, ^{lb}/ft^2 \leq 60000}$$

- Side-friction resistance (f_s)

- The β Method was used to calculate f_s because the foundation is in soil layers of sand.

$$f_s = \beta \sigma'_z$$

$$\beta = 1.5 - 0.135 \sqrt{z} \qquad \rightarrow \qquad z = depth \ to \ midpoint \ of \ soil \ layer.$$

$$0.25 \leq \beta \leq 1.2 \qquad (Table \ on \ the \ next \ page \ shows$$
$$\beta \ value \ for \ each \ layer)$$

Layer No.	f_s (ksf)	A_s (ft^2)
1	1.08	115.61
2	1.83	25.13
3	3.74	201.06
4	5.04	118.12

- $P_{(down \ allowable)} = \dfrac{q'_t A_t + \Sigma f_s A_s}{F.O.S.} = \dfrac{(34.8 \, ksf)(286.3 \, ft^2) + 1518.7 \, (k)}{3}$

$$\boxed{P_{(down \, a)} = 3827.06 \, kip} > P_u = 2435 \, kip. \qquad \checkmark OK$$

- $P_{(up \, allow)} = \dfrac{A_b \sigma'_z * 3.5 / 1000}{F.O.S.} = \dfrac{12.76 \, ft^2 \, (4200 \, psf) * 3.5 / 1000}{3}$

$$P_{(up \, allow)} = 62.5 \, kip.$$

Foundation Cost Analysis
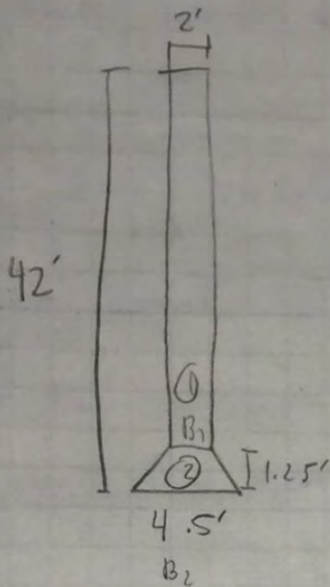
$$V_1 = \pi r^2 h = \pi (1.6 \, \text{ft})^2 (42 \, \text{ft}) = 131.95 \, \text{ft}^3$$

$$V_2 = L \times H \times \frac{(B_1 + B_2)}{2} = (1.25')\frac{(4.5' + 2')}{2}(2) = 8.13$$

$$\cdot \, V = 140.1 \, \text{ft}^2 \times 0.037037 \, \frac{yd^2}{ft^2}$$

$$\underline{V = 5.19 \, yd^2}$$

Substructure Concrete Construction Costs

Labor: $(466.19 \, \$/yd^3)(5.19 \, yd^2) = \underline{\$2419.53}$

Equipment: $(\$82.67/yd^3)(5.19 \, yd^2) = \underline{\$429.06}$

Job materials: $(\$143.34/yd^3)(5.19 \, yd^2) = \underline{\$743.93}$

Permanent materials: $(\$194.65/yd^3)(5.19 \, yd^2) = \underline{\$1010.23}$

$$\boxed{Total = \$4649.70}$$

Excavation and Back fill Cost

Quantity (yd^3) : $V_{backfill} = lb = (4.5')(42')(2') - \pi(1 \, ft)^2(42')$

$$V_b = 246.1 \, ft^3 \times 0.037037 \, yd^3/ft^3$$

$$\underline{V_b = 9.11 \, yd^3}$$

$V_{excavation} = V_x = (4.5')(4.5')(42') = 378 \times 0.037037$

$$\underline{V_x = 31.5 \, yd^3}$$
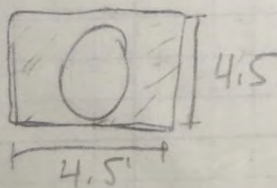
Quantity : $\underline{14 \ yd^3}$

Labor : $(31.5 \ yd^3)(\$6.32/yd^3) = \underline{\$199.08}$

Equipment : $(31.5 \ yd^3)(5.62\$/yd^3) = \underline{\$177.03}$

Job materials : $(31.5 \ yd^3)(0.11 \ \$/yd^3) = \underline{\$3.47}$

permant materials : $(31.5 \ yd^3)(6.17 \ \$/yd^3) = \$194.36$

$\boxed{total = \underline{\$578.04}}$

Dewatering Costs $\qquad$ $(Quantity = 1)(\$16,495) = \underline{\$16,495}$

Total Cost of Foundation $\qquad$ $\boxed{\$21,722.14}$

Total cost rounded up : $\boxed{\$22,000}$

Excel Spreadsheet

	A	B	C	D	E	F	G	H	I	J	K	L
1	Deep foundation Analysis											
2												
3												
4	**Preliminary**			**Reinforcement**				**Final Design**				
5	Pu (k)	2435		γ	0.69			Use 24 in diameter shaft with 16 longitudinal #10 bars				
6	Mu (k-ft)	9061		A (sqin)	452.3893			and #6 spiral reinforcement at 4 in on center				
7				Pu/A(k/sq	5.382532							
8				Mu/AB(k/	0.83455							
9	Assumptions			As(req)(sc	20.35752							
10	**Use # 8 bars**											
11	**1/2 in diameter spiral**			**Use # 10 bars**				**Toe Bearing**				
12	**3in Cover (ACI)**							q't(ksf)	34.8			
13				Ac(sqin)	254.469			q'tr(ksf)	33			
14	**Dimensions**			ρs	0.023333			At(ft^2)	286.2776			
15	B min(ft)	1.5328976		ρs=56.5As/254p								
16	Bb(ft)	4.5328976		As(sqin)	0.44							
17	B(in)	24		p(in)	4.2							
18	Bb(in)	54										
19	Depth(ft)	42		**Use #6 bars at 4 in on center**								
20	Ab(ft^2)	12.76272										
21												
22	**Beta Method**											
23												
24	**Layer Thickness**	**Depth(ft)**	**Layer No.**	**N60**	**Zf**	**β**	**β to use**	**σ'z**	**fs**	**As**		
25	7	29-36	8	29	32.5	0.730382	0.73038159	4500	3286.717	14		
26												
27	**Layer No.**	**H**	**Zf**	**β**	**β(use)**	**σ'z**	**fs(ksf)**	**As(ft^2)**				
28	1	7.2	3.6	1.243856	1.2	900	1.08	115.6106097				
29	2	5	2.5	1.286546	1.2	1525	1.83	25.13274123				
30	3	14	7	1.142824	1.142824	3275	3.7427472	201.0619298				
31	4	7.4	3.7	1.240322	1.2	4200	5.04	118.1238838				
32							ΣfsAs	1518.720724				
33												
34	Pdown(kips)	3827.0608										
35	Pup(kips)	62.537329										
36												